Design Manual for Roads and Bridges

Highway Structures & Bridges
Design

CD 354
Design of minor structures

(formerly BD 94/17)

Version 1.1.0

Summary
This document covers the design of minor highway structures including lighting columns, cantilever masts for traffic signals and/or speed cameras, CCTV masts, and fixed vertical road traffic signs. It incorporates the provisions of BS EN 40, BS EN 12899 and supersedes BD 94/17.

Application by Overseeing Organisations
Any specific requirements for Overseeing Organisations alternative or supplementary to those given in this document are given in National Application Annexes to this document.

Feedback and Enquiries
Users of this document are encouraged to raise any enquiries and/or provide feedback on the content and usage of this document to the dedicated National Highways team. The email address for all enquiries and feedback is: Standards_Enquiries@highwaysengland.co.uk

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Latest release notes

Document code	Version number	Date of publication of relevant change	Changes made to	Type of change
CD 354	1.**1**.0	February 2022	Core document	Incremental change to requirements

Updated to rectify non technical errors noted by users. Correcting errors in weld classifications in Appendix A. Informing TPB that conversion errors displayed formula differently in PDF than was displayed in CARS. Both issues addressed - also some editorial changes.

Previous versions

Document code	Version number	Date of publication of relevant change	Changes made to	Type of change
CD 354	1	March 2020		
CD 354	0	December 2019		

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Foreword

Publishing information

This document is published by National Highways.

This document supersedes BD 94/17, which is withdrawn.

Contractual and legal considerations

This document forms part of the works specification. It does not purport to include all the necessary provisions of a contract. Users are responsible for applying all appropriate documents applicable to their contract.

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Introduction

Background

This document describes the requirements for the design of minor structures. It gives guidance on the information required to allow the suitable design of lighting columns, cantilever masts, CCTV masts, road traffic sign/signal posts, environmental barriers, high masts, and other mast type structures.

This document covers the use of BS EN 40 [Ref 18.N], PLG 07 [Ref 28.N], BS EN 12899-1 [Ref 14.N] and BS EN 14388 [Ref 32.N] as relevant for the design of specific types of minor structures.

It sets out the Overseeing Organisation's requirements where these augment, or are additional to those given in the British Standard. In addition, the document gives the requirements for lighting columns made essentially from glass fibre reinforced plastic (FRP or FRPC). The technical basis for the clauses on FRPC is limited and it may become necessary to review the requirements in due course, on the basis of their performance in service.

Assumptions made in the preparation of this document

The assumptions made in GG 101 [Ref 17.N] apply to this document.

Mutual Recognition

Where there is a requirement in this document for compliance with any part of a "British Standard" or other technical specification, that requirement may be met by compliance with the Mutual Recognition clause in GG 101 [Ref 17.N].

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Abbreviations

Abbreviation	Definition
CCTV	Closed-circuit television
FRP	Fibre-reinforced polymer
FRPC	Fibre-reinforced polymer composite

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1. Scope

Aspects covered

1.1 This document shall be used for the structural design of the following minor structures:

1) lighting columns and wall mounted brackets made from concrete, steel, aluminium, and FRPC, including lighting columns mounted on other structures such as on bridges;
2) steel CCTV masts mounted on foundations in the ground;
3) cantilever masts made from steel for traffic signals and/or speed cameras;
4) fixed vertical road traffic sign/signal posts;
5) environmental and other 'fence type' barriers;
6) high masts; and,
7) other mast type structures.

NOTE 1 Requirements for the assessment or design of other structures and structural elements onto which a minor structure is mounted are outside the scope of this document.

NOTE 2 The collective term 'minor structures' is used to describe those structures within this clause.

NOTE 3 The scope of this document is limited to minor structures within the identified dimensions and parameters listed in the following clauses.

1.2 Where different structural materials such as aluminium, concrete, steel or FRPC are proposed for other minor structures, the design methods and criteria, performance limits and specification shall be:

1) in accordance with the standards relevant for the proposed materials; and
2) agreed with the Overseeing Organisation.

NOTE 1 The design requirements for lighting columns are covered in the relevant parts of BS EN 40 [Ref 18.N].

NOTE 2 The design requirements for aluminium, concrete and steel are given in the relevant parts of BS EN 1999 [Ref 13.N], BS EN 1992 [Ref 4.N] and BS EN 1993 [Ref 1.I].

1.3 This document shall not be used for the design of temporary or permanent cantilever sign/signal gantries.

NOTE The design requirements for portal and cantilever sign/signal gantries are given in CD 365 [Ref 26.N].

1.4 This document shall not be used for the design of lattice structures.

NOTE Structural requirements for lattice structures are covered by BS EN 1993-3-1 [Ref 6.N].

1.5 Structural requirements for passively safe minor structures shall be in accordance with this document but the passive safety characteristics of such structures are dealt with in BS EN 12767 [Ref 4.I].

Implementation

1.6 This document shall be implemented forthwith on all schemes involving the design and construction of minor highway structures on the Overseeing Organisations' motorway and all-purpose trunk roads according to the implementation requirements of GG 101 [Ref 17.N].

Use of GG 101

1.7 The requirements contained in GG 101 [Ref 17.N] shall be followed in respect of activities covered by this document.

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2. General principles

2.1 The siting of minor structures shall be in accordance with CD 109 [Ref 15.N], CD 146 [Ref 27.N], CD 377 [Ref 29.N], and TD 501 [Ref 30.N], as shown in Table 2.1 below.

Table 2.1 DMRB documents relevant for the siting of minor structures

Type of minor structure	DMRB documents relevant to the structure under consideration			
	CD 109	CD 146	CD 377	TD 501
Lighting columns	✓	✓	✓	✓
Cantilever masts	✓	✓	✓	
CCTV masts	✓		✓	
Road traffic signs	✓	✓	✓	
Environmental barriers and screens	✓	**	✓	**
Other mast type structures	✓	✓*	✓	✓
Notes/key: * applicable depending on type of mast ** applicable depending on location and impact				

2.2 The siting of minor structures shall include an assessment of visibility by the approaching traffic.

2.2.1 Cantilever masts should not be located on bridges.

Layout

2.3 All elements of minor structures shall have clearances in accordance with CD 127 [Ref 2.N] after allowing for deflections due to dead, live, wind and high vehicle buffeting loads.

2.4 The clear new construction headroom for routes other than high load routes shall be as defined in CD 127 [Ref 2.N] for footbridges and sign/signal gantries.

2.5 Where cantilever masts are sited on high load routes, the clear new construction headroom as defined in CD 127 [Ref 2.N] for all permanent structures over high load routes shall be taken as the minimum allowable clear headroom.

2.6 Settlement of the structure as well as structural deformations shall be factored into the calculation of headroom.

2.7 The requirements for the vehicle restraint system shall be agreed with the Technical Approval Authority.

2.8 The setback of the vehicle restraint system to the edge of the carriageway shall be agreed with the Overseeing Organisation.

2.9 Where passively safe signposts, lighting columns or traffic signal posts are provided in accordance with BS EN 12767 [Ref 4.I], further vehicle restraint systems shall not be required.

2.9.1 Further vehicle restraint should only be provided where required by the existence of other hazards.

NOTE Other hazards can include new/existing road infrastructure, a change in the ground profile or where existing infrastructure is relocated.

2.10 The clearance from the front of the vehicle restraint system to the face of the minor structure shall be selected from the working width given in BS EN 1317-2 [Ref 31.N].

Protection for road users and structure

2.11 Protection for minor structures shall be in accordance with CD 377 [Ref 29.N].

2.12 Positioning of cantilever masts and CCTV masts at locations not in accordance with CD 377 [Ref 29.N] shall be subject to the approval of the Technical Approval Authority.

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2.13 Where the post of the cantilever mast or CCTV mast is located behind a vehicle restraint system meeting the requirements of BS EN 1317-2 [Ref 31.N], further vehicle restraint systems shall not be required.

Equipment

2.14 All ancillary equipment such as luminaires, lanterns, brackets, signs, traffic signals, speed cameras and associated equipment shall be securely attached to the structure.

2.15 Vibration-resistant fixings shall be used to withstand the design loads.

2.16 The structural design shall make provision for the attachment of equipment.

2.17 The structural design shall include redundancy checks (i.e. can the failure of a single item, such as a bolt, cause the failure of the entire system?).

2.18 Any subsequent modifications to structural members shall only be carried out with the approval of the Technical Approval Authority in accordance with CG 300 [Ref 34.N].

In-situ connections

2.19 In-situ connections of main structural metal elements shall be by means of bolts.

2.20 Where other forms of in-situ connection are proposed, their static and fatigue design strength shall be calculated from first principles and agreed with the Technical Approval Authority.

2.20.1 Alternatively, the design strength may be based on the results of full-scale load tests, subject to the agreement of the Technical Approval Authority.

Identification

2.21 Identification marking of structures shall be:

1) in accordance with CG 305 [Ref 16.N]; or,
2) agreed with the Overseeing Organisation.

2.22 Minor structures that have been designed to be passively safe to BS EN 12767 [Ref 4.I] shall be marked to differentiate them from other types of structures.

2.22.1 Where the form of structure is unique and easily recognisable, specific marking may not be necessary.

NOTE Marking allows the structure to be easily recognised during a road safety audit.

2.23 The marking system shall incorporate the phrase "Crash Friendly" and be placed on the post or column in a position that does not affect the functionality of any part of the assembly or the identification marks required by CG 305 [Ref 16.N] or the Overseeing Organisation.

2.24 The form of marking appropriate for individual products shall be agreed with the Overseeing Organisation.

Use of dissimilar metals

2.25 Where dissimilar metals are to be used, the connections shall be designed to avoid the risk of galvanic corrosion.

2.26 The electrical bonding of all metal components shall be maintained.

Protection against corrosion

2.27 Surface preparation and paint protection of steel shall be in accordance with MCHW Series 1900 [Ref 25.N].

NOTE Preparation and protection is applicable to all parts of the structure including fasteners (bolts, nuts, washers, packing plates), stiffeners, brackets, etc.

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2.28 Materials, other than steel, shall have a life expectancy greater than the service life. (such as from galvanic corrosion of aluminium due to local ground conditions and ultraviolet degradation of FRPC columns).

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3. Dimensional limitations

Lighting columns

3.1 Dimensional requirements for lighting columns are given in BS EN 40-2 [Ref 21.N] and the overall dimensional limitations for lighting columns covered by this document (for steel, aluminium, FRPC and concrete columns) shall be:

1) post top columns <20 metres nominal height;
2) columns with brackets <18 metres nominal height; and,
3) bracket projections not exceeding the lesser of 0.25 x nominal height or 3 metres.

NOTE Nominal heights and bracket projections are defined in BS EN 40-2 [Ref 21.N].

CCTV masts

3.2 The nominal height of steel CCTV masts covered by this document shall be less than or equal to 25 metres.

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Figure 3.2a General arrangement of mast and CCTV

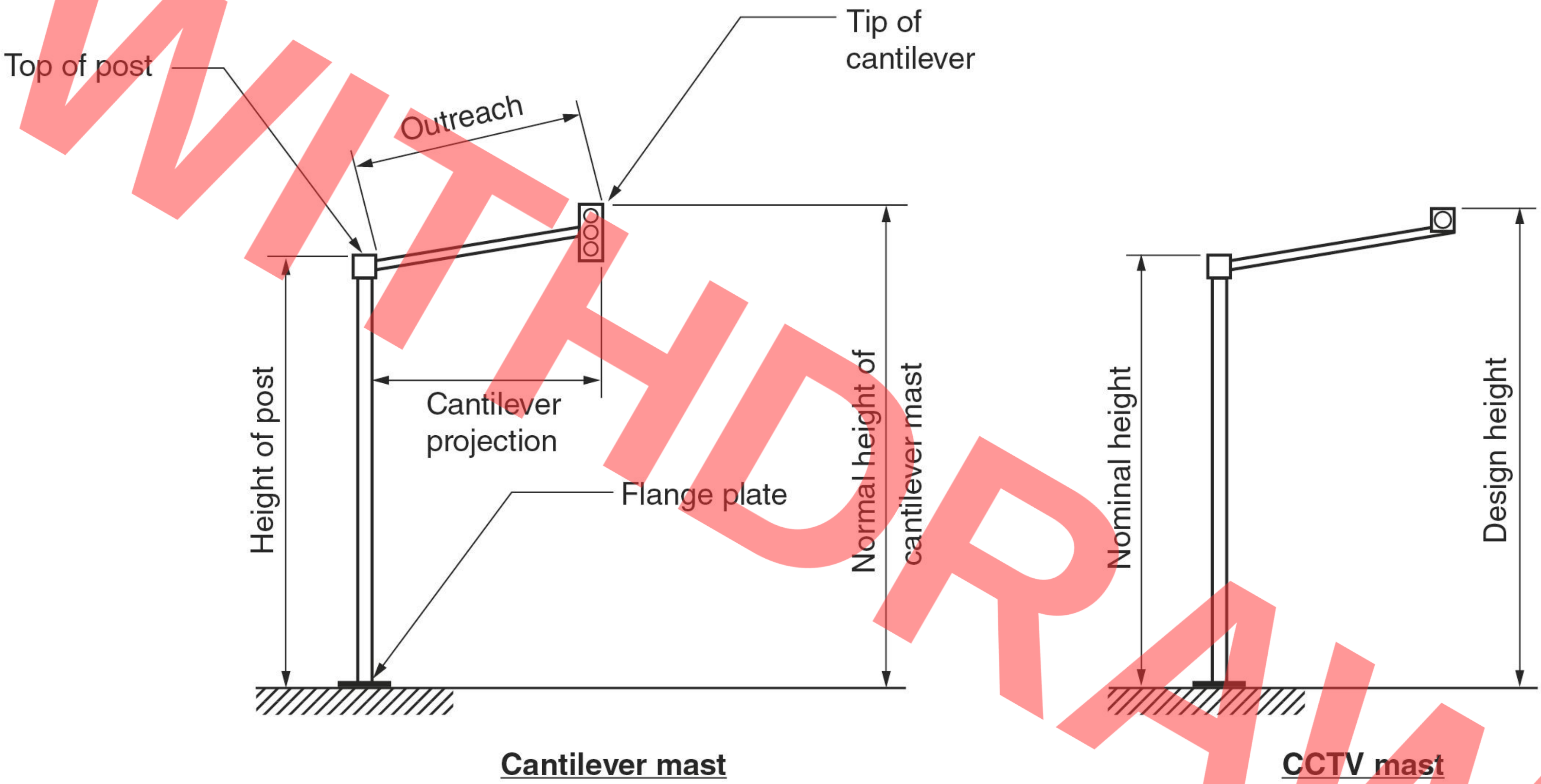

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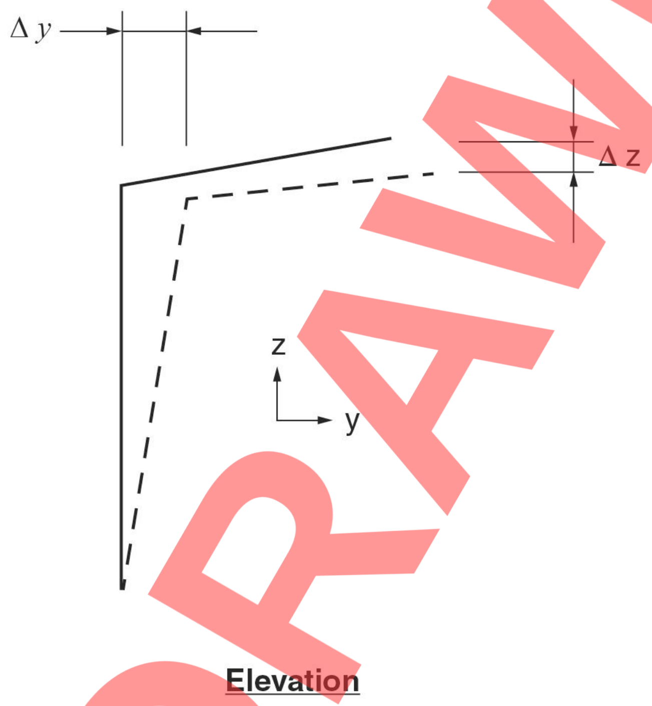
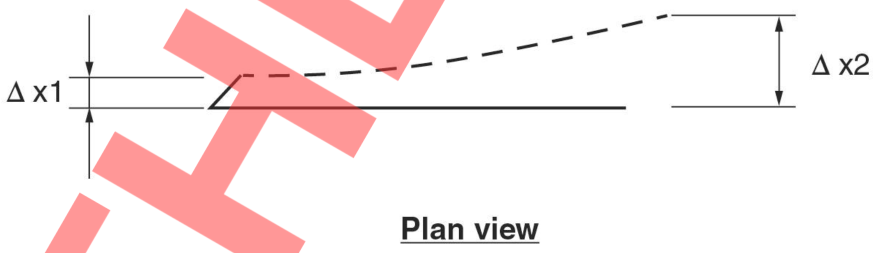
Figure 3.2b Structural deformation of cantilever mast





NOTE 1 *For CCTV masts the nominal height excludes the height of camera, mounting, etc. (refer to Figure 3.2a).*

NOTE 2 *Values for the deformation limits shown in Figure 3.2b are given in Section 5.*

3.3 The design height of a CCTV mast shall be taken as the vertical distance between the underside of the flange plate and the top of the CCTV mast or camera in its operating position, or other attachments, whichever is greater.

NOTE *The "design height" is different to the "nominal height" and is required for wind loading calculations (refer to Figure 3.2a).*

3.4 For CCTV masts, the definitions given in PLG 07 [Ref 28.N]: Section 1.4, shall be interpreted as follows:

1) HIGH MAST also refers to CCTV masts, meaning the support intended to hold one or more CCTV camera with their mountings and housings; and,
2) the term LUMINAIRE includes CCTV cameras, their mountings and housings.

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3.4.1 CCTV masts less than 10 m in height may be designed using PLG 07 [Ref 28.N].

Cantilever masts

3.5 The following shall apply for steel cantilever masts, as shown in Figure 3.2a:

1) nominal height ≤8.5 metres; and,
2) cantilever projection ≤8.5 metres.

NOTE The nominal height is taken as the vertical distance between the underside of the flange plate and the highest point on the mast (refer to Figure 3.2a).

3.6 The horizontal projected area of any signs, traffic signals, speed cameras and associated equipment, suspended above the carriageway shall not exceed 1.20 m^2.

3.7 The vertical projected area of any signs, traffic signals, speed cameras and associated equipment shall not exceed 0.3 m^2

Traffic sign/signal posts

3.8 The nominal height of steel traffic sign / signal posts shall be ≤9 metres.

3.9 Dynamic and fatigue checks shall be carried out for traffic sign/signal posts above nine metres nominal height.

Environmental barriers

3.10 The requirements for environmental barriers shall be agreed with the the Overseeing Organisation.

Other mast type structures

3.11 The nominal height of other mast type structures shall be agreed with the Overseeing Organisation based on the required end use.

3.11.1 Minor structures such as CCTV masts, cantilever masts or traffic sign/signal posts made from materials such as concrete, aluminium or FRPC may have dimension limits different to similar structures made from steel.

3.11.2 The dimensional limitations given for lighting columns, steel CCTV masts, steel cantilever masts, traffic sign/signal posts or environmental barriers may be used as guidance for determining the dimension limits for similar structures which are not made from steel.

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4. Use of British Standards and standards issued by the Overseeing Organisations

4.1 Minor structures shall be designed in accordance with the following:

1) lighting columns and cantilever signal masts - the relevant parts of BS EN 40 [Ref 18.N];
2) CCTV masts - PLG 07 [Ref 28.N];
3) environmental barriers - BS EN 14388 [Ref 32.N]; or,
4) road traffic sign posts - BS EN 12899-1 [Ref 14.N].

NOTE 1 Guidance and background information in the use of BS EN 40-3-1 [Ref 19.N] and BS EN 40-3-3 [Ref 20.N] for the design of lighting columns is given in PD 6547 [Ref 2.I].

NOTE 2 This document covers the use of PLG 07 [Ref 28.N] for the design of CCTV masts. This guide was originally developed for high mast lighting and has been revised to include CCTV masts as they have similar features.

NOTE 3 The document gives minimum requirements for the design of components of fixed vertical road signs that would otherwise fall within the scope of BS EN 12899-1 [Ref 14.N]. The electronic design requirements of certain traffic signs as defined in BS EN 12899-1 [Ref 14.N] are excluded from this document.

4.2 The installation of minor structures shall be in accordance with the requirements of the design and manufacturers' instructions.

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5. Design

General requirements

5.1 Minor structures shall be designed in accordance with the requirements of the design guide and standards given in Section 4 as applicable for the type of structure.

5.2 The design of minor structures shall be subject to technical approval in accordance with CG 300 [Ref 34.N].

5.3 Minor structures in very exposed areas shall be classified as Category 1 in accordance with CG 300 [Ref 34.N].

5.4 Very exposed sites shall be defined as follows:

1) sites at high altitude, above 250 metres;
2) sites within 5 km from the coast; or,
3) sites subject to significant local funnelling.

Structural criteria

5.5 The design life shall be 25 years, unless otherwise required by the Technical Approval Authority.

5.5.1 For some minor structures such as environmental barriers, the design life required for the superstructure may be different to that of the substructure.

NOTE The design life of the foundation can be much longer where the superstructure is to be renewed on the existing foundations.

Limit states

5.6 Minor structures shall be designed to satisfy the relevant ultimate limit states and the serviceability limit state.

5.7 Where steel structures are used they shall be designed to meet the fatigue criteria.

Lighting columns

5.8 For lighting columns, the partial safety factors and criteria for serviceability and ultimate limit states shall be taken as Class B, as given in BS EN 40-3-3 [Ref 20.N].

5.9 Horizontal deflections of each lantern connection shall conform to class 2 in Table 4 of BS EN 40-3-3 [Ref 20.N].

CCTV masts

5.10 For CCTV masts, the partial safety factors and criteria for serviceability and ultimate limit states shall be as given in PLG 07 [Ref 28.N], clauses 2.4 and 2.5.

5.11 The torsional rotation and linear deflection limits shall be:

1) in accordance with serviceability limit state given in clause 2.3.2.3 of PLG 07 [Ref 28.N]; and,
2) verified for a wind load derived by setting $p = 0.9$ in Equation (4.2) in BS EN 1991-1-4 [Ref 3.N] ($c_{prob} = 0.684$.

5.12 The torsional rotation at the top of the mast shall not exceed 25 minutes of arc (0.0073 radians).

5.13 The linear deflection at the top of the pole shall not exceed 150 mm.

5.14 For serviceability limit state the calculation shall take full account of the actual weights of the CCTV mast, cameras, mountings, housings and any other attachments.

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5.15 Vehicle collision loads shall not be included for the serviceability limit state because of the requirements to protect CCTV masts in accordance with CD 377 [Ref 29.N].

5.16 The proposed limits shall be used unless more stringent rotation and deflection criteria are agreed with the Overseeing Organisation.

5.17 The operational use of the CCTV cameras shall not be affected by the proposed limits.

Cantilever masts for traffic sign/signals and/or speed cameras.

5.18 Where any permanent load has a relieving effect, γ_F shall be taken as 1.0 in both the ultimate limit state and serviceability limit state.

5.19 The partial factors, γ_F , in Table 5.19 below shall be applied for strength, fatigue and deflection verification.

Table 5.19 Limit states and partial factors

Limit state description	Partial factor on load, γ_F				
	Limit state type	Dead load	Superimposed dead load	Wind load	Buffeting from high vehicles
Strength (STR)	ULS	1.20	1.20	1.20	-
Fatigue	SLS	1.00	1.00	1.00	1.00
Deflection	SLS	1.00	1.00	1.00	0.50*

NOTE **The partial load factor, γ_F , given here in Table 5.19 is 0.5. This is because the design pressures for buffeting due to high-sided vehicles given in Section 11 have been set to calculate the total stress range experience. That is, the response from peak positive pressure to peak negative pressure. All that is required for calculation of headroom is the deflection due to peak negative pressure from the static equilibrium position. This is approximately half the peak-to-peak response, hence the partial load factor, γ_F , of 0.5.*

5.20 Cantilever masts for traffic signal/signals and/or speed cameras shall not be designed for vehicle collision loads.

5.21 For the serviceability limit state under combined effects of permanent loads acting with all the primary live loads, the deflections and rotations due to wind loading only, shall be limited such that the deformations do not exceed the values given in Table 5.21.

Table 5.21 Limiting structural deformations of cantilever masts (see Figure 3.2b)

Element and position	Direction of deformation	Limiting
Top of post	Horizontal Δx_1 or Δy	1/100 of height of post
Tip of cantilever	Horizontal Δx_2	1/100 of outreach plus height of post
Tip of cantilever	Vertical Δz	1/100 of outreach plus height of post

5.22 More stringent deflection limits shall be applied in accordance with the performance requirements of the equipment to be mounted.

5.23 Deformation at the extremities of the structural support shall be derived from the sum of the components of the effects of the load in the support posts, cantilever and sign supports (see Figure 3.2a and Figure 3.2b).

Environmental barriers and screens

5.24 Partial factors and criteria for serviceability and ultimate limit states shall be in accordance with:

1) BS EN 14388 [Ref 32.N]; and,

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2) specific requirements agreed with the Overseeing Organisation.

Road traffic sign posts

5.25 Partial safety factors and criteria for serviceability and ultimate limit states given in BS EN 12899-1 [Ref 14.N] shall be used for road traffic sign posts.

Minimum thickness of steel sections for cantilever masts

5.26 The minimum thickness of structural steel sections used in cantilever masts shall be as given in Table 5.26.

Table 5.26 Minimum thickness of steels sections for cantilever masts

Structural steel sections used for cantilever masts	Minimum section thickness
Plates and sections other than hollow sections	6 mm
Hollow sections effectively sealed by welding, having other than a small drain hole of diameter 10 mm to 15 mm	5 mm

5.27 Where other structural materials such as aluminium are used, the minimum section thicknesses shall be agreed with the Overseeing Organisation.

Closed hollow sections for cantilever masts

5.28 Steel hollow sections used in cantilever masts shall be designed to resist the ingress and retention of water or moisture by gravity flow, capillary action or condensation.

5.29 The plates used to close the open ends of hollow sections shall be of thickness not less than the lesser of the following:

1) the thickness of the walls of the hollow section; or,
2) 8 mm.

5.30 The end plates shall be joined by continuous structural quality welding to BS EN 1011-1 [Ref 35.N] and BS EN 1011-2 [Ref 36.N].

5.31 Closed hollow sections shall be detailed to prevent water from entering and subsequently freezing anywhere in the closed hollow steel section.

5.31.1 Water may be prevented from entering and freezing in closed hollow steel sections by providing suitably sized drain holes.

5.31.2 Where specified, drain holes should be no less than 10 mm nor greater than 15 mm in diameter.

Fatigue criteria for steel structures

5.32 Fatigue assessment shall be required for:

1) steel lighting columns 9 m and above in height; and,
2) all cantilever masts.

5.32.1 Fatigue rules may not be applicable for lighting columns and cantilever masts in very exposed sites.

NOTE Fatigue rules are set out in Section 11.

5.33 CCTV masts classified as category 1 in accordance with CG 300 [Ref 34.N] that are sited in very exposed locations shall be checked for fatigue.

NOTE The stringent deflection requirements for the design of CCTV masts means that stress ranges induced by dynamic response to wind loading are likely to be low. Thus fatigue is unlikely to be a critical design condition provided suitable details are used.

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5.34 In all cases, the procedures to be used for fatigue assessment shall be agreed with the Overseeing Organisation.

5.35 Fatigue susceptible details shall be avoided throughout the structure.

NOTE Fatigue damage is most likely to occur at or adjacent to welds or near sharp corners creating stress concentrations. Particularly vulnerable positions are:

1) *flange plates:*
   a) *at the weld throat between the column and flange; and,*
   b) *in the parent metal adjacent to the weld;*
2) *door openings:*
   a) *at welded attachments; and,*
   b) *at poorly finished cut edges;*
3) *at any stiffening between the column and the flange, and,*
4) *shoulder joints:*
   a) *at the weld throat; and,*
   b) *in the parent metal adjacent to the weld.*

5.36 Fatigue is critically dependent on geometrical configurations and fabrication, and stiffened and unstiffened door openings shall be in accordance with the constraints shown in Figure 5.36.

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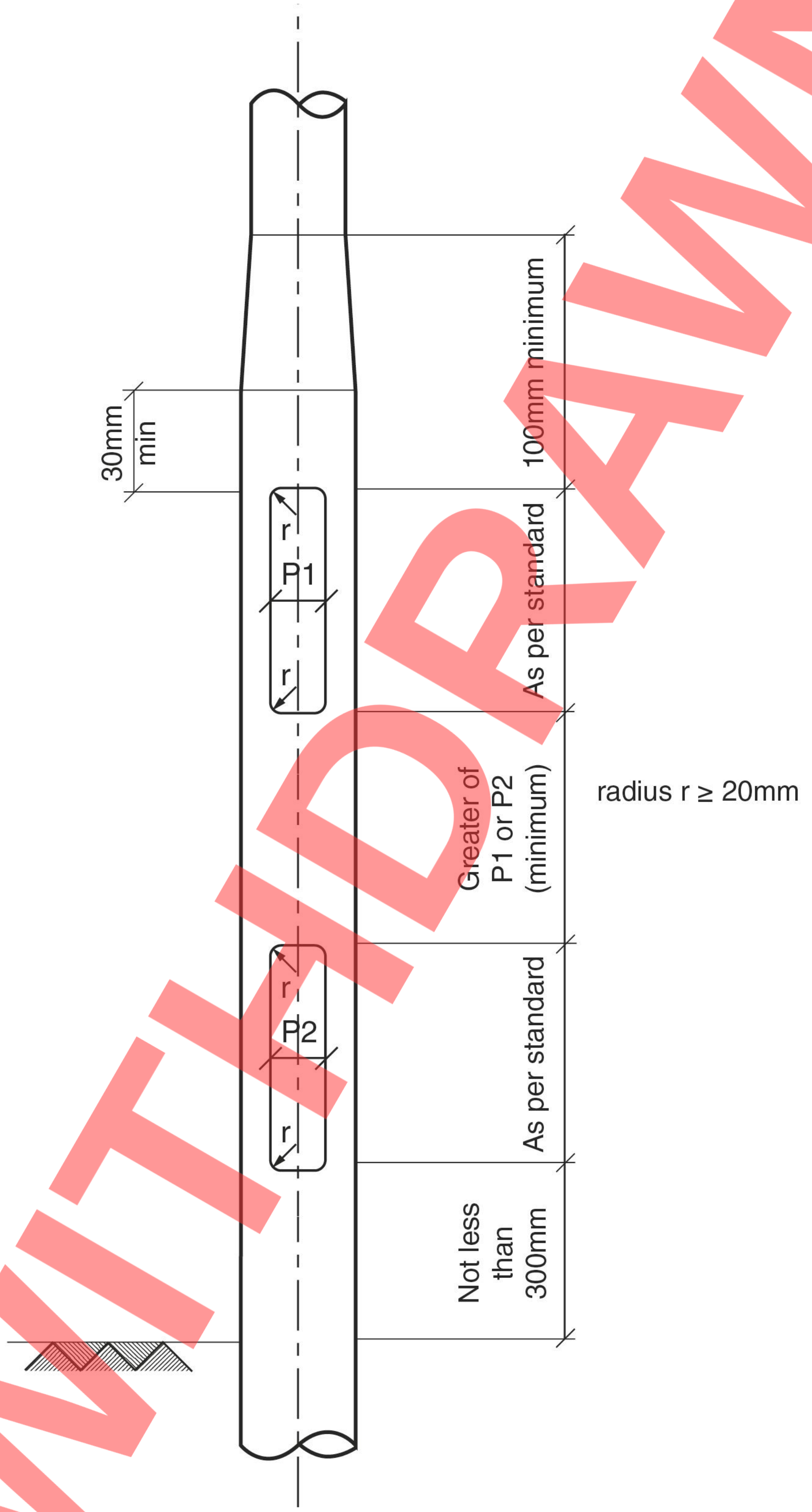
Figure 5.36 Door openings



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5.37 In addition, the following fabrication constraints shall be met:

1) sharp irregularities at free edges due to the flame cutting process are to be ground out;

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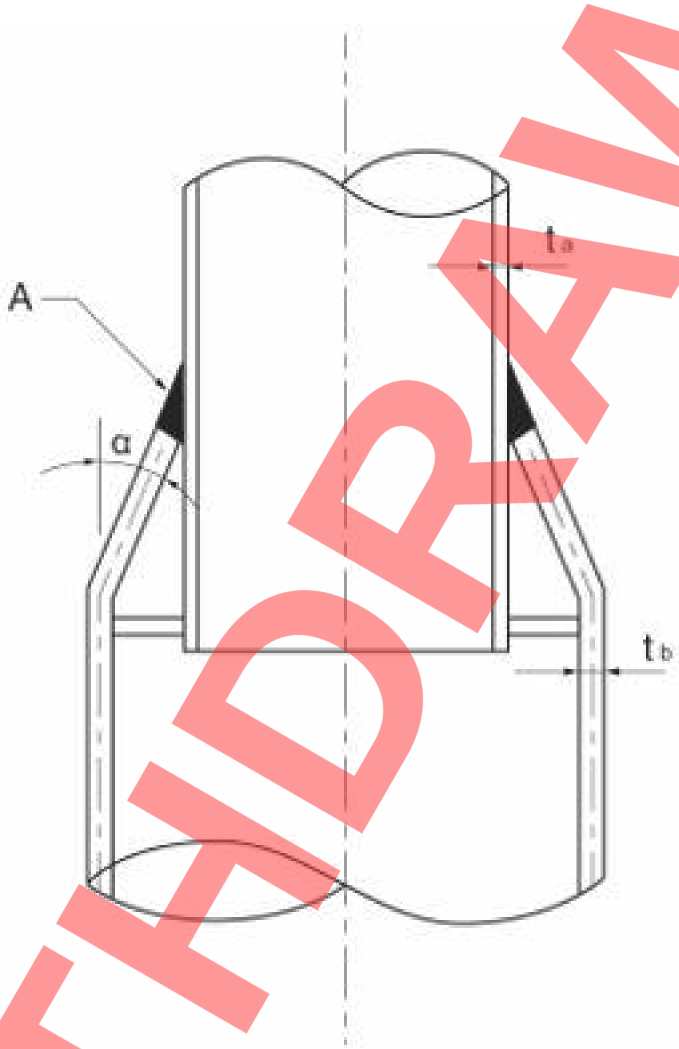
2) no welding closer than 10 mm from the edge of the unstiffened door opening;
3) longitudinal edge stiffeners to be continuous over their full extent; and,
4) where shoulder joints are used, their angle of inclination to the axis of the column has to be between 12° and 35° (see Figure 5.37 which shows a typical shoulder joint).

Figure 5.37 Typical shoulder joint



5.38 When undertaking fatigue checks, nominal stresses shall be based on nominal section properties.

5.39 The stress concentrations inherent in the make-up of a welded joint (arising, for example, from the general joint geometry and the weld shape) shall generally be included in the classification of the details.

5.39.1 Where the stress concentrations inherent in the make-up of a welded joint are not included in the weld classification, the nominal stress should be multiplied by stress concentration factors derived from stress analysis of the joint or from published data.

5.40 To undertake a fatigue check, a loading spectra shall be determined from wind data appropriate to the site.

5.40.1 In the absence of wind data, the fatigue loading provisions given in Section 11 may be adopted.

5.41 Classification shall be derived by fatigue testing of a sample of typical full-scale details in an independent testing laboratory, covering an appropriate stress range to enable a fatigue life curve to be derived.

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5.42 Tests shall be undertaken to provide a design curve representing mean minus two standard deviations.

5.43 In the absence of data on fatigue life curves and loading spectra, the procedure set out in Section 11 shall be followed.

Determination of shape coefficients

5.44 Where wind tunnel tests are necessary for the determination of shape coefficients for columns, brackets and lanterns, the testing shall be carried out in accordance with Appendix B of this document.

Fatigue criteria for materials other than steel structures

5.45 Structures in materials other than steel are not covered by the fatigue rules in this document and in such cases, the design shall be subject to technical approval in accordance to CG 300 [Ref 34.N].

5.46 Fatigue susceptible details shall be avoided throughout the structure.

5.46.1 The criteria given for steel structures may be used as the reference for avoiding fatigue susceptible details in structures made from other materials.

NOTE The fatigue requirements given in BS EN 1999-1-3 [Ref 10.N] can be used for aluminium structures.

5.47 The procedures to be used for fatigue assessment shall be agreed with the Overseeing Organisation.

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6. Fibre-reinforced polymer composite lighting columns

Design

6.1 Design loads and moments shall be determined in accordance with BS EN 40-3-1 [Ref 19.N] and BS EN 40-7 [Ref 22.N] as implemented by this document.

6.2 The factor β for the dynamic behaviour of the FRPC column shall be determined by reference to BS EN 40-7 [Ref 22.N] Annex B: Figure B.1.

Verification of structural design

General

6.3 The structural design of FRPC columns shall be verified either by calculations or by testing.

6.4 Test results shall take precedence in all cases.

Calculations

6.5 Design calculations for FRPC columns shall be in accordance with the requirements in BS EN 40-7 [Ref 22.N].

6.6 Flexural strength and the moduli in both longitudinal and transverse directions together with the shear modulus and the Poisson's ratio shall be determined by testing.

6.6.1 The mechanical properties of the FRP material to be used in the structural design calculations should be determined from tests using flat sheet samples manufactured in the same manner as that proposed for the production column.

6.7 A statistical assessment shall be made of the results to determine 95% confidence limits of the values to be used.

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7. Door openings

7.1 The sizes of door openings shall be as given in Table 7.1.

Table 7.1 Door opening sizes

Nominal column height (h) in metres	Type of door	Door opening for metal columns (height x width) in mm	Door openings for concrete columns (height x width) in mm
5 and 6	single door	500 x 100	680 x 95
8, 10 and 12	single door	600 x 115	680 x 130
8, 10 and 12	extended single door	-	900 x 130
8, 10 and 12	double doors	500 x 120 or 600 x 115 each	-

7.1.1 Alternative door openings selected from the sizes given in BS EN 40-2 [Ref 21.N] may be provided where they are suitable for the size of equipment to be housed and maintained in the column.

7.2 Doors shall be hinged or held captive by a chain or strap where:

1) columns are mounted on structures; or,
2) there is a risk of a detached door falling on the area below.

NOTE A falling detached door can cause an accident and injure workers or road users.

7.3 Where a metal chain or strap is used, it shall be capable of supporting the door in severe gales.

7.4 Doors shall include features to reduce vandalism and prevent access by unauthorised persons.

7.5 Where the section containing the door opening is steel or aluminium and circular or polygonal with eight or more sides, design strengths shall be calculated in accordance with BS EN 40-3-3 [Ref 20.N].

7.6 In all other cases, the design strength shall be calculated from first principles or based on the results of full-scale load tests with the procedures to be used agreed with the Overseeing Organisation.

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8. Wall mounted brackets

8.1 Wall mounted brackets shall be designed to suit the column brackets to be attached to them.

8.2 The mounted bracket shall be fixed to its support by means of flange plate and anchorage, designed in accordance with the provisions in Section 10.

8.3 The wall on which the wall mounted brackets are fixed shall be capable of carrying the additional loads and other forces transmitted by the bracket.

8.4 The design of the bracket shall provide the loads necessary for assessing the adequacy of the wall.

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9. Attachments

General requirements

9.1 Minor structures, other than CCTV masts, fixed traffic sign/signal posts and environmental barriers shall be designed for the attachment proposed.

NOTE Requirements for attachments to CCTV mast are provided separately in this Section.

9.2 The attachment shall be taken as a sign with the following characteristics:

1) rectangular in elevation, with a surface area of 0.3 m^2;
2) eccentricity from the centre line of the column to the centre of area of the sign of 300 mm;
3) the height above ground level at the column to the centre of area of the sign of 2500 mm; and,
4) orientation of the sign selected to produce the most adverse effects for the design condition being proposed.

9.3 Attachments shall not be allowed on cantilever masts.

9.4 The forces due to the dead and wind loads on the sign and bracket projecting from the column shall be determined in accordance with BS EN 40-3-1 [Ref 19.N].

9.5 The shape coefficient of the sign shall be taken as 1.8 unless derived from BS EN 1991-1-4 [Ref 3.N] for the specific shape and aspect ratio of the sign.

9.6 Where larger signs, waste paper containers, flower baskets and similar, are to be attached, the column shall be designed to resist the additional loading.

9.7 Additional loading shall be calculated in accordance with BS EN 1991-1-4 [Ref 3.N] and BS EN 40-3-1 [Ref 19.N].

9.8 Minor structures designed to carry attachments greater than those defined in clause 9.2 shall have identifying manufacturer's features or marks to enable them to be clearly and unambiguously identified throughout their service life.

9.9 The unique identifying mark shall be listed as required by CG 302 [Ref 1.N].

Attachments to CCTV masts

9.10 CCTV masts shall not be designed for attachments other than CCTV cameras and their associated equipment.

9.11 Where attachments are specified, they shall be incorporated into the design of the CCTV masts.

9.12 Where attachments are to be used, the mast shall be designed to resist the additional loading.

9.13 The additional dead and wind loads as a result of the attachments shall be calculated in accordance with PLG 07 [Ref 28.N].

9.14 Where attachments are required, the CCTV pole and the attachments shall be designed such that the operation of the CCTV camera is not impeded.

9.15 Access for installation, inspection or maintenance of an attachment shall not interfere with the operation of the CCTV camera.

9.16 Where attachments are located below the operating position of the camera, they shall be designed as demountable to allow the CCTV mounting to be raised and lowered.

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10. Design of flange plates

General

10.1 The procedures in this Section shall be applied for the design of the connection between the structure and its support structure.

NOTE A support structure can include a bridge deck, wall, reinforced concrete foundation block or pile cap etc.

Attachment system and anchorage

10.2 A structure with a flange plate shall be fixed to the support structure by an attachment system and anchorage.

NOTE The attachment system usually takes the form of holding-down bolts that connect with an anchorage.

10.3 The attachment system and anchorage shall provide structural restraint without causing damage to the support structure.

10.4 Anchorages of expanding type shall not be used unless their long-term in-situ performance under fatigue loading can be demonstrated to the satisfaction of the Technical Approval Authority.

10.5 The attachment system shall allow the structure to be demounted.

10.6 The attachment system shall be such that damaged minor structures are readily removed and replaced.

10.7 The attachment system and anchorages for cantilever masts shall be designed for damaged cantilever masts to be readily removed and replaced where they are located:

1) within 4.5 metres of the 'point from which set-back is measured ', in accordance with CD 377 [Ref 29.N]; or,
2) within the central reserve.

10.8 To make the damaged cantilever mast readily removable and replaceable, an internally threaded component shall be provided in the anchorage to receive the holding down bolts.

Typical arrangement of flange plates

10.9 The procedures given shall apply to square flange plates:

1) with bolt holes for four symmetrically disposed holding down bolts; and,
2) connected to circular or octagonal columns.

10.10 For flange plates not complying with clause 10.9, other suitable design methods, or full scale load methods shall be adopted, as agreed with the Overseeing Organisation.

10.11 Typical arrangement of flange plate shall be as shown in Figure 10.11a and Figure 10.11b below where:

1) details A and B are typical only and can be used with circular or octagonal columns;
2) D = 2R and R = mean radius as defined in Figure 3 of BS EN 40-3-3 [Ref 20.N]; and,
3) '*' is the radius of centrally located hole not exceeding 0.3D .

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Figure 10.11a Typical arrangement of flange plate

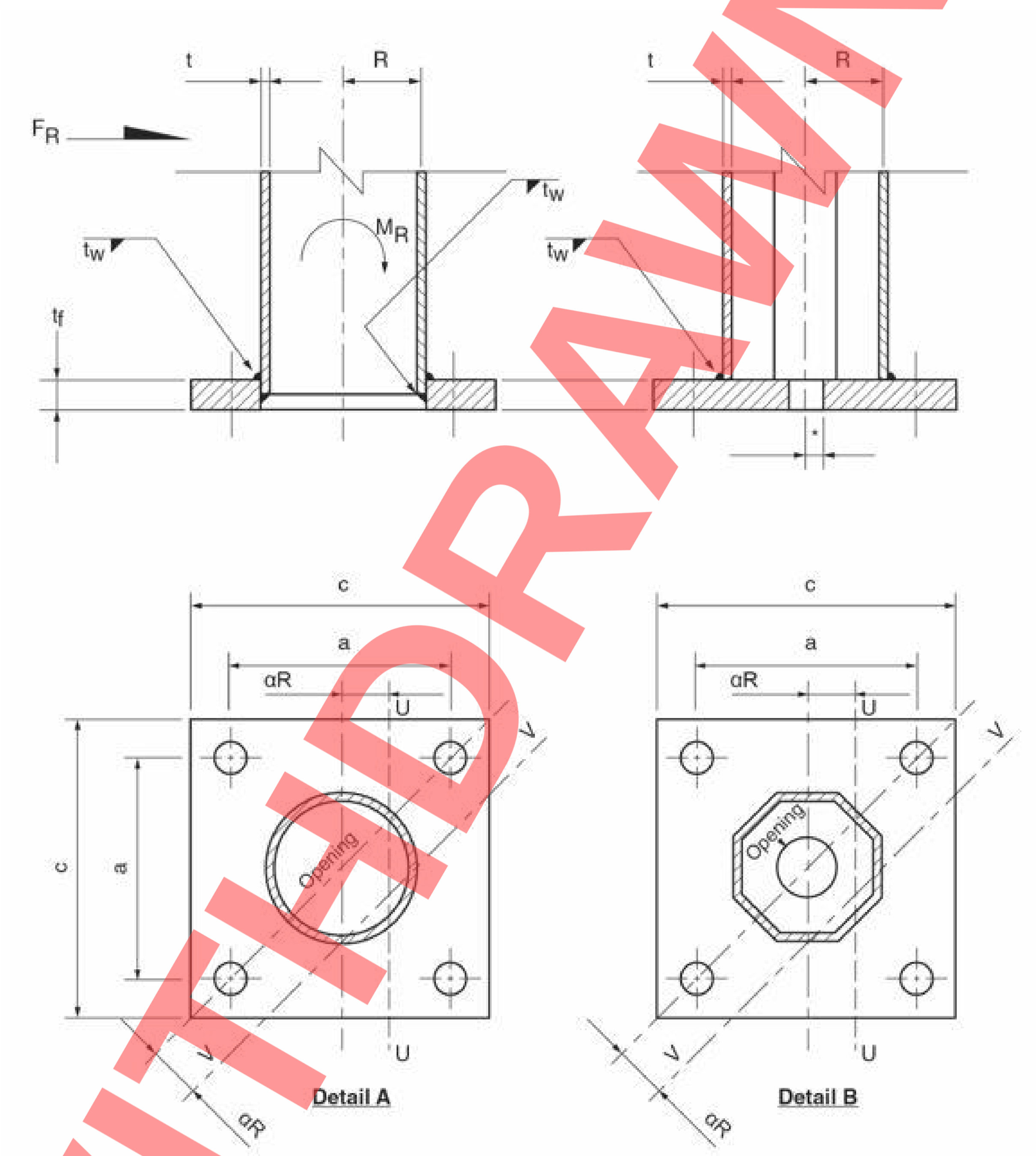

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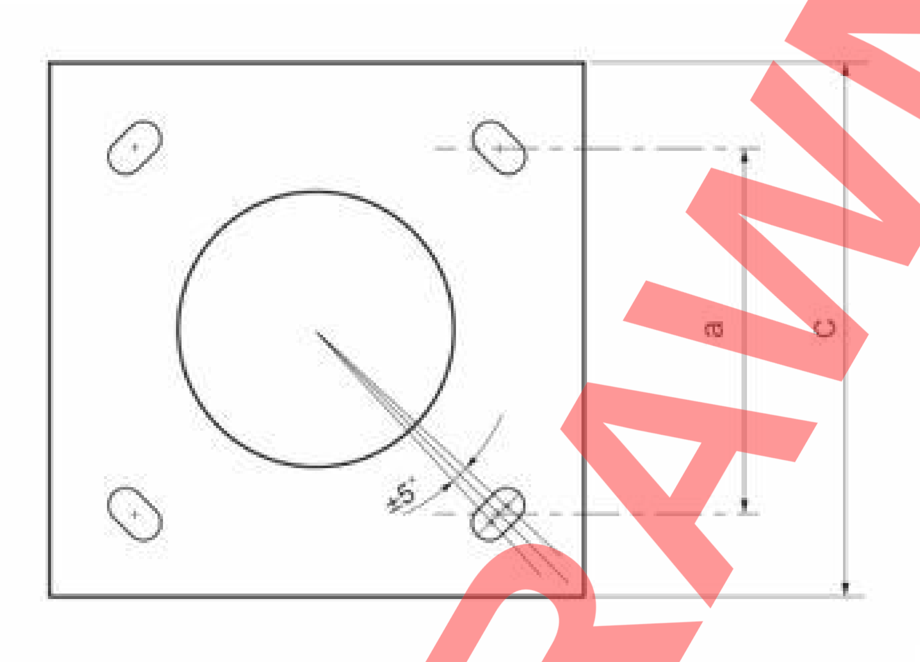
Figure 10.11b Slotted hole arrangement



NOTE Figure 10.11b shows the typical arrangement where slotted holes are used on a flange plate.

10.12 The typical arrangements shown in Figure 10.11a shall apply to both plates supported on bedding material and plates supported on levelling nuts only, without effective bedding.

10.13 When the weight of the structures is to be carried by nuts beneath the flange plate, the holding down bolts shall be designed to resist all additional stresses arising from this construction detail, and protected against corrosion.

10.14 When the weight of the structure is supported directly through the flange plate to the support structure, the space shall be packed with a suitable bedding mortar.

Minimum edge distances

10.15 The minimum distance from the centre of the bolt hole to the edge of the plate shall not be less than 1.5d, where d is the diameter of the bolt hole.

10.16 Where slotted holes are used the minimum distance from the axis of the slotted hole to the adjacent edge of the plate shall not be less than 1.5d.

10.17 Where slotted holes are used the minimum distance from the centre of the end radius of a slotted hole to the adjacent edge of the plate shall not be less than 1.5d.

10.18 The diameter of circular flange plates shall not be less than the pitch circle diameter of the holding down bolts plus 2.5 times the diameter of the bolt holes.

Structures not subject to vehicle impact

10.19 Minor structures shall not be designed for vehicle impact where they have a vehicle restraint system designed in accordance with CD 377 [Ref 29.N].

10.20 Where the minor structure is not designed for vehicle impact, the flange plate and connection shall be designed for dead loads and wind loads only.

10.21 The bending moment (M_R) in the formulae shall be taken as the bending moment at the base of the structure derived from the ultimate factored dead loads (permanent actions) and wind loads (variable actions).

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Structures subject to vehicle impact

10.22 The flange plate and its connections shall be designed to resist vehicle impact.

10.23 The bending moment, (M_R) , in the formulae shall be the bending moment at the base of the structure derived from the ultimate factored horizontal impact load, F_R , acting at a point of impact at 0.50 m above the top of the foundation.

Simplified design procedure

Column/flange plate connection

10.24 This simplified procedure for column/flange plate connection shall apply only in the design of flange plates complying with:

1) bolt holes for four symmetrically disposed holding down bolts; and,
2) connected to circular or octagonal columns; and,
3) having a centrally located hole not exceeding 0.3D

NOTE D is the column base diameter taken as 2R where R is the mean radius as defined in Figure 3 of BS EN 40-3-3 [Ref 20.N].

10.25 The simplified procedure for column/flange plate connection assumes that the bending moment shall be critical about the v-v axis as shown on Figure 10.11a Detail B.

Moment of resistance

10.26 The flange plate shall be capable of developing a moment of resistance about each axis, taken at the underside of the flange plate, at least 1.2 times the theoretical ultimate moment capacity, $M_R (= M_{UP})$ of the actual structure calculated at base level in accordance with clause 5.6.2 in BS EN 40-3-3 [Ref 20.N].

10.27 The maximum bending moment in the flange plate shall thus be given by Equation 10.27:

Equation 10.27 Maximum bending moment in the flange plate

$$M = 1.2\, M_R \left\{ 0.5 - \frac{0.63 D\sqrt{2}}{4a} \right\}$$

where:

a is the bolt spacing as shown in Figure 10.11a (mm);

D is the column base diameter (mm);

M is the maximum bending moment in flange plate (N.m); and,

M_R is the theoretical bending moment capacity (N.m).

10.28 The maximum bending in the flange plate, M , shall not exceed the plastic moment capacity M_p , given by Equation 10.28:

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Equation 10.28 Plastic moment capacity of the flange plate

$$M_P = \frac{(c\sqrt{2} - 0.63D) \times t_f^2}{4} \times \frac{f_y}{\gamma_m \times 10^3}$$

where:

M_p	is the plastic moment capacity of flange plate (N.m);
γ_m	= 1.15;
c	is the width of the flange plate (mm);
t_f	is the thickness of the flange plate (mm);
f_y	is the yield stress in the flange plate (N/mm^2); and,
D	is the column base diameter (mm).

Shear and bearing

10.29 Shear and bearing shall not govern the design provided the minimum edge distances for circular or slotted bolt holes whichever type is used are satisfied.

Design of welds

10.30 The welded connection between the column and the flange plate shall be capable of developing the theoretical ultimate moment of resistance of the actual column and the equivalent ultimate shear force, both as derived in this section.

10.31 Welds shall be designed such that the throat thickness of the top weld is not less than k $\times$ t where:

1) k is a value between 1.0 and 1.5 depending on the type of weld used; and,
2) t is the wall thickness of the column at the flange plate.

10.32 For example the value of, k , shall be taken as:

1) k = 1.5 for the fillet welds in Figure 10.11a Detail B, and for the outer fillet weld in Figure 10.11a Detail A;
2) k = 1.0 for a full penetration butt weld.

NOTE A more accurate procedure for the design of welds is given in the detailed design procedure provided later in this section.

Design of holding-down bolts

10.33 The holding-down bolts shall be capable of developing the theoretical ultimate moment capacity of the actual column, $M_R (= M_{UP})$, calculated at the base level in accordance with clause 5.6.2 of BS EN 40-3-3 [Ref 20.N] and an equivalent ultimate shear force, $F_R (= 2M_R)$.

10.34 The tensile stress (σ) in holding down bolts shall be given by Equation 10.34:

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Equation 10.34 Tensile stress in holding down bolts

$$\sigma = \frac{1.2\ \mathrm{M_R} \times 10^3}{\sqrt{2} \times \mathrm{a} \times \mathrm{A_{et}}}$$

where:

σ	is tensile stress (N/mm²);
A_{et}	is the tensile stress areas in the thread of the bolt, obtained from the appropriate standard (mm²);
a	is the bolt spacing, as shown in Figure 10.11a (mm); and,
M_R	is the theoretical ultimate moment capacity (N.m).

10.35 The shear stress (τ) in the bolts shall be as given in Equation 10.35:

Equation 10.35 Shear stress in bolts

$$\tau = \frac{1.2\ \mathrm{F_R}}{\mathrm{n_b} \times \mathrm{A_{eq}}}$$

where:

τ	is the shear stress (N/mm²);
n_b	is the total number of bolts fixing the flange plate;
A_{eq}	is the sectional area of the unthreaded shank of the bolt if the shear plane passes through the unthreaded part but taken as A_{et} if the shear plane passes through the threaded part (mm²);
A_{eq}	is the tensile stress areas in the thread of the bolt, obtained from the appropriate standard (mm²); and,
F_R	is the shear force (N).

10.36 Where slotted holes are used, n_b shall not include bolts in holes where the slot aligns with the direction of the applied shear force.

10.37 Bolts in tension and shear shall satisfy the expression:

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Equation 10.37 Combined tension and shear expression for bolts

$$\left\{\left(\frac{\sigma}{f_t}\right)^2 + 2\left(\frac{\tau}{f_q}\right)^2\right\}^{\frac{1}{2}} \leq \frac{1}{\gamma_m}$$

where:

γ_m is taken as 1.3;

f_q is the yield stress of bolts (factored by 0.85 in the case of black bolts (N/mm^2) ;

f_t is the lesser of:

a) 0.7 x minimum ultimate tensile stress (N/mm^2); or,

b) either yield stress or the stress at permanent set of 0.2%, whichever is appropriate (N/mm^2);

τ is the shear stress in bolts (N/mm^2); and,

σ is the tensile stress (N/mm^2).

10.38 The capacity of the complete anchorage to resist the forces involved ($1.5M_R$ and $1.5F_R$) shall also be verified, with regard to embedment and pull out based on a 90° cone recommended in CS CA42 [Ref 3.I].

Bearing stresses under flange plates

10.39 The bearing stress on the foundation medium is derived on a basis compatible with the assumed bending mode v-v, on either a plastic or elastic basis as required and on a plastic basis, the maximum bearing stress for bending, σ_{bs} , about v-v shall be given by Equation 10.39:

Equation 10.39 Maximum bearing stress about v-v

$$\sigma_{bs} = \frac{3\ M_R \times 10^3}{0.7 \times (0.7c - R)^2 \times (a + 0.5c + 0.7R)}$$

where:

σ_{bs} is the maximum bearing stress (N/mm^2);

M_R is the theoretical ultimate moment capacity (N.m);

R is the mean radius of the section (mm);

c is the width of the flange plate (mm); and,

a is the bolt spacing, as shown in Figure 10.11a (mm).

10.40 The bearing stresses in any bedding mortar under the flange plates shall not exceed 20 N/mm^2.

10.41 The maximum bearing stresses on the concrete under a flange plate shall be in accordance with the requirements of BS EN 1992-1 [Ref 12.N].

10.42 The requirements for foundations on masonry shall be agreed with the Overseeing Organisation.

10.43 Bases founded on steel bridge decks shall require a more thorough analysis, which is outside the scope of this document.

Design of anchorages to bolts

10.44 The design approach for the anchorage shall be dependent on the medium in which the anchorage is made.

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10.45 The anchorages shall be designed to cater for a maximum tensile force, T_A , and associated shear, F_A , as given in Equations 10.45a and 10.45b:

Equation 10.45a Maximum tensile force in anchorage

$$T_A = 1.25\sigma A_{et}$$

Equation 10.45b Associated shear force in anchorage

$$F_A = 1.25\tau A_{eq}$$

where:

σ	is the tensile stress (N/mm²);
τ	is the shear stress in bolts (N/mm²);
A_{et}	is the tensile stress areas in the thread of the bolt, obtained from the appropriate standard (N/mm²);
A_{eq}	is the sectional area of the unthreaded shank of the bolt if the shear plane passes through the unthreaded part but taken as A_{et} if the shear plane passes through the threaded part (N/mm²);
T_A	is the tensile force in anchorage (N); and,
F_A	is the shear force in anchorage (N).

NOTE The design of anchorages to bolts is dependent on the medium in which the anchorages are made.

10.46 The capacity of the anchorage shall be derived in accordance with section 12 and BS EN 1997 [Ref 8.N].

10.47 The supporting structure shall be designed to resist the anchorage loads without damage.

10.47.1 The tensile strength of the concrete should be ignored in the calculations for the anchorage capacity.

10.48 The concrete in the foundation or bridge component to which a column is fixed shall be reinforced against bursting associated with the above internal forces generated by the holding down bolts/anchorage system.

Use of levelling nuts and slotted holes

10.49 Where levelling nuts (or other system of permanent packers) are being used without effective bedding, it shall be assumed that all the bearing stresses are transferred to the levelling nuts.

10.50 The nuts and washers on both sides of the flange plate shall be sized to prevent any localised plate failure due to concentration stresses.

10.50.1 Localised plate failure may be prevented by using washers complying with BS EN ISO 7093 [Ref 5.I], provided the hole or width of the slotted hole does not exceed d_0 + 4 mm where d_0 is the diameter of the holding down bolts.

10.51 For slotted holes which provide flange plate rotations of up to ± 5° as shown in Figure 10.11b, washers of suitable thickness shall be provided on both sides of the flange plate to transfer load into the holding down bolts.

10.51.1 Washers complying with BS EN ISO 7093 [Ref 5.I] may be used provided the width of the slotted holes does not exceed d_0 + 6 mm.

10.52 Special washer plates shall be used where the hole or slotted clearances exceed d_0 + 6 mm.

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10.53 Where levelling nuts are used, the nut and washer size shall be the same above and below the flange plate.

Detailed design procedure

Column/flange plate connection

10.54 The flange plate shall have a centrally located hole either:

1) of diameter equal to that of the column (see Figure 10.11a Detail A); or,
2) not exceeding 0.3D in diameter (see Figure 10.11a Detail B).

NOTE 1 This detailed procedure applies only in the design of flange plates with bolt holes for four symmetrically disposed holding down bolts and connected to circular or octagonal columns.

NOTE 2 D is the column base diameter taken as 2R .

10.55 The detailed design procedure shall be applied in the design of square flange plates connected to circular or octagonal columns as shown in Figure 10.11a.

10.56 The detailed design criteria provided for the welds, flange plate, holding down bolts and bearing stress shall apply.

10.57 In the detailed design procedure the bending moment shall be checked about the v-v axis and u-u axis as shown on Figure 10.11a.

10.58 For flange plates not complying with the constraints given in this document for the column/flange plate connection, other suitable design methods, or full scale load tests shall be adopted.

10.59 Alternative design methods, or full scale load tests adopted shall approved by the Overseeing Organisation.

Detailed design of welds

10.60 The welded connection between the column and flange plate shall be capable of developing the ultimate moment of resistance, M_R , as derived from BS EN 40-3-3 [Ref 20.N] and the equivalent shear force, F_R .

10.60.1 The connection may be achieved by fillet welds of leg length, t_w , as shown in Figure 10.11a, detail A or detail B.

10.60.2 In the case of Figure 10.11a detail B in particular, the length of fillet weld, t_w , required may need to be considerably in excess of the wall thickness, t , in order to satisfy these requirements.

10.60.3 As an alternative to fillet welds, full penetration butt weld may be used which automatically satisfies these requirements.

Derivation of weld stresses

10.61 The stress in the fillet welds due to moment of resistance, M_R , shall be taken as:

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Equation 10.61 Stress in the fillet welds due to moment of resistance

$$\tau_1 = \frac{M_R \times 10^3}{\pi R^2(0.7t_w)}$$

where:

τ_1	stress in the fillet weld due to moment (N/mm²);
t_w	leg length of the fillet weld (mm);
R	is the mean radius of the cross section (mm); and,
M_R	is the theoretical ultimate moment capacity (N.m)

10.62 The stress in the fillet welds due to the equivalent shear force, F_R , shall be taken as:

Equation 10.62 Stress in fillet welds due to equivalent shear force

$$\tau_2 = \frac{F_R}{2\pi R(0.7t_w)} = \frac{M_R}{\pi R(0.7t_w)}$$

where:

τ_2	stress in fillet weld due to the equivalent shear force (N/mm²);
F_R	equivalent shear force (N);
t_w	is the fillet weld leg length (mm);
M_R	is the theoretical ultimate moment capacity (N.m); and,
R	is the mean radius of the cross section (mm).

10.63 The resultant weld stress, τ_R , shall be calculated as:

Equation 10.63 Resultant shear stress in fillet welds

$$\tau_R = \left(\tau_1^2 + \tau_2^2\right)^{\frac{1}{2}} = \frac{M_R}{\pi R(0.7t_w)}\sqrt{\left(\frac{1000}{R}\right)^2 + 1}$$

where:

τ_1 , τ_2 , and τ_R	are shear stresses (N/mm²);
τ_R	resultant shear stress in fillet welds (N/mm²);
R	is the mean radius of the cross section (mm);
t_w	is the fillet weld leg length (mm); and,
M_R	is the theoretical ultimate moment capacity (N.m).

Capacity of welds

10.64 The resultant stress in the fillet welds, τ_R , shall not exceed the weld capacity, τ_D , (i.e. $\tau_R < \tau_D$) . The weld capacity, τ_D , is given by:

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Equation 10.64 Weld capacity

$$\tau_D = \frac{k(f_y + 455)}{2\gamma_m\sqrt{3}}$$

where:

f_y	is the yield stress of the column section (f_s) or the flange plate (f_f), whichever is the lesser (N/mm^2);
γ_m	is taken as 1.20; and,
k	= 0.9 for side fillets where the weld is subject to longitudinal shear, or 1.4 for end fillets in end connections where the weld is subject to transverse shear, or 1.0 for all other welds.

Design for bending in flange plates

10.65 The flange plate shall be designed to resist at least the effect of, $1.2M_R$, at the base of the column where, M_R , is as derived from BS EN 40-3-3 [Ref 20.N].

10.66 The flange plate shall be checked about bending parallel to one side (axis u-u) and on the diagonal (axis v-v), see Figure 10.11a Detail A and Detail B.

Derivation of bending moments in flange plates

10.67 The maximum bending moment on the flange plate axes u-u and v-v for plates with effective bedding or supported on levelling nuts only shall be calculated using the formulae shown below in Equations 10.67a and 10.67b, respectively:

Equation 10.67a Maximum bending moment about u-u axis

$$M_{u\text{-}u} = 0.6M_R\left[1 - \alpha\frac{2R}{a}\right]$$

Equation 10.67b Maximum bending moment about u-u axis

$$M_{v\text{-}v} = 0.6M_R\left[1 - \alpha\frac{2R}{a\sqrt{2}}\right]$$

where:

$M_{u\text{-}u}$ and $M_{v\text{-}v}$	are bending moment about the axis stated (N.m);
R	mean radius of the column cross section (mm);
a	spacing of the bolts (mm); and,
α	relates to the position considered for maximum bending in the plate. In lieu of more thorough analysis, α , can be based on the centroid of the welds on the tensile side, i.e. α can be taken as 0.63.

Bending capacity of flange plate

10.68 The maximum moment in the flange plate, M , shall not exceed the plastic moment capacity of the flange plate, M_p .

10.69 Where the square flange plate has a centrally located hole not exceeding 0.3D , the plastic moment of the flange plate, M_p , shall be calculated as given in Equation 10.28.

10.70 For a square flange plate where the centrally located hole is the same diameter as the column base (refer to Figure 10.11a Detail A), M_p , shall be given by:

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Equation 10.70a Plastic moment for u-u axis

$$M_p = \left(c - 2R\sqrt{1-\alpha^2}\right)\frac{t_f^2}{4}\frac{f_f}{\gamma_m 10^3}$$

Equation 10.70b Plastic moment for v-v axis

$$M_p = \left(c\sqrt{2} - 2R\left(\alpha + \sqrt{1-\alpha^2}\right)\right)\frac{t_f^2}{4}\frac{f_f}{\gamma_m 10^3}$$

where:

M_p	plastic moment for u-u axis and v-v axis (N.m);
γ_m	is taken as 1.15;
c	is the width of the flange plate (mm);
t_f	is the thickness of the flange plate (mm); and,
f_f	is the yield stress of the flange plate (N/mm^2).
R	is the mean radius of the column cross section (mm)

Design of holding down bolts

Derivation of stresses in bolts

10.71 The tensile stress in the holding down bolts shall be taken as:

Equation 10.71 Tensile stress in holding down bolts

$$\sigma = \frac{1.2M_R \times 10^3}{n_t a A_{et}}$$

where:

A_{et}	is the tensile stress area in the thread of the bolt obtained from the appropriate standard (mm^2);
a	is the bolt spacing (mm);
M_R	is the ultimate moment of resistance (N.m);
n_b	is the total number of bolts fixing the flange plate;
n_t	is related to the number of bolts resisting tension and the assumed axis of bending and can be taken as: 1) $0.5n_b$ for bending about axis u-u; see Figure 10.11a; or, 2) $0.25n_b$ for bending about axis v-v; see Figure 10.11a;
σ	is the tensile stress (N/mm^2).

10.72 In general, $(n_t \times a)$, shall not be taken as greater than, $(a + \alpha R + 0.5c)$, for axis u-u, nor greater than, $0.7(a + 0.7\alpha R + 0.5c)$, for axis v-v to ensure compatibility with the assumed mode of bending.

10.73 The shear stress in the bolts shall be taken to be that derived using Equation 10.35.

10.74 The combined tension and shear shall be taken to be that derived using Equation 10.37.

10.75 The capacity of the anchorage shall be taken as that derived in accordance with clause 10.38.

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Check on bearing stress below the flange plate

10.76 The bending stress on the foundation medium shall be derived for both the assumed bending modes about u-u axis and v-v axis, on either a plastic or elastic basis.

10.77 The maximum calculated bearing stress shall not exceed the value determined in accordance with Equation 10.39.

10.78 On a plastic basis, the maximum bearing stress for bending about u-u shall be taken as:

Equation 10.78a Maximum bearing stress for bending about u-u axis

$$\sigma_{\text{bs(u-u)}} = \frac{3\text{M}_\text{R} \times 10^3}{\text{c}(0.5\text{c} - \alpha\text{R})(0.75\text{a} + 0.5\alpha\text{R} + 0.5\text{c})}$$

Equation 10.78b Maximum bearing stress for bending about v-v axis

$$\sigma_{\text{bs(v-v)}} = \frac{3\text{M}_\text{R} \times 10^3}{0.7(0.7\text{c} - \alpha\text{R})^2(\text{a} + 0.7\alpha\text{R} + 0.5\text{c})}$$

where:

$\sigma_{\text{bs(u-u)}}$, $\sigma_{\text{bs(v-v)}}$	maximum bearing stress about u-u and v-v (N/mm^2);
M_R	is the ultimate moment of resistance (N.m);
R	is the mean radius of the column cross section (mm);
c	is the width of the flange plate (mm);
a	is the bolt spacing (mm); and,
α	relates to the position considered for maximum bending in the plate. In lieu of more thorough analysis, α , can be based on the centroid of the welds on the tensile side, i.e. α can be taken as 0.63.

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11. Fatigue checks of steel structures

General

11.1 Fatigue checks shall be undertaken in accordance with the general criteria given in Section 5 of this document.

11.2 For reinforcement at door openings, the geometrical constraints set out in clauses 5.36, 5.37 and clauses 11.9 to 11.18 shall be met, and stress ranges around door openings need not be calculated.

11.3 Where the geometric constraints are not met, then the requirements of 11.4 or 11.5 shall be followed.

11.4 For minor structures other than cantilever masts for traffic signals and/or speed cameras that project over the carriageway, only fatigue due to gust wind loading shall be checked and the requirements of clauses 11.6 to 11.18 satisfied.

11.5 For cantilever masts for traffic signals and/or speed cameras that project over the carriageway, the fatigue effects from wind gust loading and high vehicle buffeting shall be combined and satisfy the requirements of clauses 11.19 to 11.31.

Fatigue due to gust wind loading

11.6 Checks on fatigue shall be undertaken at and adjacent to each welded section.

11.6.1 Fatigue checks should be carried out for the ends of reinforcement at door openings.

11.7 A check on fatigue shall be undertaken using a stress range given by:

Equation 11.7 Stress range

$$\sigma_{\mathrm{r}} = 0.25\sigma_{\mathrm{s}}\left(1 - \frac{1}{\beta}\right)\left(\frac{\mathrm{c_{vs}}}{\mathrm{c_{stat}}}\right)$$

where:

σ_{r} is the stress range (N/mm^2);

σ_{s} is the stress calculated at this position for the design forces and moments specified in section 4 of BS EN 40-3-1 [Ref 19.N] (N/mm^2);

β is the dynamic response factor (Clause 5.2.4 of BS EN 40-3-1 [Ref 19.N];

$\mathrm{c_{stat}}$ is the average shape coefficient for the top half of the column as used for the static analysis and given in Figure 3 of BS EN 40-3-1 [Ref 19.N];

$\mathrm{c_{vs}}$ is 1.2 for circular sections;

is 1.3 for octagonal sections with r/D > 0.075 ;

is 1.45 for octagonal sections with r/D < 0.075 ;

r is the radius of the corner (mm); and,

D is the distance across the flats (mm).

11.8 This stress range, σ_{r} , shall be less than that obtained using the fatigue curves in Appendix A - Figure A.1 or Figure A.2, for the class of detail being considered and for a number of cycles, n_1 , given by:

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Equation 11.8 Number of oscillation cycles

$$n_1 = 10^6 N_f L$$

where:

N_f is the frequency of vibration of the column (Hz);

L is the design life of the structure (in years); and,

n_1 is number of cycles.

11.8.1 For a design life of L years, Appendix A - Figure A.1 may be used by adopting an effective frequency N_{fe} as the horizontal scale given by:

Equation 11.8.1 Effective frequency for a given design life

$$N_{fe} = N_f \times \frac{L}{25}$$

where:

N_{fe} is the effective frequency (Hz);

N_f is the frequency of vibration of the column (Hz); and,

L is the design life of the structure (in years).

NOTE The basis of the fatigue life curves Appendix A - Figure A.1 and Figure A.2.

Geometrical configurations and fabrication constraints

11.9 The geometric and fabrication constraints on cross sections of steel lighting columns shall be satisfied, in order to use the classes of typical weld details as provided in Appendix A - Table A.2 to Table A.9.

NOTE 1 Geometric and fabrication constraints are associated with flange plates, shoulder joints and door openings on the cross sections of lighting columns.

NOTE 2 Guidance for weld classification is provided in Appendix A.

Flange plates

11.10 The column/flange plate weld 1A, 2/1 and 2/2 shown in Appendix A - Table A.2, Table A.3 and Table A.4 shall have a throat size K times greater than the thickness of the adjacent shaft material, where the value of K is determined from Table 11.10.

Table 11.10 Values of K for various weld detail types

Weld	K
1A	1.10
2/1	1.25
2/2	1.25*
*or use full penetration butt weld	

11.11 The thickness of the material, t_b , shall be not less than the thickness of the adjacent shaft material, t_s (i.e $t_b > t_s$) .

Shoulder joints

11.12 Welded shoulder joints, as shown in Appendix A - Table A.5, Table A.6 and Table A.7 shall have an angle of inclination to the axis of the columns, α , between the following limits: $12^o < \alpha < 35^o$.

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11.13 The shoulder joint weld A, as shown in Appendix A - Table A.6 and Table A.7 shall have a throat size 10% greater than the thickness of the adjacent shaft material, t_s .

11.14 To ensure that weld detail type 6 shown in Appendix A - Table A.8 behaves as intended, the lapped length shall be at least 1.5 times the diameter of the lapped shaft.

Door openings

11.15 Stiffened and unstiffened door openings shall be in accordance with the constraints shown in Appendix A - Table A.9.

11.16 Sharp irregularities at free edges due to the flame cutting process shall be ground out.

11.17 No welding shall be closer than 10 mm from the edge of the door unstiffened opening.

11.18 Longitudinal edge stiffeners shall be continuous over each stiffener.

Fatigue from high vehicle buffeting

11.19 Checks on fatigue shall be undertaken at the following positions:

1) at and adjacent to each welded section; and,
2) the end of the reinforcement at door openings.

11.20 The stress range, σ_{r2i} , in any part of the structure for fatigue due to high vehicle buffeting shall be calculated by applying:

1) a pressure of P_d to the portion of the cantilever arm and any attachments above the carriageway vertically downwards; and,
2) a pressure of P_d to the portion of the cantilever arm and any attachments above the carriageway horizontally against the direction of the traffic.

11.21 The design pressure , P_d (in N/m^2) , shall be given by:

Equation 11.21 Design pressure

$$P_d = 600h^{-0.25} - 400$$

where h is either:

i) the distance from the top of the high-sided vehicle to the underside of any horizontal surface; or,

ii) the distance from the top of the high-sided vehicle to the centre of pressure of any vertical surface.

11.22 A typical high-sided vehicle height of 4.2 metres shall be used.

NOTE The formula for P_d applies for a value of up to $h = 5$ m.

11.22.1 Where a particular site has a higher average vehicle height then this should be used instead.

NOTE 1 A high-sided vehicle height of 4.2 metres has been adopted for calculating the pressure, P_d , as a representative height of such vehicles currently in use on UK highways.

NOTE 2 The design pressure, P_d , assumes that the maximum speed of the high-sided vehicle is limited to 60 mph. Where regulations permit higher maximum speeds then specialist advice has to be sought.

11.23 Applied loads shall be calculated as the product of the design pressure and projected area.

11.24 Partial load factor γ_{fl} shall be taken as 1.0.

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Fatigue damage assessment

Fatigue due to gust wind loading

11.25 For fatigue due to gust wind loading, the number of cycles, n_1 , shall be calculated from Equation 11.8.

11.26 The corresponding number of cycles to failure, N_1 , shall be given by:

Equation 11.26 Number of cycles to failure for fatigue due to gust wind loading

$$N_1 = 2 \times 10^6 \left(\frac{\sigma_o}{\sigma_r}\right)^m$$

where:

N_1	is the number of cycles to failure;
σ_o	is the details category (50, 120);
σ_r	is the stress range; and,
m	is the slope of curve (m is taken as 3 for Figure A.1 and 4 for Figure A.2 as given in Appendix A).

Fatigue due to high vehicle buffeting

11.27 For fatigue due to high vehicle buffeting, the number of cycles for each lane in a carriageway, n_{2i} , shall be given by:

Equation 11.27 Number of cycles for fatigue due to high vehicle buffeting

$$n_{2i} = 1.6 \times 10^7 (L \times F_i)$$

where:

n_{2i}	is the number of cycles for each lane in a carriageway;
L	is the design life of the structure (in years); and,
F_i	is the lane allocation factor (see Table 11.29).

NOTE The number of cycles for high vehicle buffeting is based on:

1) *the passage of 7000 such vehicles per day on each carriageway; and,*
2) *a total logarithmic decrement of damping of 0.03. Specialist advice is required where damping is less than this value.*

11.27.1 Where flows of high-sided vehicles are less than this average, then the values of n_{2i} should be reduced in proportion.

11.28 Flows of high-sided vehicles shall be determined by traffic survey and the total number of high-sided vehicles cannot be reduced below a value of 1000 for design purposes.

11.29 The corresponding number of cycles to failure, N_{2i} , shall be given by:

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Equation 11.29 Number of cycles to failure for fatigue due to high vehicle buffeting

$$N_{2i} = 2 \times 10^6 \left(\frac{\sigma_o}{\sigma_{r2i}}\right)^m$$

where:

L	is the design life of the structure (years);
σ_o	is the details category (50, 120) (see clause 5.43 and, for relevant details, from clauses 11.9 to 11.18);
F_i	is the lane allocation factor (see Table 11.29); and,
m	is the slope of curve (m is taken as 3 for Figure A.1 and 4 for Figure A.2 as given in Appendix A).

Table 11.29 Lane allocation factors

Type of carriageway	Lane Allocation Factors, F_i			
	Lane 1	Lane 2	Lane 3	Lane 4
D2M	0.7	0.3	-	-
D3M	0.6	0.4	0.0	-
D4M	0.4	0.4	0.2	0.0

Fatigue due to combined gust wind loading and high vehicle buffeting

11.30 The fatigue effects from wind gust loading and high vehicle buffeting shall be combined.

11.31 The combined fatigue effects shall satisfy the criterion in Equation 11.31:.

Equation 11.31 Criteria for combined fatigue effects

$$\frac{n_1}{N_1} + \sum_{i}^{T} \frac{n_{2i}}{N_{2i}} \leq 1$$

where:

n_1	is the number cycles due to gust wind loading;
n_{2i}	is the number of cycles for each lane in a carriageway;
N_1	is the corresponding number of cycles to failure for fatigue due to gust wind loading;
N_{2i}	is the corresponding number of cycles to failure for fatigue due to high vehicle buffeting;
i	is the first lane directly beneath the cantilever arm of the structure; and,
T	is the number of lanes directly beneath the cantilever arm of the structure.

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12. Foundations

General

12.1 The foundation design shall be subject to:

1) technical approval in accordance with CG 300 [Ref 34.N]; and,
2) geotechnical certification in accordance with CD 622 [Ref 23.N].

12.1.1 The CD 622 [Ref 23.N] geotechnical certification procedures may be condensed as agreed with the Overseeing Organisation to reflect the complexity of the proposed foundation design.

12.2 Foundations shall either consist of:

1) planted columns and posts, designed in accordance with clauses 12.7 and 12.8;
2) planted prefabricated concrete or metal columns designed in accordance with clauses 12.9 to 12.22; or,
3) reinforced concrete, designed in accordance with clauses 12.23 to 12.33.

12.3 Alternative forms of foundations shall be subject to approval by Overseeing Organisation.

NOTE *Unreinforced concrete spread foundations rely on the tensile strength of concrete and are unlikely to meet the requirements for durability.*

12.4 The design rules given in this section shall not apply to foundations on slopes, where stability of the ground needs to be factored into the design.

12.5 For foundations on slopes and where stability of the ground needs to be factored into the design, specialist geotechnical advice shall be sought.

NOTE *Some guidance on the design of supports for sign structures can be found in Sign Structures Guide [Ref 33.N] Chapter 5.*

12.6 Foundations types adopted for columns and posts for CCTV masts or speed cameras shall be designed to factor in the effects of ground movements on the operation of equipment on the column or post.

NOTE *The general provisions in this section for planted foundations are unlikely to provide the restraint required to limit deflection for CCTV masts or speed cameras so more detailed geotechnical investigation and design is necessary.*

Foundations for planted columns, posts and prefabricated foundations

Planting depth

12.7 Where a minor structure is to be planted directly in the ground, the planting depth shall be selected from Table 7 of BS EN 40-2 [Ref 21.N] related to the overall height of the structure.

12.7.1 Where the nominal height is less than 2 m, a depth of 600 mm may be adopted, provided that , $M_g > \gamma_{s;d} \times M_{DS}$.

12.8 The calculation procedure given below shall be used to check the adequacy of the selected planting depth taking into account the ground conditions at the site.

12.9 For prefabricated foundations, the planting depth and effective diameter shall be selected to ensure compliance with the calculation method provided.

12.10 The greatest destabilising moment, M_{DS} , arising from application of the unfactored design loads (such as wind load or dynamic load from snow clearance) to the minor structure and its supports shall be calculated with the fulcrum point located at $1/\sqrt{2}$ of the planting depth below ground P .

12.10.1 The destabilising moment may be obtained from the designer where it is not calculated.

12.11 The greatest destabilising moment, M_{DS} , shall be multiplied by a model factor, $\gamma_{s;d}$ of 1.25 (i.e. $\gamma_{s;d} \times M_{DS}$).

12.12 The ground resistance moment, M_g , shall be calculated using the following formula:

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Equation 12.12 Ground resistance moment

$$M_g = \frac{G \times D \times P^3}{10}$$

where:

G is the soil factor. It is dependent on the ground in which the column is planted (in kN/m^2 per m). Refer to Table 12.12 for typical values of G ;

D diameter in accordance with clause 12.14; and,

P is the planting depth below ground.

Table 12.12 Ground factor, G, and soil impact factor, ksi.

Quality of Soil	Soil description/characteristics	Ground factor G (kN/m^2 per m)	Soil impact factor k_{si}
Good	Compact, well-graded sand and gravel, hard clay, well-graded fine and coarse sand, decomposed granite rock and soil. Good soils drain well.	630	0.2
Average	Compact fine sand, medium clay, compact well drained sandy loam, loose coarse sand and gravels. Average soils drain sufficiently well that water does not stand on the surface.	390	0.3
Poor	Soft clay, clay loam, poorly compacted sand, clays containing a large amount of silt and vegetable matter, and made-up ground.	230	0.5

12.12.1 The values of the suitable ground factor, G , taken from Table 12.12 may be used for lighting columns, traffic sign/signal posts and environmental barriers,.

12.13 Where the quality of soil is unknown, it shall be taken as poor.

12.14 The diameter D shall be the minimum diameter (or minimum distance across the flats for multi-sided sections) of the post in the ground (in m).

12.15 The planting depth below ground shall be satisfactory if $M_g > \gamma_{s;d} \times M_{DS}$.

12.15.1 Where $M_g \leq \gamma_{s;d} \times M_{DS}$, then the ground moment of resistance may be improved by:

1) increasing the planting depth; and/or,
2) increasing the effective diameter of the minor structure.

NOTE The calculation of ground resistance moment, M_g , is based on the excavated hole into which the minor structure is planted being backfilled with the excavated material or material of better quality.

Backfilling

12.16 Backfilling material shall be mass concrete or an acceptable material in accordance with MCHW1 SHW MCHW Series 0600 [Ref 24.N].

12.16.1 Excavated material from the hole dug for the minor structure foundation may be reused as acceptable material.

12.17 All backfilling material shall be placed in 150mm-thick layers and well compacted.

12.18 Where precast foundations are used, the backfilling material and procedure shall be in accordance with the foundation manufacturers requirements.

12.19 The corrosion protection system for the minor structure shall not be damaged during compaction.

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12.20 Where the excavated hole is backfilled with concrete, the concrete shall extend from the base of the minor structure to ground level.

12.20.1 Where paving or bituminous surfacing is to be applied around the minor structure, the top level of the concrete may be reduced by the thickness of the surfacing.

12.21 Planted columns shall incorporate a mechanism which prevents rotation of the column or post in the ground under wind loading to resist the torsional effects of the loading.

12.22 Settlement due to ground movement shall be reflected in the design of planted columns.

NOTE Settlement of the planted column or post due to ground movement can affect the headroom, clearances or height limits allowed in the design.

Foundations for columns with flange plates

12.23 The design principles of foundations shall be based on the design methods given in BS EN 1997-1 [Ref 9.N].

12.24 The foundation shall be designed to resist the foundation design moment, M_{fd} , and foundation design shear force, F_{fd} .

12.25 M_{fd} shall be the greater of the impact moment, M_i , and the moment obtained from BS EN 40-3-1 [Ref 19.N], BS EN 12899-1 [Ref 14.N], PLG 07 [Ref 28.N] or BS EN 1991 [Ref 11.N] factored by the appropriate partial factor for an action, γ_F (refer to BS EN 1997-1 [Ref 9.N]), for the failure mode under consideration.

12.26 F_{fd} shall be the greater of the impact shear force, F_i , and the horizontal force obtained from BS EN 40-3-1 [Ref 19.N], BS EN 12899-1 [Ref 14.N] , PLG 07 [Ref 28.N], BS EN 1991 [Ref 11.N], or BS EN 1991 [Ref 11.N] factored by the appropriate partial factor for an action, γ_F (Refer to BS EN 1997 [Ref 8.N]), for the failure mode under consideration).

12.27 The partial factor for an action shall be taken as:

1) $\gamma_{G;dst} \geq 1.5$ for destabilising actions (such as overturning moment); and,
2) $\gamma_{G;stb} \leq 0.9$ for stabilising actions (such as gravity).

12.28 The impact moment and impact shear force, M_i and F_i respectively shall be derived as follows:

Equation 12.28a Impact moment

$$M_i = k_{si} M_R$$

where:

M_i	is the impact moment (in N.m);
M_R	is the ultimate moment of resistance (in N.m); and,
k_{si}	is the soil impact factor as given in Table 12.12.

Equation 12.28b Impact shear force

$$F_i = k_{si} F_R$$

where:

F_i	is the impact shear force (in N);
F_R	is the equivalent ultimate shear force (in N); and,
k_{si}	is the soil impact factor as given in Table 12.12.

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12.29 The ultimate moment of resistance of the actual column at the base level, M_R , and the equivalent ultimate shear force, F_R , shall be calculated in accordance with clause 5.6.2 of BS EN 40-3-3 [Ref 20.N] .

12.29.1 An upper bound value for the equivalent ultimate shear force may be taken as: $F_R = 2 \times M_{up}$

NOTE 1 This assumes that the point of impact is 0.5 m above the top of the foundation.

NOTE 2 Refer to BS EN 40-3-3 [Ref 20.N] for the calculation of M_{up} which is the bending moment of resistance for closed rectangular cross sections.

Foundations for cantilever masts with flange plates

12.30 Cantilever masts positioned in locations where the quality of soil is unknown, and therefore taken as being poor, the foundations shall be designed in accordance with the procedure given in the clauses below.

12.31 Foundations shall consist of reinforced concrete blocks designed in accordance with BS EN 1992-1-1 [Ref 5.N].

12.32 The design loads for the foundation shall be the nominal loads and nominal wind loading applied by the cantilever mast when designed in accordance with this document, factored by the relevant partial factor for actions, γ_F taken from BS EN 1997-1 [Ref 9.N].

12.33 The design of the foundation shall be based on the design methods given in BS EN 1997-1 [Ref 9.N], using the partial factors on actions taken as:

1) $\gamma_{G;dst} \geq 1.5$ for destabilising actions (such as overturning moment); and,
2) $\gamma_{G;stb} \leq 0.9$ for stabilising actions (such as gravity).

12.33.1 There is a difference in the structural behaviour of the cantilever mast and its foundation, hence in the absence of more accurate information, the following may be assumed:

1) the basic wind load transferred from the cantilever mast to the substructure at the top of the substructure reduces to $\frac{1}{\beta}$ of this value, at the bottom of the substructure and foundation.
2) β is the factor for dynamic behaviour given in BS EN 40-3-1 [Ref 19.N] Clause 5.2.4.

12.34 Unless otherwise agreed with the Technical Approval Authority, the criteria given for foundation for columns with flange plates shall apply when cantilever masts are positioned in locations other than those where the quality of soil is taken as poor.

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13. Normative references

The following documents, in whole or in part, are normative references for this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Ref 1.N	Highways England. CG 302, 'As-built, operational and maintenance records for highway structures'
Ref 2.N	Highways England. CD 127, 'Cross-sections and headrooms'
Ref 3.N	BSI. BS EN 1991-1-4, 'Eurocode 1: Actions on structures. Part 1-4: General actions – Wind actions'
Ref 4.N	BSI. BS EN 1992, 'Eurocode 2. Design of concrete structures'
Ref 5.N	BSI. BS EN 1992-1-1, 'Eurocode 2: Design of concrete structures. General rules and rules for buildings'
Ref 6.N	BSI. BS EN 1993-3-1, 'Eurocode 3 - Design in steel structures - Towers, masts and chmineys - towers and masts'
Ref 7.N	BSI. BS EN 1993-1-9, 'Eurocode 3. Design of steel structures. Fatigue.'
Ref 8.N	BSI. BS EN 1997, 'Eurocode 7 - Geotechnical Design.'
Ref 9.N	BSI. BS EN 1997-1, 'Eurocode 7: Geotechnical design - Part 1: General rules'
Ref 10.N	BSI. BS EN 1999-1-3, 'Eurocode 9: Design of aluminium structures. Structures susceptible to fatigue'
Ref 11.N	BSI. BS EN 1991, 'Eurocode 1: Actions on structures'
Ref 12.N	BSI. BS EN 1992-1, 'Eurocode 2: Design of concrete structures'
Ref 13.N	BSI. BS EN 1999 , 'Eurocode 9: Design of Aluminium Structures'
Ref 14.N	BSI. BS EN 12899-1, 'Fixed, vertical road traffic signs. Fixed signs'
Ref 15.N	Highways England. CD 109, 'Highway link design'
Ref 16.N	Highways England. CG 305, 'Identification marking of highway structures'
Ref 17.N	National Highways. GG 101, 'Introduction to the Design Manual for Roads and Bridges'
Ref 18.N	BSI. BS EN 40, 'Lighting Columns (All Parts)'
Ref 19.N	BSI. BS EN 40-3-1, 'Lighting columns. Design verification. Specification for characteristic loads '
Ref 20.N	BSI. BS EN 40-3-3, 'Lighting columns. Design verification. Verification by calculation'
Ref 21.N	BSI. BS EN 40-2, 'Lighting columns. General requirements and dimensions'
Ref 22.N	BSI. BS EN 40-7, 'Lighting columns. Requirements for fibre reinforced polymer composite lighting columns'
Ref 23.N	Highways England. CD 622, 'Managing geotechnical risk'
Ref 24.N	Highways England. MCHW Series 0600, 'Manual of Contract Documents for Highway Works, Volume 1 Specification for Highway Works. Series 600 Earthworks'
Ref 25.N	Highways England. MCHW Series 1900, 'Manual of Contract Documents for Highway Works, Volume 1 Specification of Highways Works, Series 1900, Protection of Steelwork against Corrosion'
Ref 26.N	Highways England. CD 365, 'Portal and cantilever signs/signals gantries'

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Ref 27.N	Highways England. CD 146, 'Positioning of signalling and advance direction signs'
Ref 28.N	Institution of Lighting Professionals. PLG 07, 'Professional Lighting Guide PLG 07 High Masts for Lighting and CCTV (2013 edition)'
Ref 29.N	Highways England. CD 377, 'Requirements for road restraint systems'
Ref 30.N	Highways England. TD 501, 'Road lighting'
Ref 31.N	BSI. BS EN 1317-2, 'Road restraint systems. Performance classes, impact test acceptance criteria and test methods for safety barriers including vehicle parapets '
Ref 32.N	BSI. BS EN 14388, 'Road traffic noise reducing devices. Specifications'
Ref 33.N	Institute of Highway Engineers (2010). Gallagher, J; Morris, K; Morgan, S. Sign Structures Guide, 'Support design for permanent UK traffic signs - Revised for Eurocodes and passive safety'
Ref 34.N	Highways England. CG 300, 'Technical approval of highway structures'
Ref 35.N	BSI. BS EN 1011-1, 'Welding. Recommendations for welding metallic materials. Guidance for arc welding'
Ref 36.N	BSI. BS EN 1011-2, 'Welding. Recommendations for welding of metallic materials. Arc welding of ferritic steels'

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14. Informative references

The following documents are informative references for this document and provide supporting information.

Ref 1.I	BSI. BS EN 1993 , 'Eurocode 3: Design of steel structures'
Ref 2.I	BSI. PD 6547, 'Guidance on the use of BS EN 40-3-1 and BS EN 40-3-3.'
Ref 3.I	Concrete Society. CS CA42, 'Holding down bolt design to Eurocode 2'
Ref 4.I	BSI. BS EN 12767, 'Passive safety of support structures for road equipment. Requirements, classification and test methods.'
Ref 5.I	BSI. BS EN ISO 7093, 'Plain washers. Large series.'

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Appendix A. Fatigue stress curves and guidance for weld classification

A1 Fatigue stress curves

For a design life of 25 years, the maximum allowable stress range is given in the fatigue life curves, Figure A.1 or Figure A.2, appropriate to the class of detail under consideration and dependent on the frequency (Hz).

These curves are for design and incorporate a partial factor on fatigue strength. The method of defining the S-N curves given in Figure A.1 and Figure A.2 is by two numbers joined by a hyphen. The first number is the reference strength at $2x10^6$ cycles and the second is the m value which is a constant applicable to values of up to $5x10^6$ cycles. This is the procedure adopted for defining fatigue strength in BS EN 1993-1-9 [Ref 7.N].

A1.1 Basis of fatigue life curves

Figure A.1 and Figure A.2, the fatigue life curves, are based on:

(a) No.of cycles to failure $N = 2 \times 10^6 \left(\frac{\sigma_o}{\sigma_r}\right)^m$

where:

$N =$ number of cycles

$\sigma_o =$ details category (50, ... 120 ...)

$m =$ slope of curve (m is taken as 3 for Figure A.1 and 4 for Figure A.2)

$\sigma_r =$ stress range

(b) The number of cycles, n , relate to the frequency of vibration, N_f , by Equation 11.8:

$n = 10^6 N_f L$

(c) Thus for a design life, L , of 25 years:

$n = 25 \times 10^6 N_f$

(d) Thus, the relationship between σ_R and N_f (the plots of Figure A.1 and Figure A.2) is:

$2 \times 10^6 \left(\frac{\sigma_o}{\sigma_r}\right)^m = 25 \times 10^6 N_f$

i.e $N_f = \frac{8}{100} \left(\frac{\sigma_o}{\sigma_r}\right)^m$

For example for class detail 120:4:

$\sigma_o = 120$

$m = 4$

$N_f = \frac{8}{100} \left(\frac{120}{\sigma_r}\right)^4$

Table A.1 Values frequency of vibration for given stress ranges

σ_r	100	90	80	70	60	50
N_f	0.166	0.253	0.405	0.691	1.280	2.650

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A1.2 Fatigue life curves

Figure A.1 Fatigue of column stress range limit for class of weld detail based on a 25 year design life requirement (m=3)

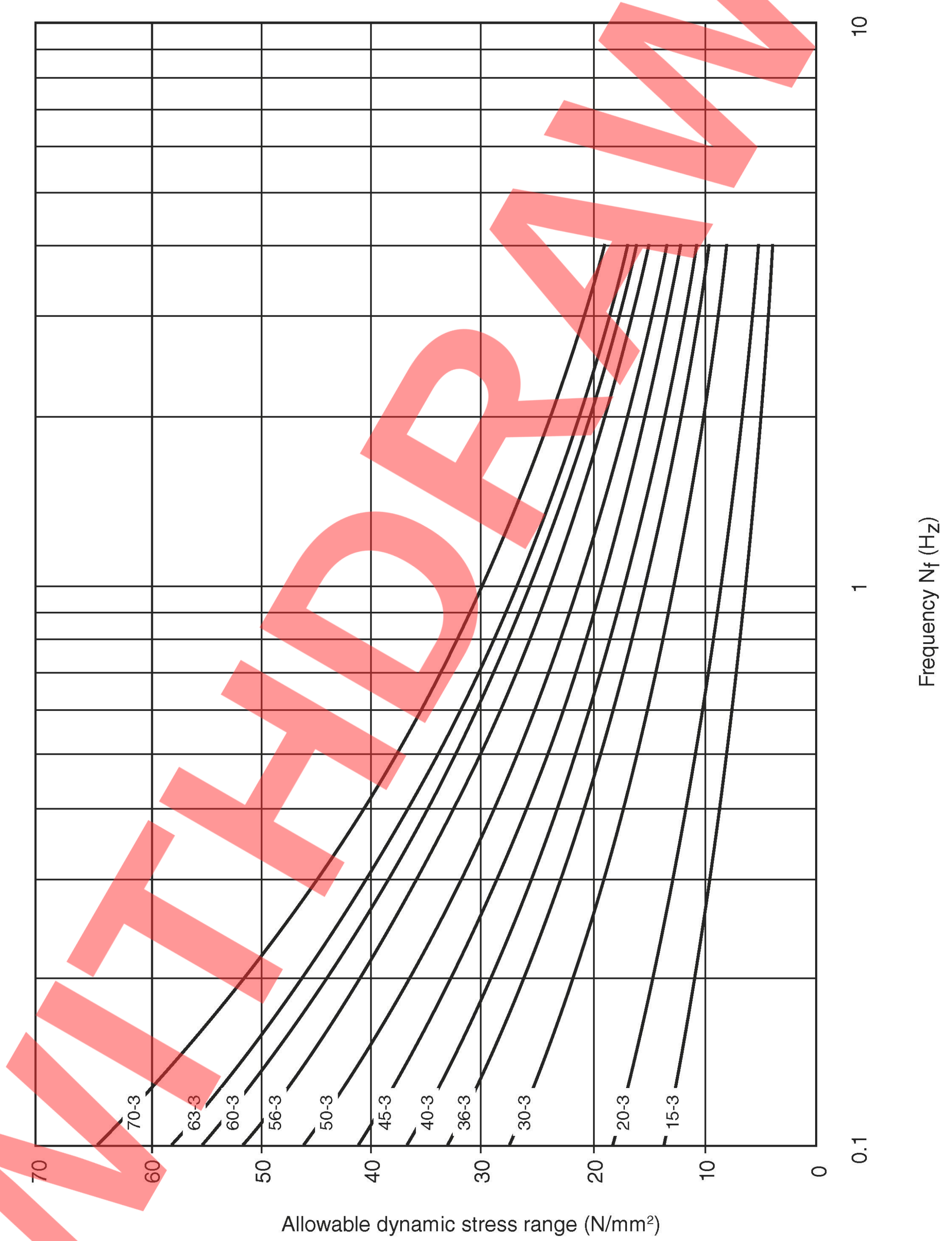

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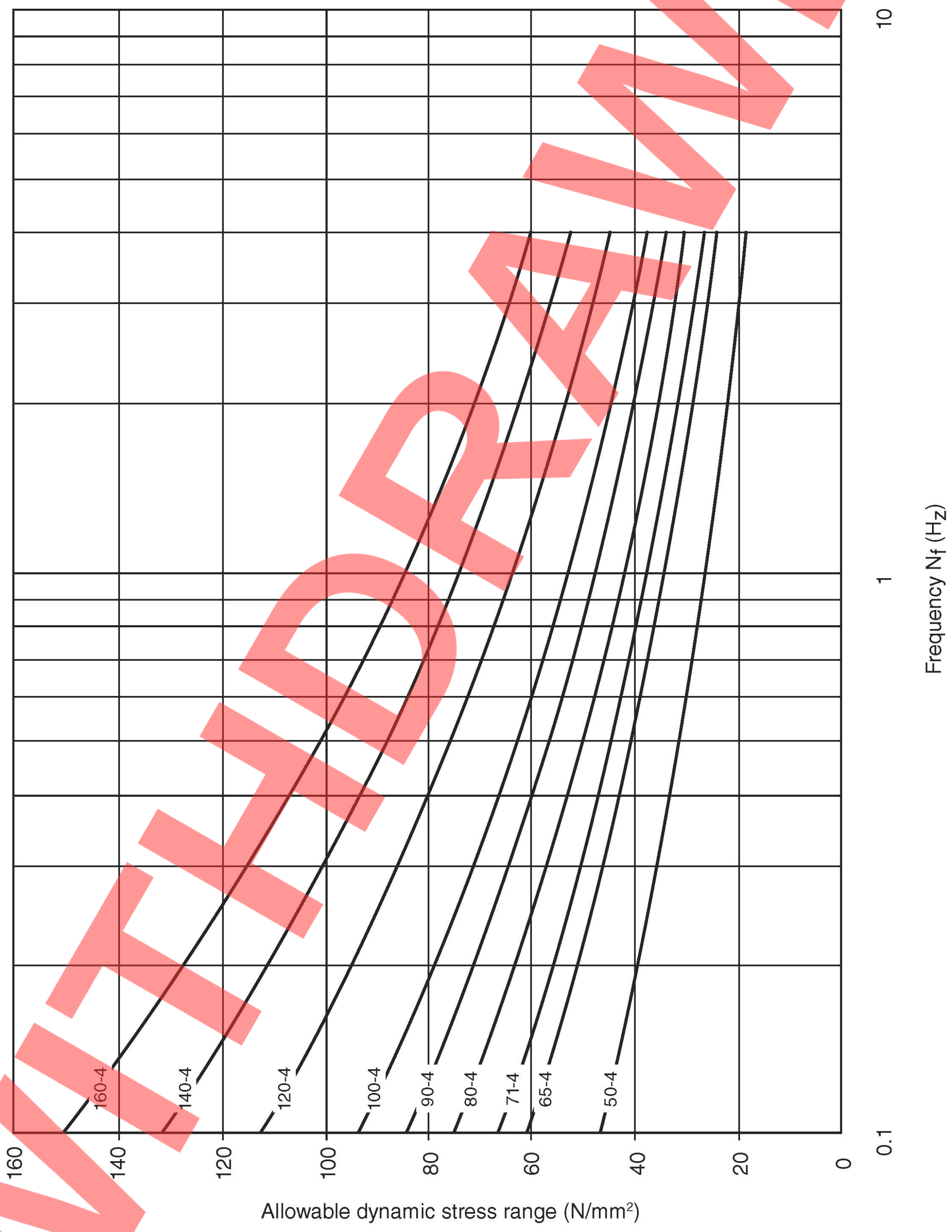
Figure A.2 Fatigue of column stress range limit for class of weld detail based on a 25-year design life requirement (m=4)



A2 Guidance for weld classification

Guidance on classes of typical weld details incorporating stress concentration factors, K_f , which comply with the constraints in A1 are given in Table A.2 to Table A.9 for welds made using normal commercial practice, e.g. manual welds without NDT or other testing.

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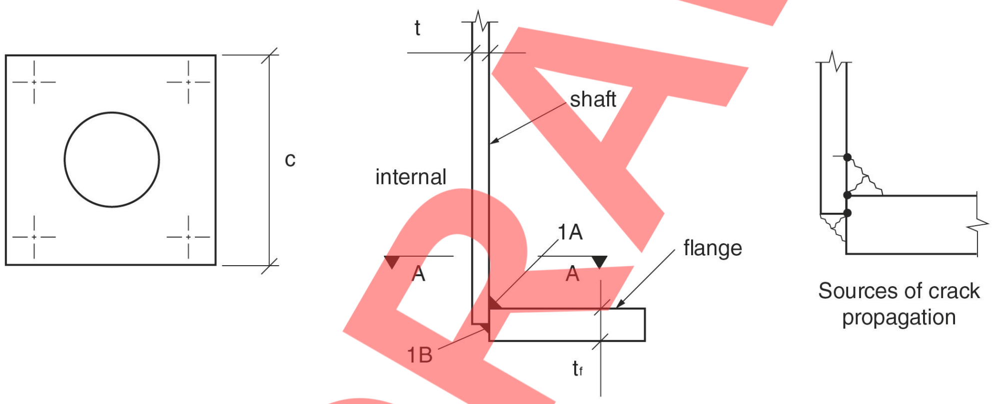
This guidance was based on fatigue tests of a representative number of details provided by a range of UK lighting column manufacturers. However classification is critically dependent on welding quality and fabrication methods, and hence the information provided is for guidance only.

Closer control of the welding and fabrication process and/or post-weld treatment may improve the weld classification. For other welded details specialist advice should be sought and reference made to BS EN 1993-1-9 [Ref 7.N].

A2.1 Weld classification details

Table A.2 Weld detail type 1



Weld	Section to be checked	Class of parent metal	$K_f = K_t K_b K_h$
1A 1B	A-A	$30K_f - 4$ (1)	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_t = \left(\frac{\sqrt{t_f}}{5}\right)^3$ where $K_b \geq 0.33$ and ≤ 1.45 $K_h = 1.0$

Notes:

1) Provided weld 1A is designed for transfer of the total load and weld 1B is for sealing only. Otherwise a detailed stress analysis is to be undertaken and the resulting stress concentration factors used.
2) No fatigue check need be undertaken on the weld throat provided the criteria of geometrical configurations and fabrication constraints in section 11 are met. The parent metal still needs to be checked.

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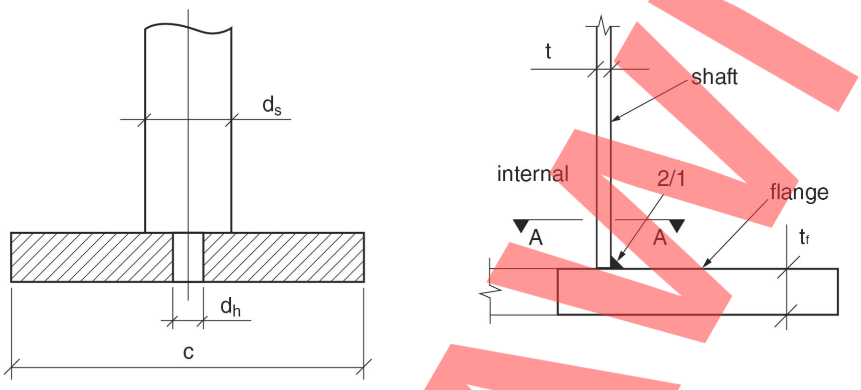
Table A.3 Weld detail type 2/1



Weld	Section to be checked	Class of parent metal	$K_f = K_t K_b K_h$
2/1	A-A	$30K_f - 4$	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_b = \left(\frac{\sqrt{t_f}}{5}\right)^3$ where $K_b \geq 0.33$ and ≤ 1.45 $K_h = \frac{2}{1+\left(\frac{d_h}{d_s-2t}\right)^3}$

Note: No fatigue check need be undertaken on the weld throat provided the criteria of geometrical configurations and fabrication constraints in section 11 are met. The parent metal still needs to be checked.

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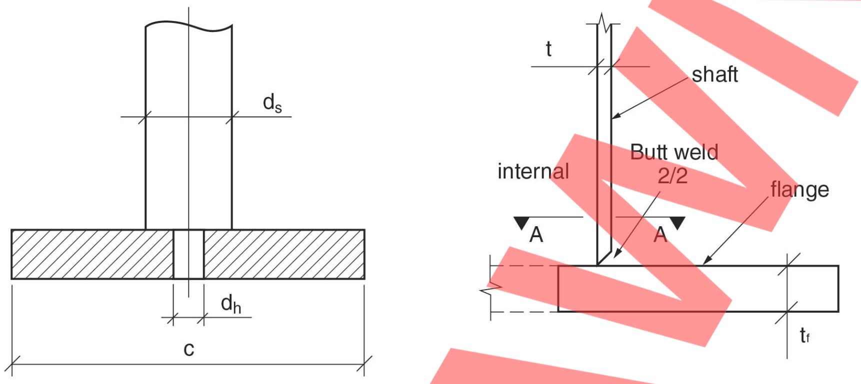
Table A.4 Weld detail type 2/2



We-ld	Section to be checked	Class of parent metal	$K_f = K_t K_b K_h$
2/2	A-A	$30K_f - 4$	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_b = \left(\frac{\sqrt{t_f}}{5}\right)^3$ where $K_b \geq 0.33$ and ≤ 1.45 $K_h = \frac{2}{1+\left(\frac{d_h}{d_s-2t}\right)^3}$

Note: No fatigue check need be undertaken on the weld throat provided the criteria of geometrical configurations and fabrication constraints in section 11 are met. The parent metal still needs to be checked.

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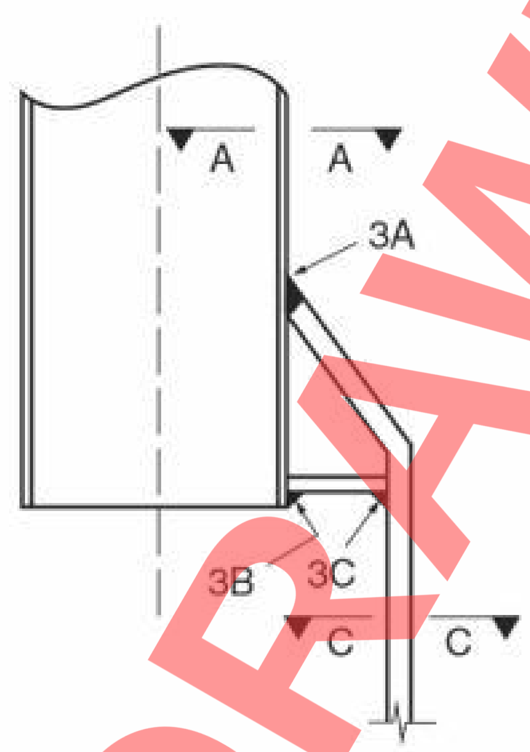
Table A.5 Weld detail type 3



Weld	Section to be checked	Class	
		Parent metal	Weld throat
3A	A-A	$90 - 4$ (1)	See (2)
3B	See (3)	$90 - 4$ (1)	See (2)
3C	C-C		

Notes:

1) Incorporates stress concentration factor, take $K_f = 1.0$
2) No fatigue check need be undertaken on the weld throat provided the criteria of geometrical configurations and fabrication constraints in section 11 are met. The parent metal still needs to be checked.
3) No check necessary on the section if the criteria of geometrical configurations and fabrication constraints in section 11 are met.

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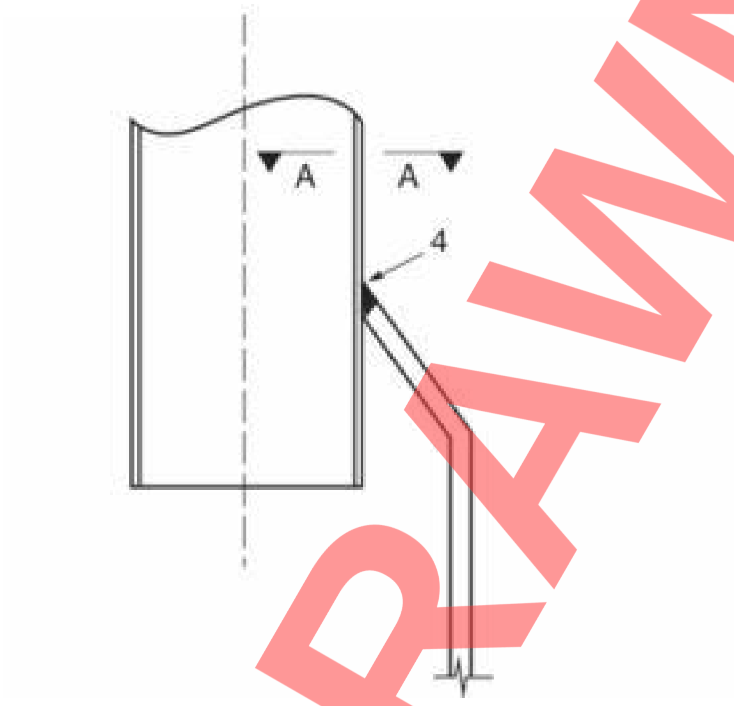
Table A.6 Weld detail type 4



Weld	Section to be checked	Class	
		Parent metal	Weld throat
4	A-A	$71 - 4$ (1)	See (2)

General note: Joint detail not recommended for other than lightly loaded short columns.
Other notes:
(1) Incorporates stress concentration factor, take $K_f = 1.0$
(2) No fatigue check need be undertaken on the weld throat provided the criteria of geometrical configurations and fabrication constraints in Section 11 are met. The parent metal is still to be checked.

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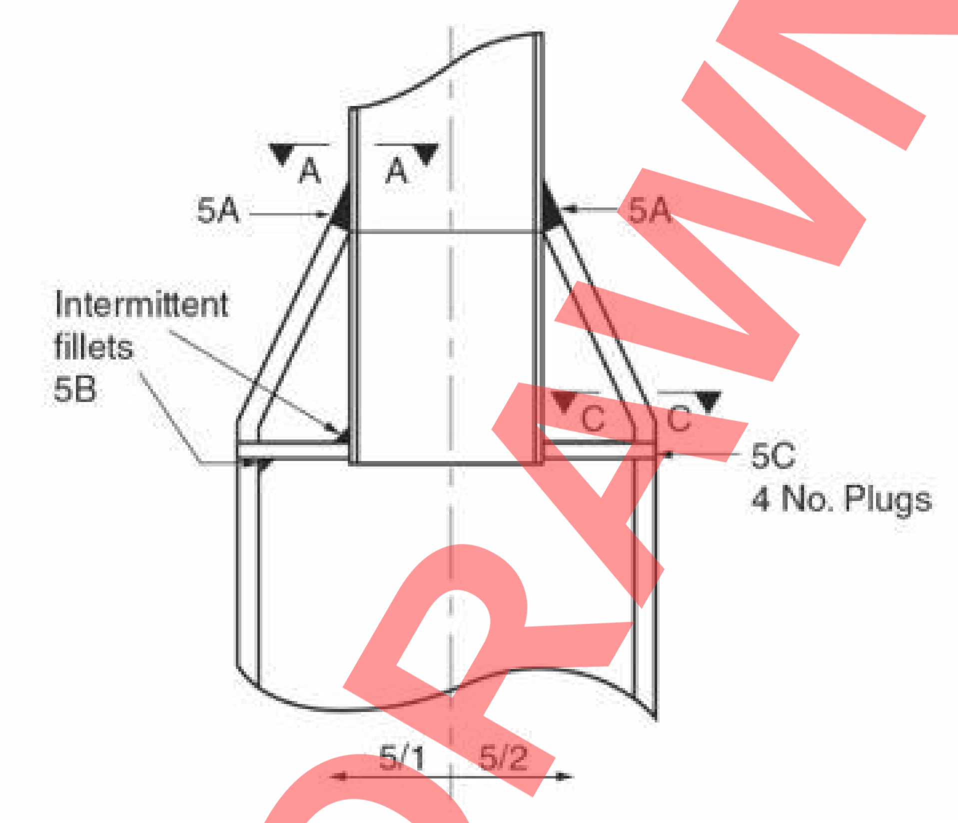
Table A.7 Weld detail type 5



Weld	Section to be checked	Parent metal	Weld throat
5A	A-A	$90 - 4$ (1)	See (2)
5B	See (3)	$120 - 4$ (1)	See (2)
5C	C-C. Plugs not ground smooth	$90 - 4$	-
	C-C . Plugs ground smooth	$120 - 4$	-

1) Incorporates stress concentration factor. Take $K_f = 1.0$.
2) No fatigue check need be undertaken on the weld throat provided the criteria of geometrical configurations and fabrication constraints in Section 11 are met. The parent metal is still to be checked.
3) No check necessary on the section if the criteria of geometrical configurations and fabrication constraints in section 11 are met.

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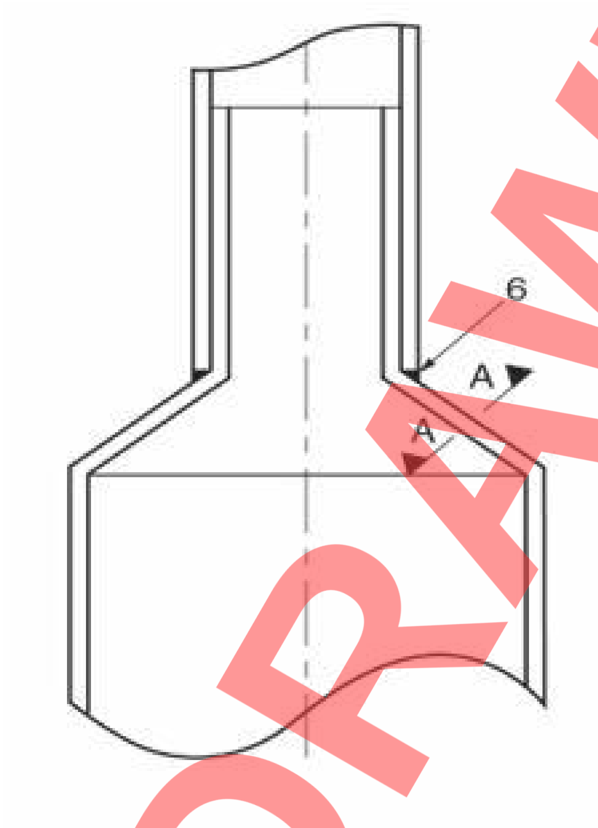
Table A.8 Weld detail type 6



Wel-d	Section to be checked	Class		
		Upper tube parent metal	Lower tube parent metal	Weld throat
6	A-A	N/A (1)	$71 - 4$	See (2)

1) Assumes tight fit between tubes for load transfer by shear.
2) No fatigue check need be undertaken on the weld throat. The parent metal still needs to be checked.

General note:
For this joint to behave as intended, the lapped length is to be at least 1.5 times the diameter of the lapped shaft.

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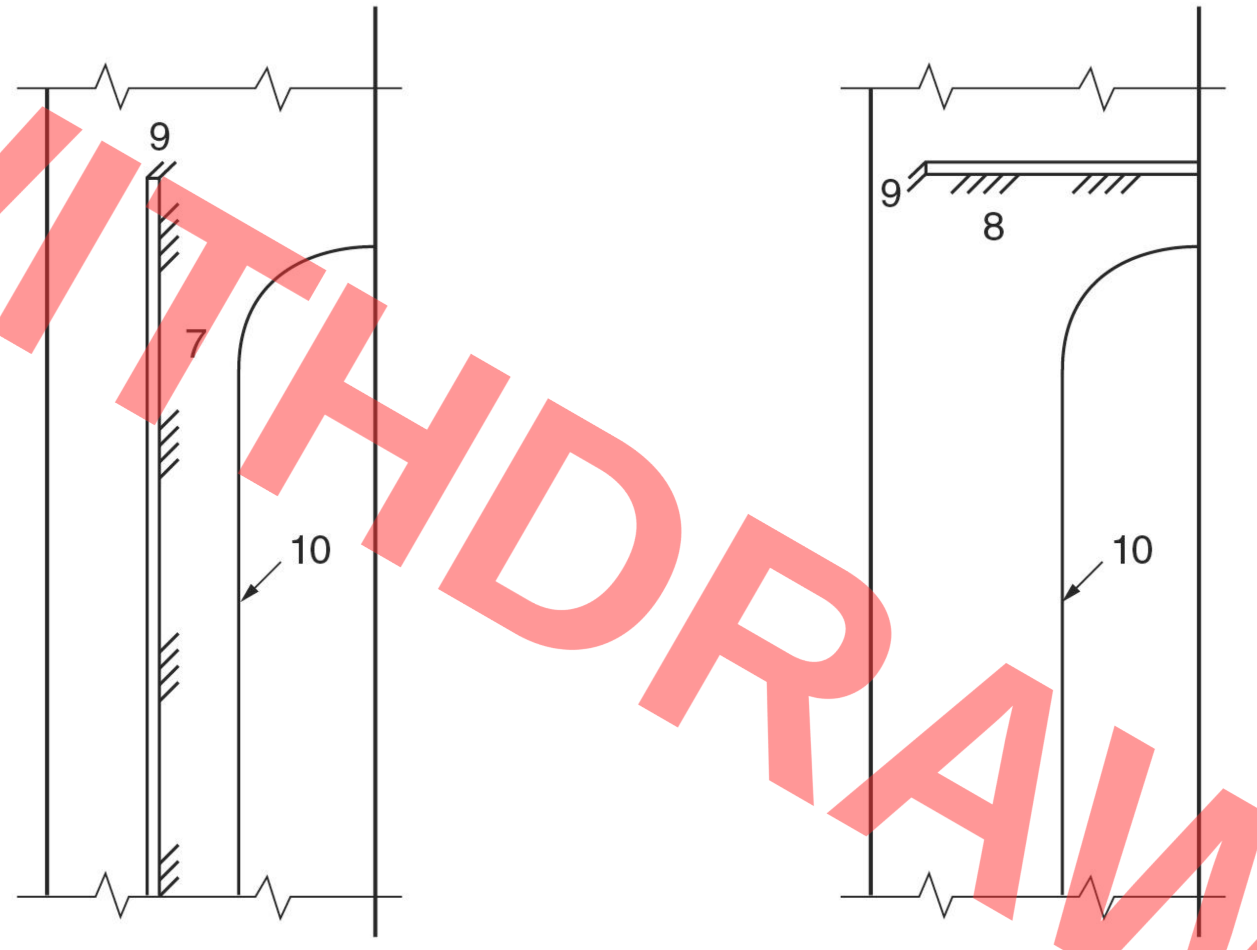
Table A.9 Weld detail type 7 to 10



Welded stiffener adjacent to opening parallel to column

Welded stiffener adjacent to opening transverse to column

Weld	Section to be checked	Class

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Table A.9 Weld detail type 7 to 10 (continued)

7	Intermediate weld	$80 - 3$
8	Intermediate weld	$71 - 3$
9	End weld	$50 - 3$
10	Flame cut edge	$112 - 4$

General note:
No fatigue stress calculations need be undertaken provided the criteria of geometrical configurations and fabrication constraints in Section 11 have been met. Otherwise the above classification can be adopted in conjunction with a detailed stress analysis incorporating appropriate stress concentration factors.

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Appendix B. Determination of shape coefficients by testing

B1 Shape coefficients for columns

Properly conducted wind tunnel tests on columns and brackets have to be undertaken only when shape coefficients are not available from BS EN 40-3-1 [Ref 19.N] or from recognised International Standards. Adoption of values from these standards or from wind tunnel tests are to be agreed with the Technical Approval Authority. Particular care should be taken to ensure that the values of shape coefficients relate to cross-sections of members of infinite length.

Wind tunnel tests to establish shape coefficients should be carried out using full scale specimens which accurately represent the final proposed column. The forces on the specimen should be measured in the direction of the air flow and the direction normal to the air flow.

Previous wind tunnel tests have indicated that small angular rotations of specimens can cause considerable differences in shape coefficients. The specimens are therefore to be turned in the wind tunnel and measurements taken at angular increments. In the region of each shape coefficient the measurements should be reduced to approximately 1° of rotation. Comparisons are to be made with the values of similar sections given in recognised International Standards as part of the adoption and agreement procedure with the Technical Approval Authority set out in Section 4 of this document.

B2 Shape coefficients for luminaires, cameras, signs and brackets

The shape and lift coefficients for luminaires, cameras and signs may be determined from wind tunnel tests as required by BS EN 40-3-1 [Ref 19.N]. These tests have to be carried out on a full scale shape of the element in a wind tunnel sufficiently large to reduce side effects to an insignificant level. The surface condition of the specimen has to accurately represent that of the production version. Where optional attachments are to be made to the element, such as photo-electric control units, gear component extensions and suchlike, these should be included in the test specimen.

When carrying out a wind tunnel test, forces both in the direction of the airflow and in the direction normal to the airflow have to be measured, as shape and lift coefficients are required for all the increments of rotation for which the forces on an element are to be measured. All shape coefficients should be based on the projected area of the element normal to the airflow.

Forces on an element are to be measured at increments of rotation of approximately 1° between the limit of ±10° to the horizontal. BS EN 40-3-1 [Ref 19.N] requires the maximum value between ±5° to the horizontal but a more conservative value should be adopted where large increases of coefficients are obtained between 5° and 10° to the horizontal. During testing the effects of small plan rotations about the point of fixing are also to be taken into account. Where an increase in shape coefficient is obtained with a rotation within the limits of ±10° then this value has to be adopted.

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