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## TII Publications



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# Design of Support Structures for Roadside Furniture

**DN-STR-03018**  
December 2014

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## Document Attributes

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## NRA DMRB and MCDRW References

For all documents that existed within the NRA DMRB or the NRA MCDRW prior to the launch of TII Publications, the NRA document reference used previously is listed above under 'historical reference'. The TII Publication Number also shown above now supersedes this historical reference. All historical references within this document are deemed to be replaced by the TII Publication Number. For the equivalent TII Publication Number for all other historical references contained within this document, please refer to the TII Publications website.

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## **Design of Support Structures for Roadside Furniture**

**December 2014**

**Summary:**

This Standard covers the design of support structures for roadside furniture, 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 IS EN 40 and IS EN 12899.

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**VOLUME 2 ROAD STRUCTURES:  
DESIGN (SUBSTRUCTURES AND SPECIAL  
STRUCTURES), MATERIALS**

**SECTION 2 SPECIAL STRUCTURES**

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**PART 1**

**NRA BD 94/14**

**DESIGN OF SUPPORT STRUCTURES FOR  
ROADSIDE FURNITURE**

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# 1. INTRODUCTION

- 1.1 Support structures for roadside furniture refers to support structures (excluding foundations) such as lighting columns; CCTV masts; traffic signal masts; fixed vertical posts for traffic signs; and the like. In general, these are proprietary systems supplied by a manufacturer. It is the responsibility of the Design Engineer to ensure that when these products are used on national road schemes, they are fit for purpose and comply with this standard and all relevant product standards such as the Euronorms.
- 1.2 In situations where a particular support structure is being designed from first principles this standard shall be used by the Design Engineer. Chapter 11 should be used in the design of foundations to support structures for roadside furniture.

## General

- 1.3 This Standard covers the use of the relevant parts (as defined herein) of IS EN 40 and IS EN 12899 for the structural design of:
- a) lighting columns made from concrete, steel, aluminium, and fibre reinforced polymer composite (FRPC);
  - b) cantilever masts for traffic signals and/or safety cameras (hereafter called cantilever masts) made from steel;
  - c) closed circuit television (CCTV) masts made from steel;
  - d) fixed vertical road traffic sign/signal posts.

### Notes:

- i) Further guidance and background information in the use of EN 40-3-1 and EN 40-3-3 for the design of lighting columns is given in PD6547: 2004.
- ii) This Standard covers the use of The Institution of Lighting Professionals, Professional Lighting Guide, 'PLG07 High Masts for Lighting and CCTV'. The PLG covers safety, design, assembly, installation, inspection and maintenance of High Masts and has been revised following the publication of the Eurocodes. It replaces the ILE TR07 Report.
- iii) Requirements for the design of fixed vertical road signs are given in IS EN 12899: Part 1. These requirements are supplemented by this Standard.

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

Where materials other than concrete, steel, aluminium or FRPC are used for other support structures a departure from standards shall be sought from the National Roads Authority.

## Scope

- 1.4 This Standard sets out the structural design requirements for the following minor road structures for use on National roads including motorways:
- a) lighting columns and wall mounted brackets made from concrete, steel, aluminium, concrete and FRPC.
  - b) steel CCTV masts mounted on foundations in the ground. The requirements for CCTV masts mounted on other structures e.g. gantries are outside the scope of this Standard;
  - c) cantilever masts made from steel for traffic signals and/or speed cameras. This Standard excludes the design requirements for permanent and temporary cantilever sign and signal gantries for which NRA BD 51 shall be used;
  - d) fixed vertical road traffic sign/signal posts. This Standard excludes the electronic design requirements of certain traffic signs, as defined in IS EN 12899-Part 1.

### Notes:

- i) The structural requirements for lattice structures are outside the scope of this Standard; refer to IS EN 1993-3-1 and the associated Irish National Annex.
- ii) The structural requirements for passively safe structures should comply with this Standard but the passive safety characteristics of such structures are dealt with in IS EN 12767 with guidance in UK HA TD 89.

## 2. GENERAL PRINCIPLES

### Siting

- 2.1 The siting of support structures for roadside furniture shall accord with the TDs and TAs as shown in Table 1 as relevant to the structure considered. This shall include consideration of visibility by the approaching traffic.

	NRA TD9	NRA TD19	NRA TD33	NRA TD34
Lighting columns	✓	✓		✓
Cantilever masts	✓	✓	✓	
CCTV masts	✓	✓		
Road traffic signs	✓	✓	✓	

**Table 1: TDs and TAs**

NOTE: Cantilever masts shall not be located on under-bridges.

### Layout

- 2.2 All elements of support structures for roadside furniture shall comply with the clearances specified in NRA TD 27 after allowing for deflections due to permanent, variable, wind and High Vehicle buffeting loads.
- 2.3 The clear new construction headroom for routes shall be 5,700 mm minimum, as defined in NRA TD 27 Table 7 for Footbridges and Sign/Signal Gantries. In addition to structural deformations, consideration shall be given to settlement when calculating headroom.
- 2.4 Requirements for the vehicle restraint system shall be agreed with the National Roads Authority. The setback of the vehicle restraint system to the edge of the carriageway shall be in accordance with the requirements of the National Roads Authority. Where passively safe signposts, lighting columns or traffic signal posts are provided, in accordance with UK HA TD 89, further vehicle restraint systems shall not be required, unless required by the existence of other hazards.
- 2.5 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 IS EN 1317, Part 2, or other relevant standards.

### Protection for Road Users and Structure

- 2.6 Cantilever masts and CCTV masts shall be located as far from the carriageway as reasonably practicable but no less than either:
- a) more than 4.5 metres from the edge of the carriageway closest to the post (the carriageway is the part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips); or

- b) on a slope such that the underside of the flange plate is more than 2 metres vertically above the edge of the carriageway closest to the post; or
- c) behind a safety barrier conforming to the requirements of NRA TD 19 and an appropriate working width.

Positioning of cantilever masts and CCTV masts in other locations shall be subject to the approval of the National Roads Authority.

- 2.7 Where the post of the cantilever mast or CCTV mast is located behind a vehicle restraint system meeting the requirements of IS EN 1317: Part 2, further vehicle restraint systems are not required.

### **Equipment**

- 2.8 All luminaires, lanterns, brackets, signs, traffic signals, speed cameras and associated equipment shall be securely attached to the structure using vibration resistant fixings strong enough to withstand design loads. The structural design shall make adequate provision for the attachment of equipment. Any subsequent modifications to structural members shall only be carried out with the approval of the National Roads Authority in accordance with NRA BD 02.

### **In-Situ Connections**

- 2.9 In situ connections of main structural metal elements shall be by means of bolts. If other forms of in-situ connection are proposed then their static and fatigue design strength shall be calculated from first principles and shall be agreed with the National Roads Authority. Alternatively, the design strength may be based on the results of full- scale load tests, subject to the agreement of the National Roads Authority.

### **Identification**

- 2.10 The structure site identification marking shall be in accordance with the NRA Eirspan System.

Where not readily identifiable by the design, structures that have been designed to be passively safe to IS EN 12767 shall be marked to differentiate them from other types of structures. The marking system will incorporate the phrase “Crash Friendly” and be placed on the post or column in a position that will not affect the functionality of any part of the assembly or the identification marks required by the NRA Eirspan System. The form of marking appropriate for individual products shall be agreed with the National Roads Authority.

### **Use of Dissimilar Metals**

- 2.11 Where dissimilar metals are to be used, the connections shall be designed to avoid the risk of galvanic corrosion. The electrical bonding of all metal components shall nonetheless be maintained.

### **Protection Against Corrosion**

- 2.12 Surface preparation and paint protection of steel shall comply with the relevant clauses of the Series 1900 in the NRA Specification for Road Works (NRA MCDRW).
- 2.13 For materials other than steel it shall be demonstrated that they will have a life expectancy greater than the service life. (e.g. galvanic corrosion of aluminium due to local ground conditions and UV degradation of FRPC columns)

## 3. DIMENSIONAL LIMITATIONS

### Lighting Columns

- 3.1 The dimensional requirements for lighting columns are given in IS EN 40-2. The overall dimensional limitations for the lighting columns covered by this Standard shall be:

For steel, aluminium, FRPC and concrete columns:

- a) post top columns  
< 20 m nominal height
- b) columns with brackets  
< 18 m nominal height
- c) bracket projections  
- not exceeding the lesser of 0.25 x nominal height or 3 metres

NOTE: Nominal heights and bracket projections are defined in IS EN 40-2: 2004.

### CCTV Masts

- 3.2 The nominal height of steel CCTV masts covered by this Standard shall be less than or equal to 25m. The nominal height is taken as the vertical distance between the underside of the flange plate and the top of the mast.

NOTE: The nominal height excludes the height of camera, mounting etc (refer to Figure 1).

- 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 1).

NOTES: The definitions given in ILP PLG07 should be interpreted as follows:

- a) HIGH MAST shall also refer to CCTV masts, meaning the support intended to hold one or more CCTV cameras with their mountings and housings.
- b) The term LUMINAIRE shall be taken as including CCTV cameras, their mountings and housings.
- c) ILP PLG07 may be used in the design of CCTV masts less than 10m in height.

### Cantilever Masts

- 3.4 For cantilever masts, as shown in Figure 1:

- a) Nominal Height  $\leq$  8.5m

Where nominal height is taken as the distance between the underside of the flange plate and the highest point on the mast. (See Figure 1.)

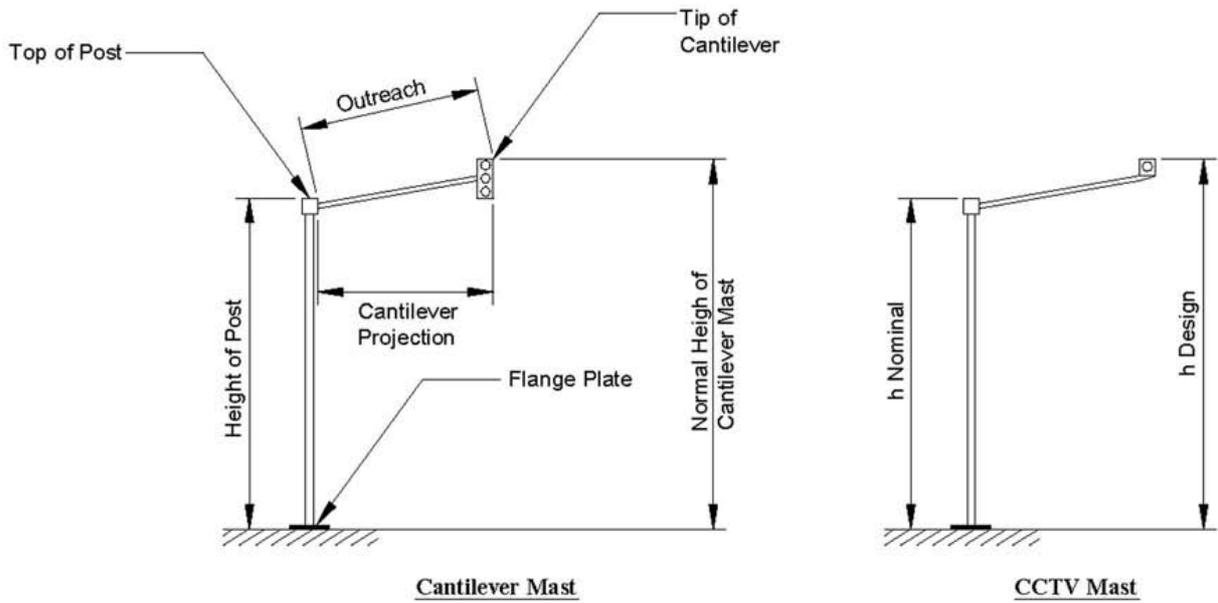
- b) Cantilever Projection  $\leq$  8.5m.

- c) The horizontal projected area of any signs, traffic signals, speed cameras and associated equipment, suspended above the carriageway shall not exceed 1.2m<sup>2</sup> and the vertical projected area shall not exceed 0.3m<sup>2</sup>.

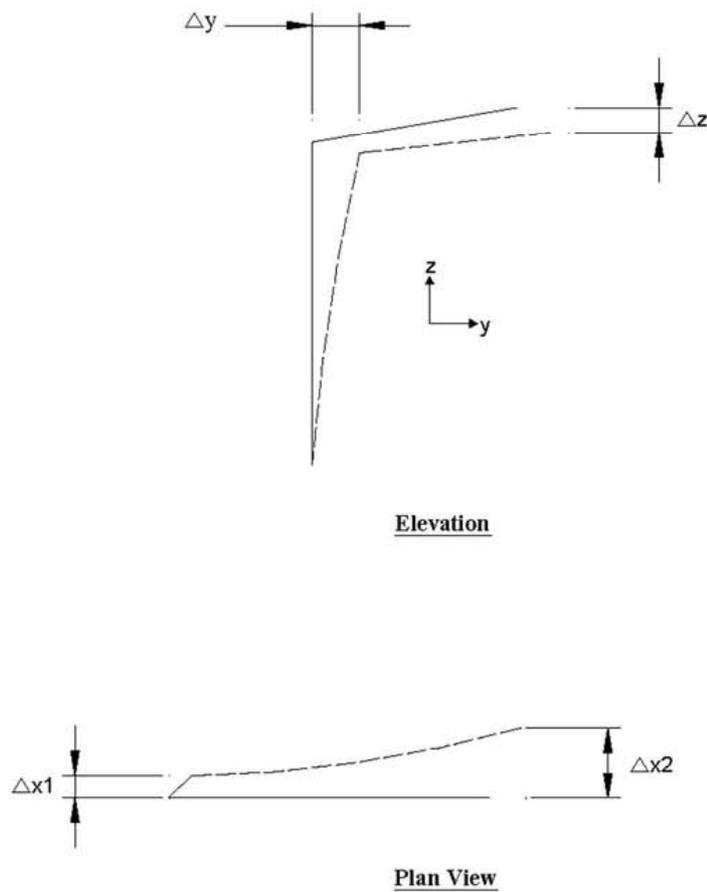
### **Traffic Sign/Signal Posts**

3.5 The nominal height of traffic sign/signal posts shall be  $\leq 9\text{m}$ .

NOTE: Above this height dynamic factors and fatigue shall be considered.



a) General arrangement of cantilever mast and CCTV mast



b) Structural deformations of cantilever mast

Figure 1: General Arrangement of Cantilever Mast and CCTV Mast and Structural Deformations of Cantilever Masts (see Table A2)

## **4. USE OF EUROCODES, IRISH STANDARDS AND STANDARDS ISSUED BY THE NRA**

- 4.1 The design, manufacture and installation of lighting columns and cantilever signal masts shall comply with the following:
- a) Lighting columns and cantilever signal masts – the relevant parts of IS EN 40.
  - b) CCTV masts – ILP PLG 07.
  - c) Road traffic sign posts – IS EN 12899-1.
  - d) The design loading requirements for all structures shall be in accordance with the Eurocodes as implemented by the NRA DMRB.

All as implemented by this Standard and by the NRA Specification for Road Works (NRA MCDRW), hereinafter referred to as “the specification”.

- 4.2 The specific National Roads Authority’s procedures for the Technical Acceptance of structures for use on motorways and National roads are given in NRA BD 02.
- 4.3 For support structures at very exposed sites, the Technical Acceptance procedures for high masts, given in NRA BD 02 shall apply. Lighting columns, CCTV masts, road traffic sign/signal posts and cantilever masts shall be classified as Category 1 in accordance with NRA BD 02.
- 4.4 Within Ireland, very exposed sites are defined as:
- a) sites at high altitude, above 250m;
  - b) sites within 5km from the coast; and sites subject to significant local funnelling.

## 5. DESIGN

### General Requirements

- 5.1 Support structures for roadside furniture shall be designed in accordance with the relevant requirements of the standards listed in paragraph 4.1, as implemented by the NRA Specification for Road Works (NRA MCDRW) and by this Standard.

### Structural Criteria

- 5.2 The design working life category classifications for support structures in compliance with IS EN 1990 Clause 2.3 shall be as described in Table 2 below:

Working Life Category 3 (short term structures, up to 50 years design working life)	Lighting columns, masts for traffic signs/signals
Working Life Category 4 (50-120 years design working life)	CCTV masts; high mast lighting

**Table 2: Design Working Life**

### Limit States

- 5.3 Support structures shall be designed to satisfy the relevant ultimate limit states and the serviceability limit state of the Eurocodes, including, for steel structures, meeting fatigue criteria.

#### *Lighting Columns:*

- 5.4 For lighting columns the partial safety factors and criteria for serviceability and ultimate limit states shall be taken as Class B as given in IS EN 40-3-3.

The horizontal deflections of each lantern connection shall conform to class 2 as specified in IS EN 40 3-3 Table 4.

#### *CCTV Masts:*

- 5.5 For CCTV masts the partial safety factors, partial load factors and criteria for serviceability and ultimate limit states shall be as given in ILP PLG 07.

NOTE: For the serviceability limit state:

- a) Vehicle collision loads do not need to be considered if the requirements of clause 2.6 of this Standard are applicable.

#### *Cantilever masts for traffic signals and/or speed cameras:*

- 5.6 Cantilever masts for traffic signals and/or speed cameras shall meet the criteria of 5.6.1 to 5.6.3.

- a) The Ultimate Limit States, Serviceability Limit States, combination and partial factors for use in the design of structures can be found in IS EN 1990 and the associated Irish National Annex.

Note: Vehicle collision loads shall not be considered if the requirements imposed by clause 2.6 of this Standard are applicable.

- b) In the serviceability limit state, the deflections and rotations due to wind loading only shall be limited such that the deformations do not exceed the values given in Table A1 of Annex A\*.

\* More stringent deflection limits shall be necessary when the performance requirements of the equipment to be mounted so require them.

- c) The 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 1].

### ***Road Traffic Sign Posts:***

- 5.7 For road traffic sign posts the partial safety factors and criteria for serviceability and ultimate limit states are given in IS EN 12899-1.

### **Minimum Thickness of Steel Sections for Cantilever Masts**

- 5.8 The minimum thickness of structural steel sections used in cantilever masts shall be as follows:

- a) plates and sections other than hollow sections: 6mm
- b) hollow sections effectively sealed by welding, other than a small drain hole with a diameter of between 10mm and 15mm: 5mm

### **Closed Hollow Sections for Cantilever Masts**

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

- a) the thickness of the walls of the hollow section;
- b) 8 mm.

The end plates shall be joined by continuous structural quality welding to IS EN 1011: Parts 1 and 2. Should there be a possibility of water entering and subsequently freezing, then drain holes shall be provided. The size of the hole shall be appropriate to the void being drained, but shall not be less than 10 mm or greater than 15 mm diameter. Hollow sections in non-corrosive or galvanised steel shall be provided with such drain holes at all low points.

### **Fatigue Criteria for Steel Structures**

- 5.10 The rules set out in 5.10 to 5.16 shall be used for steel lighting columns 9m and above in height and to all steel cantilever masts. These rules may not be applicable to very exposed sites; in such cases the design shall be subjected to Technical Approval procedures as set out in 4.3. Structures in materials other than steel are not covered by the fatigue rules in this Standard and in such cases the design shall be subjected to Technical Approval procedures as set out in 4.2.

In all cases the procedures to be used for fatigue assessment shall be agreed between the designer, the Employer's Representative and the National Roads Authority, see 4.2 above.

- 5.11 The stringent deflection requirements for the design of CCTV masts mean 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. However for CCTV masts sited in very exposed locations, as defined in 4.4 fatigue shall be considered.
- 5.12 Fatigue damage is most likely to occur at or adjacent to welds or near sharp corners creating stress concentrations; particularly vulnerable positions are:
- a) flange plates:
    - i) at the weld throat between the column and flange;
    - ii) in the parent metal adjacent to the weld;
  - b) door openings:
    - i) at welded attachments;
    - ii) at poorly finished cut edges;
  - c) at any stiffening between the column and the flange;
  - d) shoulder joints:
    - i) at the weld throat;
    - ii) in the parent metal adjacent to the weld.

At such positions, fatigue prone details should be avoided.

- 5.13 Fatigue is critically dependent on geometrical configurations and fabrication. Stiffened and unstiffened door openings should comply with the constraints shown in Figure 2. In addition the following fabrication constraints should be met:
- a) sharp irregularities at free edges due to the flame cutting process should be ground out;
  - b) no welding should be closer than 10mm from the edge of the unstiffened door opening;
  - c) longitudinal edge stiffeners should be continuous over their full extent.
  - d) Where shoulder joints are used, they should have an angle of inclination to the axis of the column of between 12° and 35°. (See Figure 3 which shows a typical shoulder joint)
- 5.14 Generally, when undertaking fatigue checks nominal stresses should be used based on nominal section properties. The stress concentrations inherent in the make-up of a welded joint (arising, for example, from the general joint geometry and the weld shape) are generally taken into account in the classification of the details. Otherwise the nominal stresses should be multiplied by stress concentration factors derived from stress analysis of the joint or from published data.
- 5.15 In order to undertake a fatigue check it is necessary to determine a loading spectra from wind data appropriate to the site. In the absence of such data, the fatigue loading provisions given in Annex B may be adopted.
- 5.16 Classification may be derived by fatigue testing of a sample of typical full-scale details in an independent testing laboratory and covering an appropriate stress range to enable a fatigue life curve to be derived. Sufficient tests should be undertaken to provide a design curve representing mean minus 2 standard deviations.

- 5.17 In the absence of data on fatigue life curves and loading spectra, the procedure set out in Annex B shall be followed.

**Determination of Shape Coefficients**

- 5.18 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 Annex D of this Standard.

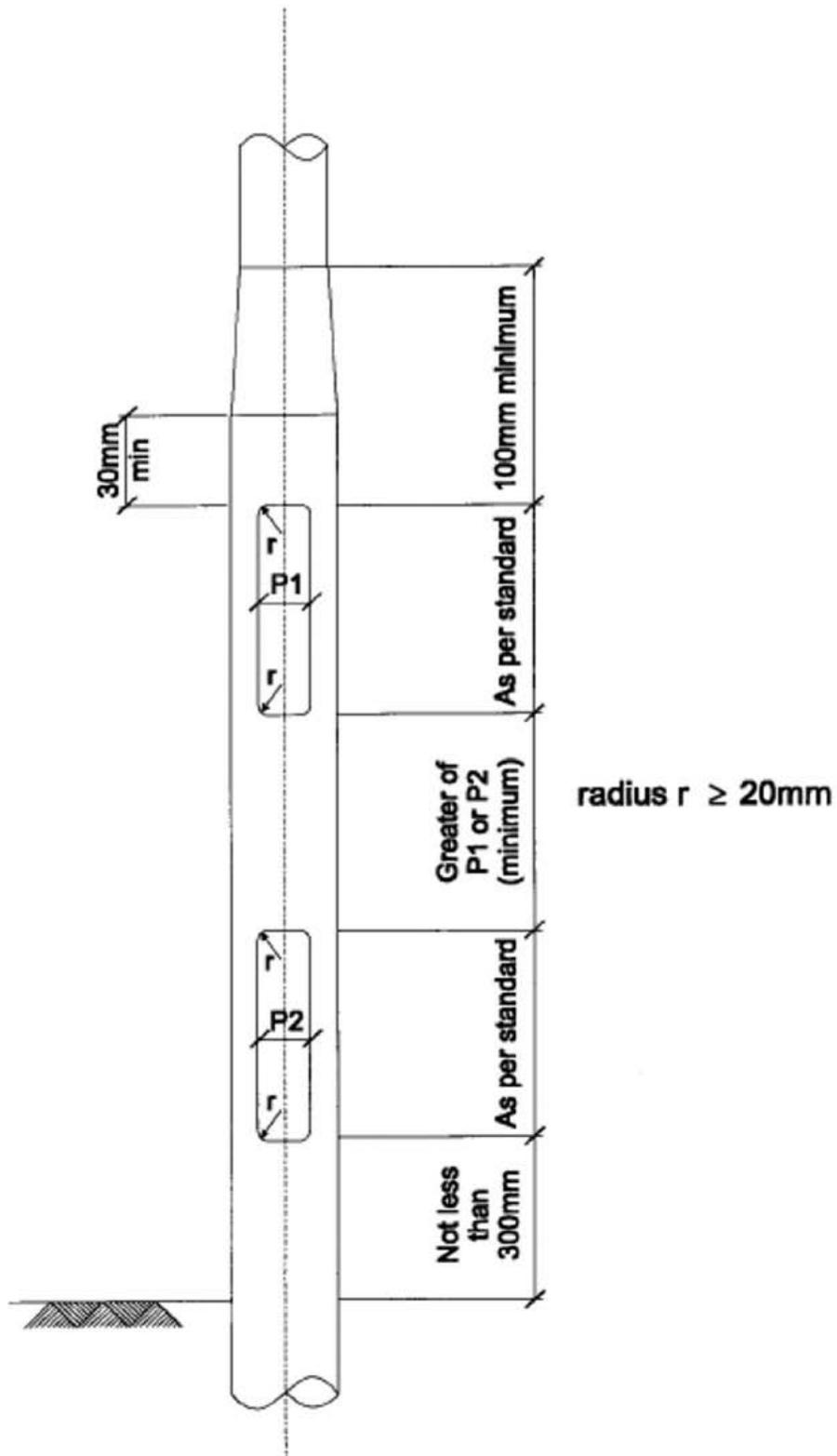


Figure 2: Door Openings

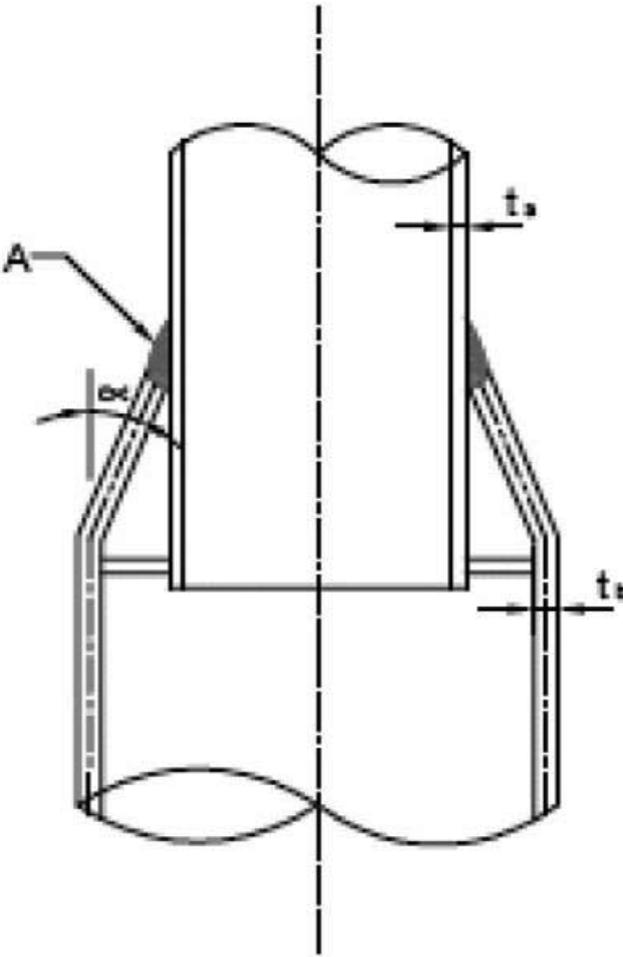


Figure 3: Typical Shoulder Joint

## 6. FIBRE REINFORCED POLYMER COMPOSITE LIGHTING COLUMNS

### Design

- 6.1 Loading. Design loads and moments shall be determined in accordance with IS EN 40-3-1 and IS EN 40-7 as implemented by this Standard.
- 6.2 The factor  $\beta$  for the dynamic behaviour of the FRPC column shall be determined by reference to IS EN 40-7: 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. The test results take precedence in all cases.

#### *Calculations*

- 6.4 Design calculations for FRPC columns shall be in accordance with the requirements IS EN 40-7.
- 6.5 The mechanical properties of the FRP material to be used in the structural design calculations shall be determined from tests using flat sheet samples manufactured in the same manner as that proposed for the production column. Flexural strength and the moduli in both longitudinal and transverse directions shall be determined together with the shear modulus and the Poisson's ratio,  $\delta_{12}$ . A statistical assessment shall be made of the results to determine 95% confidence limits of the values to be used.

#### *Use of Other Materials*

- 6.6 All other materials incorporated in the FRP columns shall comply with the Specification and the relevant parts of IS EN 40.

## 7. DOOR OPENINGS

- 7.1 Where door openings are required, the sizes given in Table 3 should be specified when providing information for Appendix 13/1 of the Specification.
- 7.2 Alternative door openings selected from the sizes given in IS EN 40-2 may be used, provided they are shown to be adequate for the size of equipment to be housed and maintained, in the column.
- 7.3 Columns mounted on structures or in situations where there is a risk that a detached door could cause an accident if it fell on the area below shall have their doors hinged or held captive by an approved metal chain or strap which shall be sufficiently robust, to support the door in severe gale conditions.

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 IS EN 40-3-3.

In all other cases the design strength shall be calculated from first principles. Alternatively, the design shall be based on the results of full-scale load tests. In all such cases the procedures to be used shall be agreed between the designer, the Employer's Representative and the National Roads Authority, see 4.2 above.

Nominal column height (h) in metres	Type of door	Door opening for metal columns (height x width) (mm)	Door openings for concrete columns (height x width) (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	-

**Table 3: Door Opening Sizes**

- 7.4 Doors shall be of a proven vandal resistant design.

## **8. WALL MOUNTED BRACKETS**

- 8.1 Wall mounted brackets shall be designed, in accordance with the relevant requirements for column brackets. The bracket shall be fixed to its support by means of a flange plate and anchorage which shall be designed in accordance with paragraph 10.9.
- 8.2 The wall on which the wall mounted brackets are fixed shall be capable of carrying the additional loads and other forces that may be transmitted by the bracket. The designer of the bracket shall provide the necessary loads for others to assess the adequacy of the wall.

## 9. ATTACHMENTS

### General Requirements

- 9.1 Support structures for roadside furniture, other than CCTV masts (see 9.6 to 9.8 below) and fixed traffic sign/signal posts (see IS EN 12899-1) shall be designed for the attachment given in paragraph 9.2. Attachments shall not be allowed on cantilever masts.
- 9.2 The attachment shall be taken as a sign, details of which shall be:
- The sign shall be taken as rectangular in elevation, with a surface area of 0.3 m<sup>2</sup>.
  - The eccentricity from the centre line of the column to the centre of area of the sign shall be taken as 300 mm.
  - The height above ground level at the column to the centre of area of the sign shall be taken as 2500 mm.
  - The orientation of the sign shall be selected to produce the most adverse effects for the design condition being considered.
- 9.3 The forces due to dead and wind loads on the sign and bracket projecting from the column shall be determined in accordance with
- IS EN 40-3-1. The shape coefficient of the sign shall be taken as 1.8 unless derived from
- IS EN 1991-1-4 for the specific shape and aspect ratio of the sign.
- 9.4 Where larger signs, waste paper containers, flower baskets etc., are to be attached, the column shall be designed to resist the additional loadings. Where appropriate the additional loadings shall be calculated in accordance with paragraph 9.3.
- 9.5 Support structures designed to carry attachments greater than those defined in 9.2 shall have identifying manufacturer's features or marks to enable them to be clearly and unambiguously identified throughout their service life. The unique identifying mark shall be the structure reference number as required by Eirspan Inventory. All other requirements for the identifying mark shall be as required in the Specification. (See 4.1)

### Attachments to CCTV Masts

- 9.6 CCTV masts shall not be designed for attachments other than CCTV cameras and their associated equipment unless otherwise specified. Where attachments are specified they shall be incorporated into the design of the CCTV masts in accordance with the following provisions:
- 9.7 Where attachments are to be used, the mast shall be designed to resist the additional loading, which shall be described in Appendix 13 of the Notes for Guidance on the NRA Specification for Road Works (MCDRW). Where appropriate the additional dead and wind loads shall be calculated in accordance with ILP PLG07.
- 9.8 Where attachments are required the CCTV pole and the attachments shall be designed such that the operation of the CCTV camera is not impeded. Similarly, access for installation, inspection or maintenance of an attachment shall not interfere with the operation of the CCTV camera. 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.

## 10. FLANGE PLATE CONNECTION BETWEEN STEEL STRUCTURE AND FOUNDATION

### General

- 10.1 Where foundations consist of reinforced concrete, the connection between the structure and its substructure shall be designed in accordance with clauses 10.2 to 10.24 as appropriate.
- 10.2 A structure with a flange plate shall be fixed to the foundation or bridge deck by an attachment system and anchorage which shall be capable of providing the required restraint. This will usually take the form of holding down bolts which connect with an anchorage. Anchorages of expanding type shall not be used. The attachment system shall allow the structure to be demounted, or, for lighting columns, be such that removal and replacement of damaged lighting columns may be readily achieved.

Note 1: The procedure given in 10.4 to 10.21 is based on the flange plate and its connections being designed to resist vehicle impact. For minor structures that satisfy the requirements of clauses 2.6(i), 2.6(ii) or 2.6(iii) design against vehicle impact is not required. In such cases the flange plate and its connections shall be designed for dead load and wind loads only. This shall be achieved by taking  $M_R$  in the formulae following as the bending moment at the base of the structure derived from the ultimate factored dead load (permanent actions) and wind loads (variable actions).

Note 2: Where cantilever masts are located within 4.5 metres of the edge of the carriageway (the carriageway is the part of the running surface which includes all traffic lanes, hard shoulders, hard strips and marker strips) or within the central reserve, the design of attachment systems and anchorages shall be such that removal and replacement of damaged cantilever masts may be readily achieved. This shall be achieved by providing an internally threaded component in the anchorage to receive the holding down bolts.

- 10.3 Typical arrangements are shown in Figure 5 which apply to both plates supported on bedding material and plates supported on levelling nuts only, without effective bedding.
- 10.4 When the weight of the structure 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. When the weight of the structure is supported directly through the flange plate to the substructure, the space should be packed with a suitable bedding mortar.
- 10.5 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 bolts.
- 10.6 In the following procedure it is assumed that bending about the v-v axis will be critical, which is the case for columns on square flange plates with four holding down bolts as shown in Figure 5. The more general case is covered in Annex C.
- 10.7 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 IS EN 40-3-3: Clause 5.6.2.

10.8 The bending moment in the flange plate shall thus be taken as:

$$M = 1.2M_R \left\{ 0.5 - \frac{0.63D\sqrt{2}}{4a} \right\} \text{ (in N.m)}$$

where  $D = 2R$  and  $R$  is the mean radius as defined in IS EN 40-3-3: Figure 3; and  $a$  is the bolt spacing as shown in Figure 5.

10.9 The maximum bending in the flange plate,  $M$ , shall not exceed the plastic moment capacity of the flange plate,  $M_p$ . For a square flange plate with a centrally located hole not exceeding  $0.3D$  in diameter (refer to Figure 5, detail B),  $M_p$  is given by:

$$M_p = \frac{(\sqrt{2}c - 0.63D) \times t_f^2}{4} \times \frac{f_y}{\gamma_{M0} \times 10^3} \text{ (in N.m)}$$

where:  $\gamma_{M0}$  is given in the Irish National Annex to IS EN 1993-1-1;  $c$  = the width of the flange plate (in mm);  $t_f$  = the thickness of the flange plate (in mm);  $f_y$  = the yield stress in the flange plate (in N/mm<sup>2</sup>); and  $D$  is as defined in 10.8. Where the centrally located hole and the column base are the same diameter (refer to Figure 5, Detail A),  $M_p$  shall be calculated in accordance with the procedure given in Annex C.

10.10 Shear and bearing should not govern the design of the flange plate, provided edge distances of the holding down bolts comply with the following requirements. 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 hole.

In addition, for slotted holes the minimum distance from the axis of the slotted hole to the adjacent edge of the plate shall not be less than  $1.5d$  and 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  $1.5d$ .

### Design of Welds

10.11 The 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 10.7 above.

10.12 Welds shall be deemed to meet these requirements provided the throat thickness of the top weld is not less than  $k \times t$  where:  $k$  = a value between 1.0 and 1.5 depending on the type of weld use. For example  $k = 1.5$  for the fillet welds of detail B in Figure 5 and for the outer fillet weld of detail A in Figure 5,  $k = 1.0$  for a full penetration butt weld.

$t$  = the wall thickness of the column at the flange plate.

A more accurate procedure for the design of welds is given in Annex C.

### Design of Holding Down Bolts

10.13 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 IS EN 40-3-3: Clause 5.6.2 and an equivalent ultimate shear force,  $F_R (=2M_R)$ .

10.14 The tensile stress ( $\sigma$ ) in holding down bolts may be taken as:

$$\sigma = \frac{1.2M_R \times 10^3}{\sqrt{2} a A_{et}} (N/mm^2)$$

where:  $A_{et}$  = the tensile stress areas in the thread of the bolt obtained from the appropriate standard;  $a$  = the bolt spacing as shown in Figure 5.

10.15 The shear stress ( $\tau$ ) in the bolts may be taken as:

$$\tau = \frac{1.2 M_R}{n_b A_{eq}} (N/mm^2)$$

where:  $A_{eq}$  = 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;  $n_b$  = total number of bolts fixing the flange plate. 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.

10.16 Bolts in tension and shear shall comply with:

$$\frac{F_{v,Ed}}{F_{v,Rd}} + \frac{F_{t,Ed}}{1.4F_{t,Rd}} \leq 1.0$$

Where:  $F_{v,Ed}$  is the design shear force per bolt for the ultimate limit state

$F_{v,Rd}$  is the design shear resistance per bolt

$F_{t,Ed}$  is the design tensile force per bolt for the ultimate limit state

$F_{t,Rd}$  is the design tension resistance per bolt

10.17 Due consideration of the capacity of the complete anchorage to resist the forces involved ( $1.5 M_R$  and  $1.5 F_R$ ) should also be made with regard to embedment and pull out based on a  $90^\circ$  cone recommended in "Holding down systems for steel stanchions" CS/BCSA/Constrado, 1980.

### Bearing Stresses Under Flange Plates

10.18 The bearing stress on the foundation medium should be derived on a basis compatible with the assumed bending mode v-v, on either a plastic or elastic basis as required. On a plastic basis, the maximum bearing stress for bending about v-v may be taken as:

$$\frac{3M_R \times 10^3}{0.7(0.7c - R)^2(a + 0.5c + 0.7R)} (N/mm^2)$$

where  $M_R$ ,  $c$ ,  $a$  and  $R$  are all as defined above.

10.19 The bearing stresses in any bedding mortar under the flange plates shall not exceed  $20 N/mm^2$ . The maximum bearing stresses on the concrete under a flange plate shall be in accordance with the requirements of IS EN 1992.

10.20 The requirements for foundations on masonry shall be agreed with the National Roads Authority.

- 10.21 For bases founded on steel bridge decks a more thorough analysis is required and is outside the scope of this standard.

### Design of Anchorages to Bolts

- 10.22 This is dependent on the medium in which the anchorages are made. The anchorages shall be designed to cater for a maximum tensile force,  $T_A$ , and associated shear,  $F_A$ , as follows:

$$T_A = 1.25 \sigma A_{et} \text{ (in N); and}$$

$$F_A = 1.25 \tau A_{eq} \text{ (in N).}$$

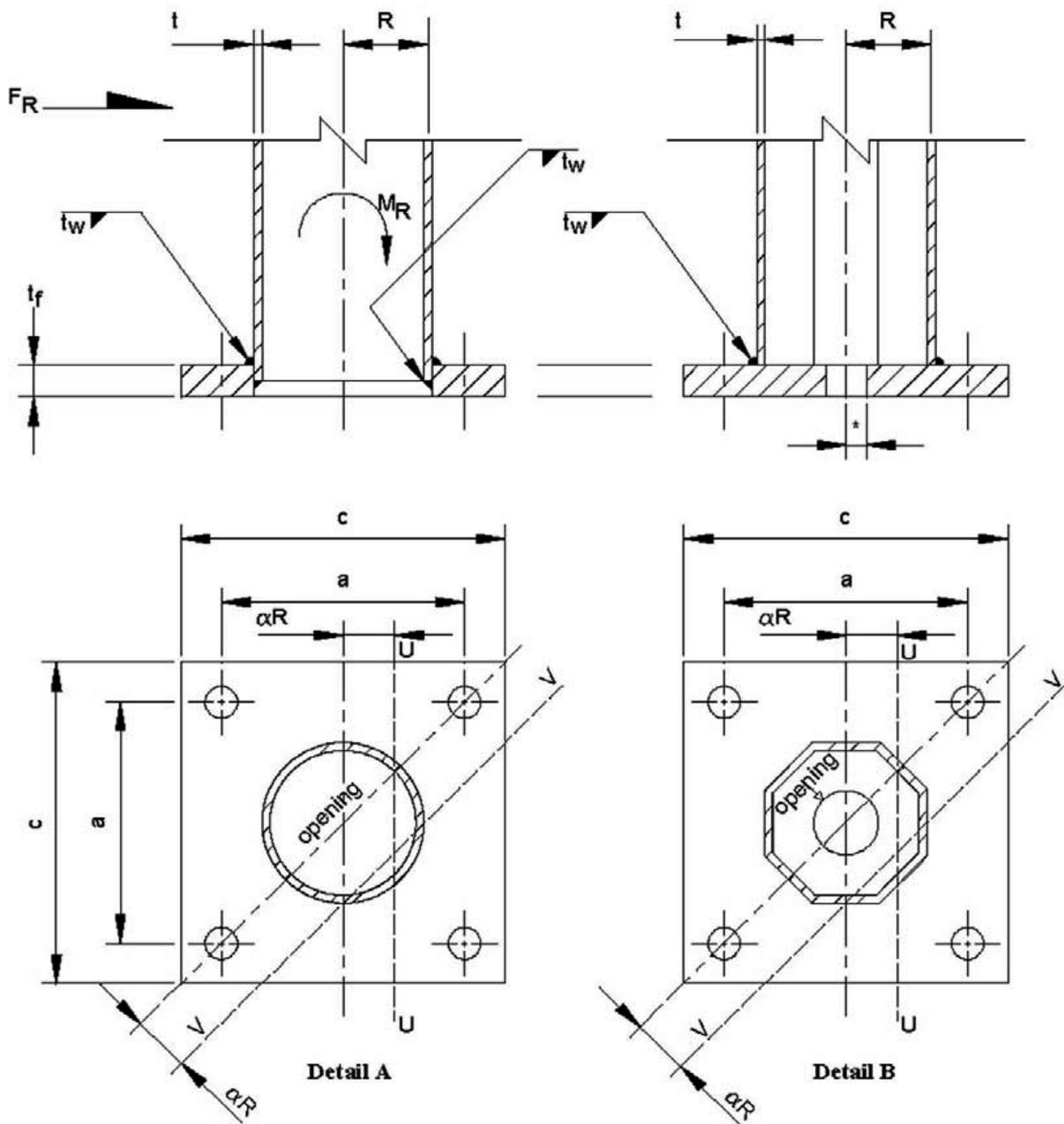
where  $\sigma$ ,  $\tau$ ,  $A_{et}$  and  $A_{eq}$  are all as derived above.

The capacity of the anchorage shall be derived in accordance with Section 11 (and relevant parts of IS EN 1997.)

- 10.23 The supporting structure shall be designed to resist the above anchorage loads without damage. The tensile strength of the concrete should be ignored in the calculations. 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

- 10.24 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. The nuts and washers on both sides of the flange plate thus need to be sufficiently oversized to prevent any localised plate failure due to concentration of stresses. This may be achieved by using washers complying with IS EN ISO 7093, provided the hole or width of the slotted hole does not exceed  $d_o + 4\text{mm}$  where  $d_o$  is the diameter of the holding down bolts.
- 10.25 For slotted holes, which provide flange plate rotations of up to  $\pm 5^\circ$  as shown in Figure 6, washers of adequate thickness shall be provided on both sides of the flange plate to transfer load into the holding down bolts. Washers complying with IS EN ISO 7093 may be used provided the width of the slotted holes does not exceed  $d_o + 6\text{mm}$ .
- 10.26 Where hole or slotted clearances are greater than the above values, consideration should be given to the use of special plate washers. Where levelling nuts are used the nut and washer size shall be the same above and below the flange plate.



**Figure 5: Typical Arrangement of Flange Plate**

- NOTE:
1. Details A and B are typical only and may be used with circular or octagonal columns if required.
  2.  $R$  = mean radius as defined in IS EN 40-3-3 (Figure 3).
  3.  $*$ : Radius of centrally located hole shall not exceed  $0.3R$

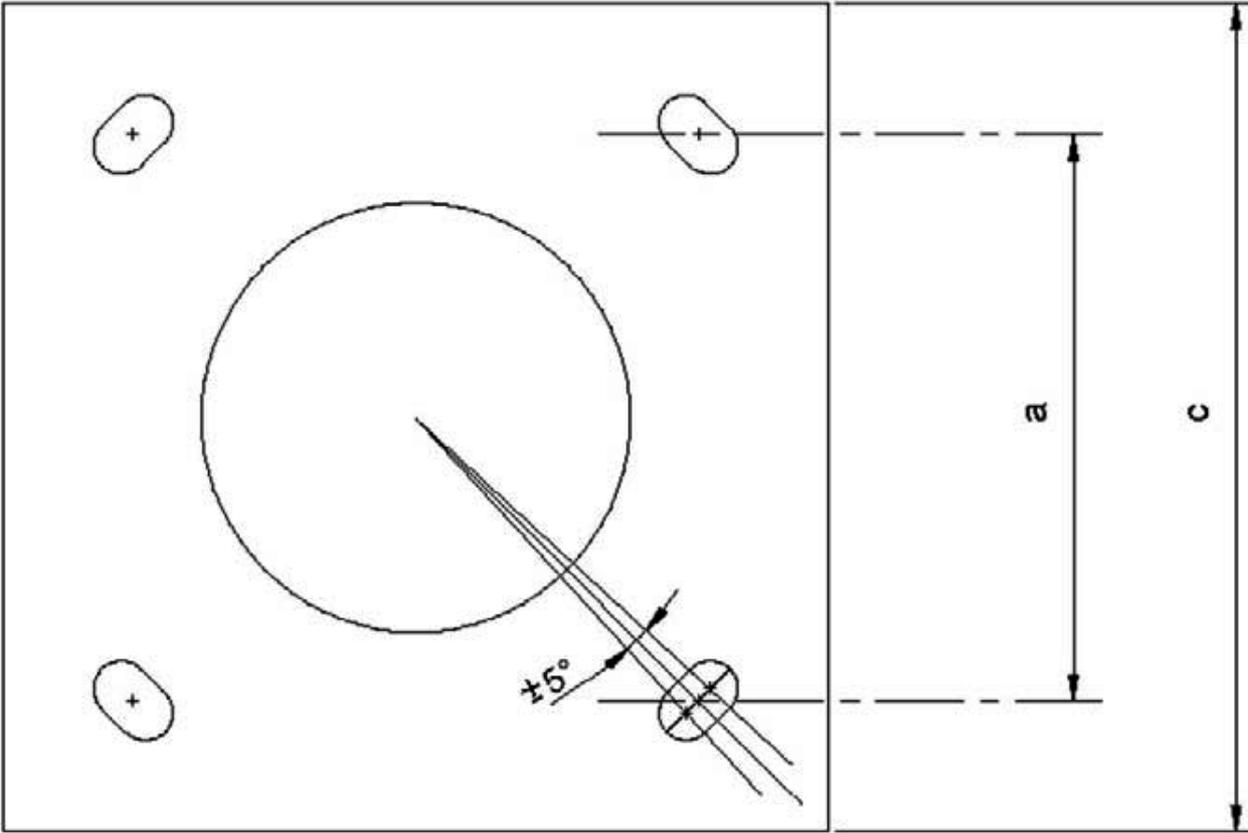


Figure 6: Slotted Holes Arrangement

# 11. FOUNDATIONS

## Foundations – General

11.1 Foundations shall either consist of:

- a) reinforced concrete, designed in accordance with paragraphs 11.10 to 11.16 as appropriate; or
- b) planted columns and posts, designed in accordance with paragraphs 11.3 to 11.9 as appropriate;
- c) planted prefabricated concrete or metal columns designed in accordance with 11.3 to 11.17 as appropriate.

Note: The design rules given in paragraphs 11.3 to 11.12 do not apply to foundations on slopes, where stability of the ground needs to be taken into account. In such instances, specialist geotechnical advice shall be sought.

Note: Planted columns shall not be used for CCTV masts.

11.2 Alternative forms of foundation may be used subject to the approval of the National Roads Authority.

### Foundations for Planted Columns, Posts and Prefabricated Foundations *Planting Depth:*

11.3 Where a support structure is to be planted directly in the ground, the planting depth shall be selected from Table 7 of IS EN 40-2 related to the overall height of the structure. In the case of prefabricated foundations the planting depth and effective diameter shall be selected to ensure compliance with the calculation method provided below.

Note: For traffic sign/signal posts, the values appropriate to the central column of this table may be used. Where the height is less than 2m a depth of 600mm may be adopted, provided the requirements of paragraph 11.6 are satisfied.

To check the adequacy of the selected planting depth, taking account of the ground conditions at the site, the calculation procedure given below shall be adopted.

11.4 The greatest destabilising moment, MDS, arising from application of the un-factored design loads (e.g. wind load or dynamic load from snow clearance) to the support structure and its supports should either be calculated or obtained from the designer. The destabilising moment shall be calculated about a fulcrum point located at  $1/\sqrt{2}$  of the planting depth below ground. The destabilising moment shall be multiplied by a model factor  $\gamma_{s;d}$  of 1.25.

11.5 The ground resistance moment  $M_g$ , should be calculated using the following formula:

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

Where:

G is a factor dependent on the ground in which the column is planted (in kN/m<sup>2</sup> per m). Refer to Table 3 for typical values of G.

D is the minimum diameter (or minimum distance across flats for multi-sided sections) of the traffic sign in the ground (in m).

P is the planting depth.

11.6 The planting depth is satisfactory if:  $M_g > \gamma_{s;d} \times M_{DS}$

11.7 If this criterion is not satisfied then the planting depth shall be increased and/or the effective diameter of the minor structure shall be increased. The latter can be achieved by backfilling the excavation hole with mass concrete or an acceptable fill material (refer to 'Backfilling' below); the effective diameter of the traffic sign/signal post may then be taken as the minimum diameter of the excavation hole.

Quality of soil	G (kN/m <sup>2</sup> per m)	Soil Impact Factor $k_{si}$
<b>Good:</b> 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
<b>Average:</b> 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
<b>Poor:</b> Soft clay, clay loam, poorly compacted sand, clays containing a large amount of silt and vegetable matter, and made-up ground.	230	0.5

**Table 4: Ground Factor G and soil impact factor  $k_{si}$**

Where the Quality is unknown, it shall be taken as Poor.

**Back-filling:**

11.8 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.

The following shall be specified to the installer:

- a) all back-filling material shall be placed in 150mm thick layers and well compacted; where the manufacturer proposes to use precast foundations, the backfilling material and procedure shall be described;
- b) during compaction, care shall be taken to ensure that the corrosion protection system for the minor structure is not damaged;
- c) where the excavated hole is back-filled with concrete, the concrete shall extend from the base of the minor structure to ground level; and
- d) 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.

- 11.9 Planted columns shall incorporate a mechanism which prevents rotation of the column or post in the ground under wind loading where significant torsional loading can arise. The design of planted columns shall take account of settlement and its effect on clearances if relevant.

### Foundation for Columns with Flange Plates

- 11.10 The design principles of foundations shall be based on the design methods given in IS EN 1997-1 and the associated National Annex. The foundation shall be designed to resist the foundation design moment  $M_{fd}$  and foundation design shear force  $F_{fd}$  derived as follows.

$M_{fd}$  shall be the greater of the impact moment  $M_i$  and the moment obtained from IS EN 40-3-1, IS EN 12899-1 or ILP PLG07 as appropriate, factored by the appropriate partial factor on load,  $\gamma_f$  (refer to IS EN 1997), for the failure mode under consideration.

$F_{fd}$  shall be the greater of the impact shear force  $F_i$  and the horizontal force obtained from IS EN 40-3-1, IS EN 12899-1 or ILP PLG 07 as appropriate, factored by the appropriate partial factor on load,  $\gamma_f$  (refer to IS EN 1997), for the failure mode under consideration.

For destabilizing actions (e.g. overturning moment)  $\gamma_{F;dst}$  shall be taken as at least 1.5. For stabilizing actions (e.g. gravity resistance to overturning)  $\gamma_{G;stb}$  shall be taken as 0.9 or less.

$M_i$  and  $F_i$  are derived as follows:

$$M_i = k_{si} M_R$$

$$F_i = K_{si} F_R$$

where the ultimate moment of resistance of the actual column at the base level,  $M_R$ , is calculated in accordance with IS EN 40-3-3: clause 5.6.2 together with an equivalent ultimate shear force,  $F_R$ . An upper bound to the equivalent ultimate shear force may be taken as:

$$F_R = \frac{M_{up}}{0.5} = 2M_{up} \text{ Refer to IS EN 40-3-3}$$

for the calculation of  $M_p$

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

The soil impact factor,  $k_{si}$  is given in Table 3 based on the three types of soil listed therein.

### Foundations for Cantilever Masts with Flange Plates

- 11.11 When cantilever masts are positioned in locations as given in paragraph 2.6 the following procedure may be used.
- 11.12 Foundations shall consist of reinforced concrete blocks. The structural concrete shall be designed in accordance with IS EN 1992.
- 11.13 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 Standard, factored by the appropriate partial factors on load,  $\gamma_F$  (refer to IS EN 1997).
- 11.14 The design of the foundation shall be based on the design methods given in IS EN 1997, using the partial factors on actions given in 11.10 above.

- 11.15 Because of the difference in the behaviour of the cantilever mast and its foundation, in the absence of more accurate information, the following may be assumed:

The basic wind load transferred from the cantilever mast to the substructure at the top of the substructure reduces to  $1/\beta$  of this value at the bottom of the substructure and foundation.  $\beta$  is the factor for dynamic behaviour given in IS EN 40-3-1 : Clause 3.2.4.

- 11.16 Unless otherwise agreed with the National Roads Authority, the criteria given in 11.10 shall apply when cantilever masts are positioned in locations other than those given in paragraph 2.6.

## 12. REFERENCES

### 12.1 National Standards Authority of Ireland

- a) IS EN 40: Lighting Columns:
  - i) Part 1: Definitions and terms
  - ii) Part 2: General requirements and dimensions Part 3-1: Design and verification – Specification for characteristic loads
  - iii) Part 3-2: Design and verification – Verification by testing
  - iv) Part 3-3: Design and verification – Verification by calculation
  - v) Part 4: Requirements for reinforced and prestressed concrete lighting columns
  - vi) Part 5: Requirements for steel lighting columns Part 6: Requirements for aluminium lighting columns
  - vii) Part 7: Requirements for fibre reinforced polymer composite lighting columns
- b) IS EN 1991-1-4: Actions on structures. Part 1.4 Wind Actions
- c) IS EN 1997-1 Geotechnical Design – General Rules
- d) IS EN 1992-1: Eurocode 2 Design of Concrete Structures
- e) IS EN 1994-1: Eurocode 5 Design of masonry structures
- f) IS EN 1993-1-1: Eurocode 3: Design of Steel Structures: Part 1.1: General Rules and Rules for Buildings
- g) IS EN 12899-1: Fixed, vertical road traffic signs – Part 1: Fixed signs
- h) IS EN ISO 7093 – Plain washers – Large series – Product grades A and C

### 12.2 British Standards Institute

- a) PD 6547: Guidance on the use of BS EN 40-3-1 and BS EN 40-3-3, BSI

### 12.3 National Roads Authority Design Manual for Roads and Bridges

- a) NRA BD 02 Technical Approval of Road Structure on Motorways and Other National Roads
- b) NRA BD 51 Design Criteria for Portal and Cantilever Sign / Signal Gantries

### 12.4 NRA Manual of Contract Documents for Roadworks. (MCDRW)

- a) Volume 1: NRA Specification for Road Works
- b) Volume 2: Notes for Guidance on the Specification for Road Works

### 12.5 Other Publications

- a) Holding down systems for steel stanchions – Published by The Concrete Society, The British Constructional Steelwork Association (BCSA) and Constructional Steel Research and Development Organisation (Constrado) – October 1980.
- b) The Institution of Lighting Professionals, Professional Lighting Guide, ‘PLG07 High Masts for Lighting and CCTV’.

## 13. ENQUIRIES

- 13.1 All technical enquiries or comments on this document or any of the documents listed as forming part of the NRA DMRB should be sent by e-mail to [infoDMRB@nra.ie](mailto:infoDMRB@nra.ie), addressed to the following:

“Head of Network Management, Engineering Standards & Research  
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Dublin 4”



.....  
Pat Maher  
Head of Network Management,  
Engineering Standards & Research

## APPENDIX A: LIMIT STATES FOR CANTILEVER MASTS

Limit State Description	Partial Factor on Load $\gamma_F$				
	Limit State Type	Fabricated Metal Dead Load	Superimposed Dead Load	Wind Load	Buffeting from High Vehicles
Strength (STR)	ULS	All Partial Load factors are given in IS EN 1990 and the associated Irish National Annex			-
Fatigue	ULS				1.00
Deflection	SLS				0.50

Table A1: Limit States and Partial Factors

Element and Position	Direction of Deformation	Limiting
Top of Post	Horizontal $\Delta x_1$ or $\Delta y$	1/100 of height of post
Tip of Cantilever	Horizontal $\Delta x_2$	1/100 of outreach plus height of post
Tip of Cantilever	Vertical $\Delta z$	1/100 of outreach plus height of post

Table A2: Limiting Structural Deformations of Cantilever Masts [See Figure 1]

## APPENDIX B: FATIGUE CHECKS OF STEEL STRUCTURES FOR WELD CLASSIFICATION

- B.1. When undertaking fatigue checks in accordance with the following rules, nominal stresses shall be used based on nominal section properties. The stress concentrations inherent in the make-up of a welded joint (arising, for example, from the general joint geometry and the weld shape) have been taken into account in the classification of the details.

Where indicated, however, the nominal stresses shall be multiplied by stress concentration factors, indicative values of which are provided in the relevant clause.

- B.2. For reinforcement at door openings the geometric constraints set out in 5.13 and B.8 shall be met, and stress ranges around door openings need not be calculated. However if these constraints are not met then the requirements of B.3 or B.4 shall be followed, as appropriate.
- B.3. For minor structures other than cantilever masts for traffic signals and/or speed cameras that project over the carriageway, only fatigue due to wind gust loading shall be considered and the requirements of B5 to B10 shall be satisfied.
- B.4. 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 the requirements of B11 to B13 shall be satisfied.

### Fatigue Due to Gust Wind Loading

- B.5. A check on fatigue at and adjacent to each welded section, including the ends of reinforcement at door openings where relevant, shall be undertaken using a stress range  $\sigma_r$ , given by:

$$\sigma_r = 0.25\sigma_s \left(1 - \frac{1}{\beta}\right) \left(\frac{C_{vs}}{C_{stat}}\right)$$

where:

$\sigma_s$  is the stress calculated at this position for the design forces and moments specified in Section 4 of IS EN 40-3-1;

$\beta$  is the dynamic response factor (Clause 3.2.4 of IS EN 40-3-1);

$C_{stat}$  is the average shape coefficient for the top half of the column as used for the static analysis and given in Figure 3 of IS EN 40-3-1;

$C_{vs}$  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;

$D$  is the distance across the flats.

- B.6. This stress range shall be less than that obtained from B.7, appropriate to the class of detail being considered and for a number of cycles  $N_1$  given by:

$$N_1 = 10^6 N_f L$$

where:

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

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

- B.7. For a design life of 25 years, the maximum allowable stress range is given in Figure B1.1(a) or Figure B1.1(b) appropriate to the class of detail under consideration and dependent on the frequency  $N_f$  (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 B1.1(a) and Figure B1.1(b) is by two numbers joined by a hyphen. The first number is the reference strength at  $2 \times 10^6$  cycles and the second is the  $m$  value which is a constant applicable to values of  $n_1$  up to  $5 \times 10^6$  cycles. This is the procedure adopted for defining fatigue strength in IS EN 1993-1-9. The basis of the curves in Figures B1.1(a) and B1.1(b) is given in B.8.

Note: For a design life of  $L$  years Figure B1.1 may be used by adopting an effective frequency  $N_{fe}$  as the horizontal scale given by:

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

- B.8. Fatigue is critically dependent on geometrical configurations and fabrication.

The following geometric and fabrication constraints on cross sections of steel lighting columns shall be satisfied, in order to use the classes of details as provided in B.9.

#### *Flange Plates*

- a) The column/flange plate weld 1A, 2/1 and 2/2 shown in Figures B.2, B.3 and B.4 shall have a throat size  $K$  times greater than the thickness of the adjacent shaft material, where  $K$  is given by:

Weld	K
1A	1.10
2/1	1.25
2/2	1.25*

\* Or use full penetration butt weld

- b) The thickness of the base material  $t_b$  shall be not less than the thickness of the adjacent shaft material,  $t_s$ .

### Shoulder Joints

- c) Welded Shoulder joints as shown in Figures B.5, B.6 and B.7 shall have an angle of inclination to the axis of the columns,  $\alpha$ , between the following limits:  
 $12^\circ < \alpha < 35^\circ$
- d) The shoulder joint weld A as shown in figures B5, B6 and B7 shall have a throat size 10% greater than the thickness of the adjacent shaft material,  $t_s$ .
- e) To ensure that weld detail 6 (see Figure B.8) behaves as intended the lapped length shall be at least 1.5 times the diameter of the lapped shaft. Each section shall be galvanised to avoid the risk of premature failure due to rusting.

### Door Openings

- f) Stiffened and unstiffened door openings shall comply with the constraints shown in Figure B.9. In addition the following fabrication constraints shall be met:
  - i) sharp irregularities at free edges due to the flame cutting process shall be ground out;
  - ii) no welding shall be closer than 10mm from the edge of the door unstiffened opening.

Longitudinal edge stiffeners shall be continuous over their full extent.

B.9. Guidance on classes of typical weld details incorporating stress concentration factors,  $K_f$ , which comply with the constraints of B.6 are given in Figures B.2 to B.9 for welds made using normal commercial practice, e.g. manual welds without NDT or other testing. 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 IS EN 1993-1-9.

B.10. Figures B1.1(a) and (b), 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  $\sigma_o$  = details category (50, ... 120 ...)
  - $m$  = slope of curve (3 for figure B1.1(a) and 4 for Figure B.1.(b))
  - $\sigma_R$  = stress range
- b) The number of cycles relate to the frequency by the equation in B.4:  
 $N = 10^6 N_f L$
- c) Thus for a design life of L of 50 years:  
 $N = 50 \times 10^6 N_f$

d) Thus, the relationship between  $\sigma_R$  and  $N_f$  (the plots of Figures B1.1(a) and B1.1 (b)) is:

$$2 \times 10^6 \left( \frac{\sigma_o}{\sigma_R} \right)^m = 50 \times 10^6 N_f$$

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

e.g. for class detail 120: 4

$$\sigma_o = 120$$

$$m = 4$$

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

$\sigma_R$	100	90	80	70	60	50
$N_f$	0.166	0.253	0.405	0.691	1.280	2.650

### Fatigue from High Vehicle Buffeting

B.11. The stress range  $\sigma_{r2i}$  in any part of the structure for fatigue due to high vehicle buffeting shall be calculated by applying:

- a pressure of  $P_d$  to the portion of the cantilever arm and any attachments above the carriageway vertically downwards; and
- 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.

The pressure  $P_d$  shall be calculated as:

$$P_d = 600h^{-0.25} - 400 \text{ (in N/m}^2\text{)}$$

Where h is either:

- The distance from the top of the high sided vehicle to the underside of any horizontal surface; or
- The distance from the top of the high sided vehicle to the centre of pressure of any vertical surface.

A typical high sided vehicle height of 4.2 metres shall be used. (see note 3, clause B.12). The formula for  $P_d$  applies for a value of up to  $h = 5\text{m}$ .

Applied loads shall be calculated as the product of the appropriate pressure and projected area. Partial load factor  $\gamma_{FL}$  shall be taken as 1.0.

## Fatigue Damage Assessment

B.12. Fatigue damage shall be assessed as follows:

- a) For fatigue due to gust wind loading

The number of cycles,  $n_1$ , shall be calculated from B.6

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

$$N_1 = 2 \times 10^6 \left( \frac{\sigma_o}{\sigma_{r1}} \right)^m$$

- b) For fatigue due to high vehicle buffeting:

The number of cycles for each lane in a carriageway,  $n_{2i}$ , shall be given by:

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

The corresponding number of cycles to failure,  $N_{2i}$ , is given by:

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

Where:

L is the design life of the structure (years)

$\sigma_o$  is the details category (50, ... 120 ...), (see paragraph 5.15 and, for relevant details, from B.8);

$F_i$  is the lane allocation factor (see Table B1); and

m is the slope of curve (see paragraph B.7).

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

**Table B1: Lane Allocation Factors**

- c) The fatigue effects from wind gust loading and high vehicle buffeting shall be combined and shall satisfy the following criterion:

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

where T is the number of lanes directly beneath the cantilever arm.

Notes:

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

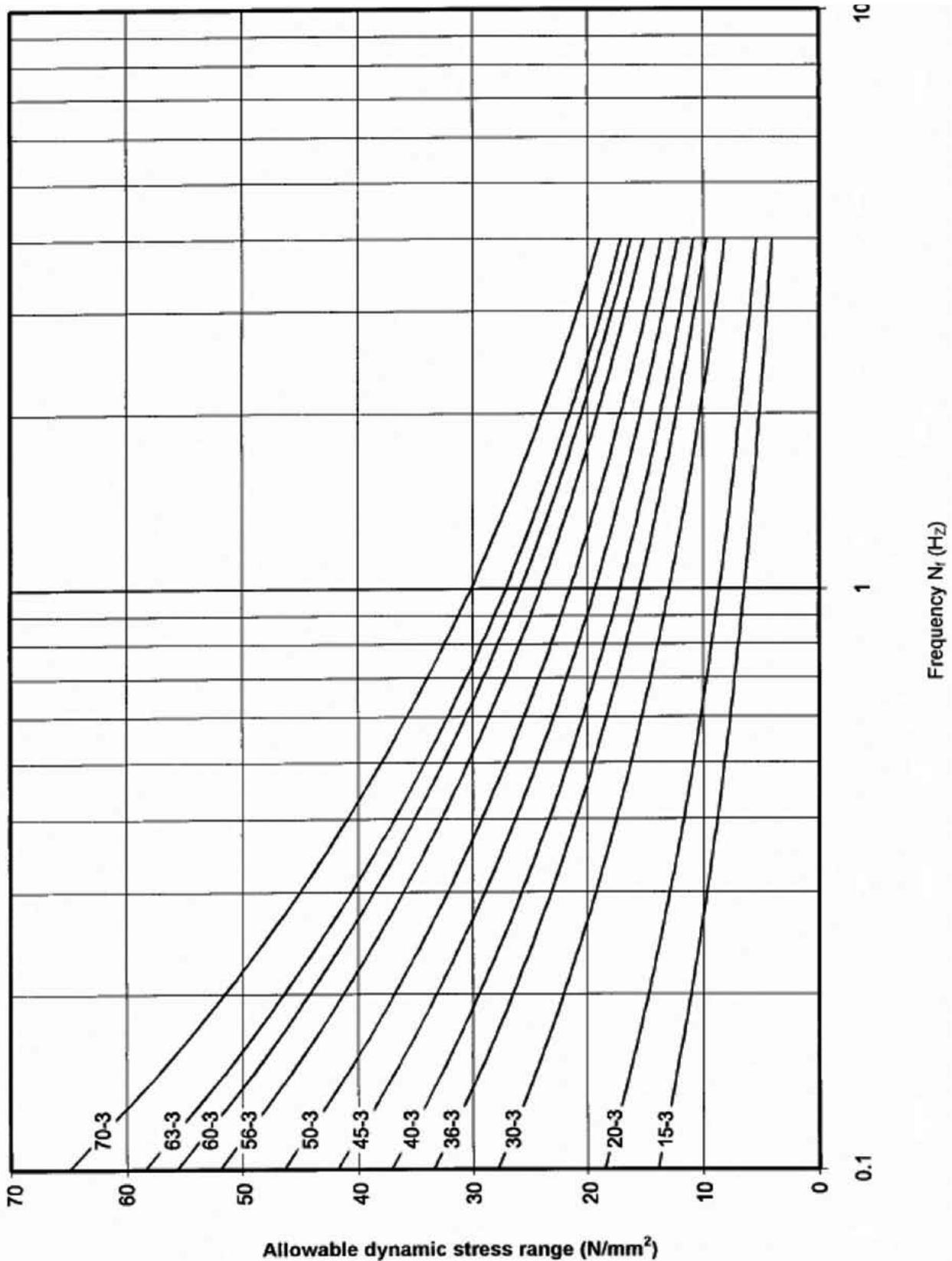
- a) The passage of 7,000 such vehicles per day on each carriageway. Where flows are less than this average, then the values of  $n_{2i}$  should be reduced in proportion. Flows of high sided vehicles shall be determined by traffic survey. The total number of high sided vehicles shall not be reduced below a value of 1,000 for design purposes.
- b) A total logarithmic decrement of damping of 0.03; where damping is less than this value then specialist advice should be sought.

Furthermore:

- c) 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 Irish roads; where a particular site has a significantly higher average vehicle height then this should be used instead.
- d) The design pressure,  $P_d$ , assumes that the maximum speed of the high sided vehicle is limited to 60mph. Where regulations permit higher maximum speeds then specialist advice should be sought.

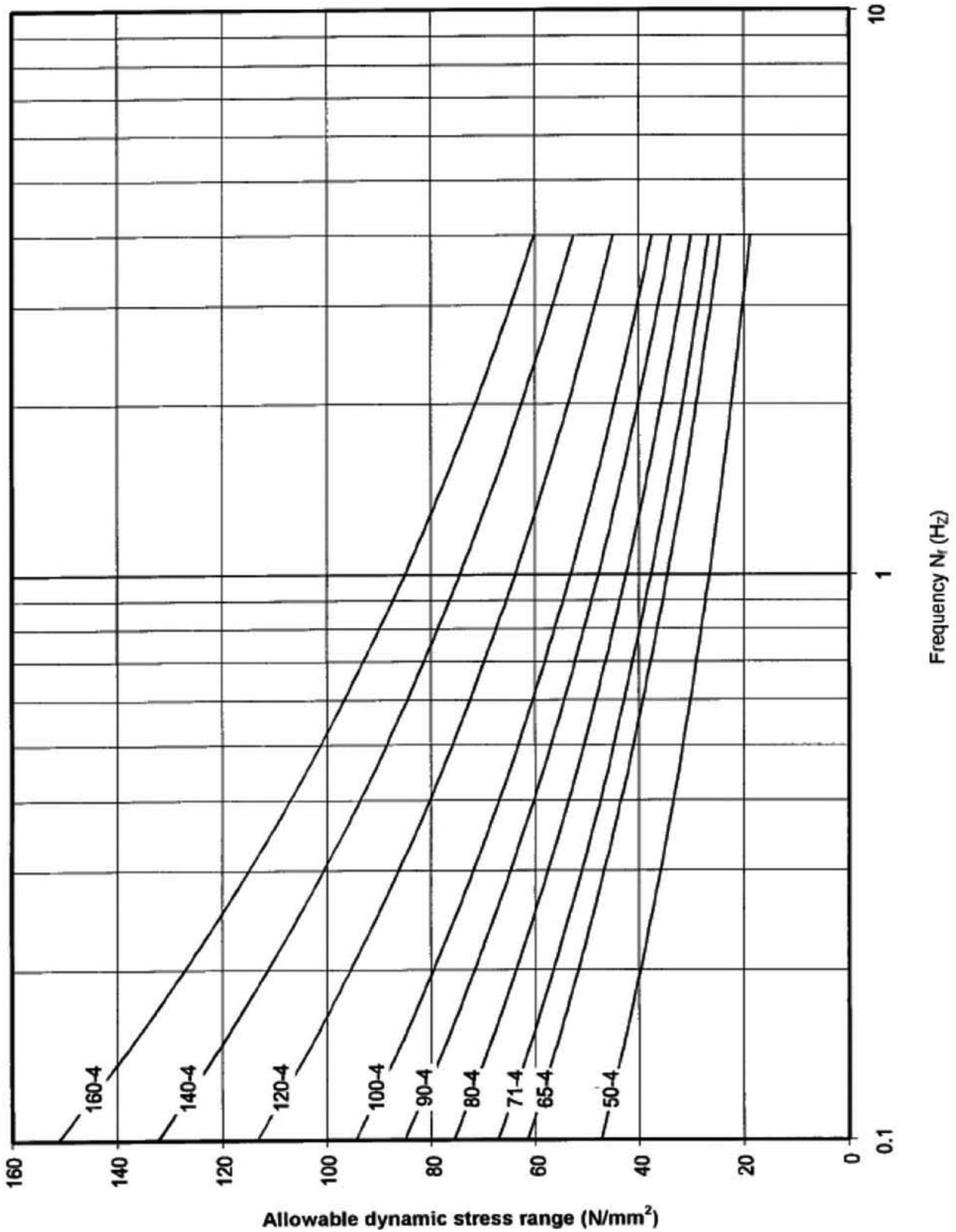
B.13. Checks on fatigue shall be undertaken at the following positions:

- a) at and adjacent to each welded section; and
- b) the end of the reinforcement at door openings.



Note: For basis of curves see B.9

**Figure B1.1(a): Fatigue of Column Stress Range Limit for Class of Weld Detail Based on a 25 Year Design Life Requirement (m=3)**



Note: For basis of curves see B.9

Figure B1.1(b): Fatigue of Column Stress Range Limit for Class of Weld Detail Based on a 25 Year Design Life Requirement ( $m=4$ )

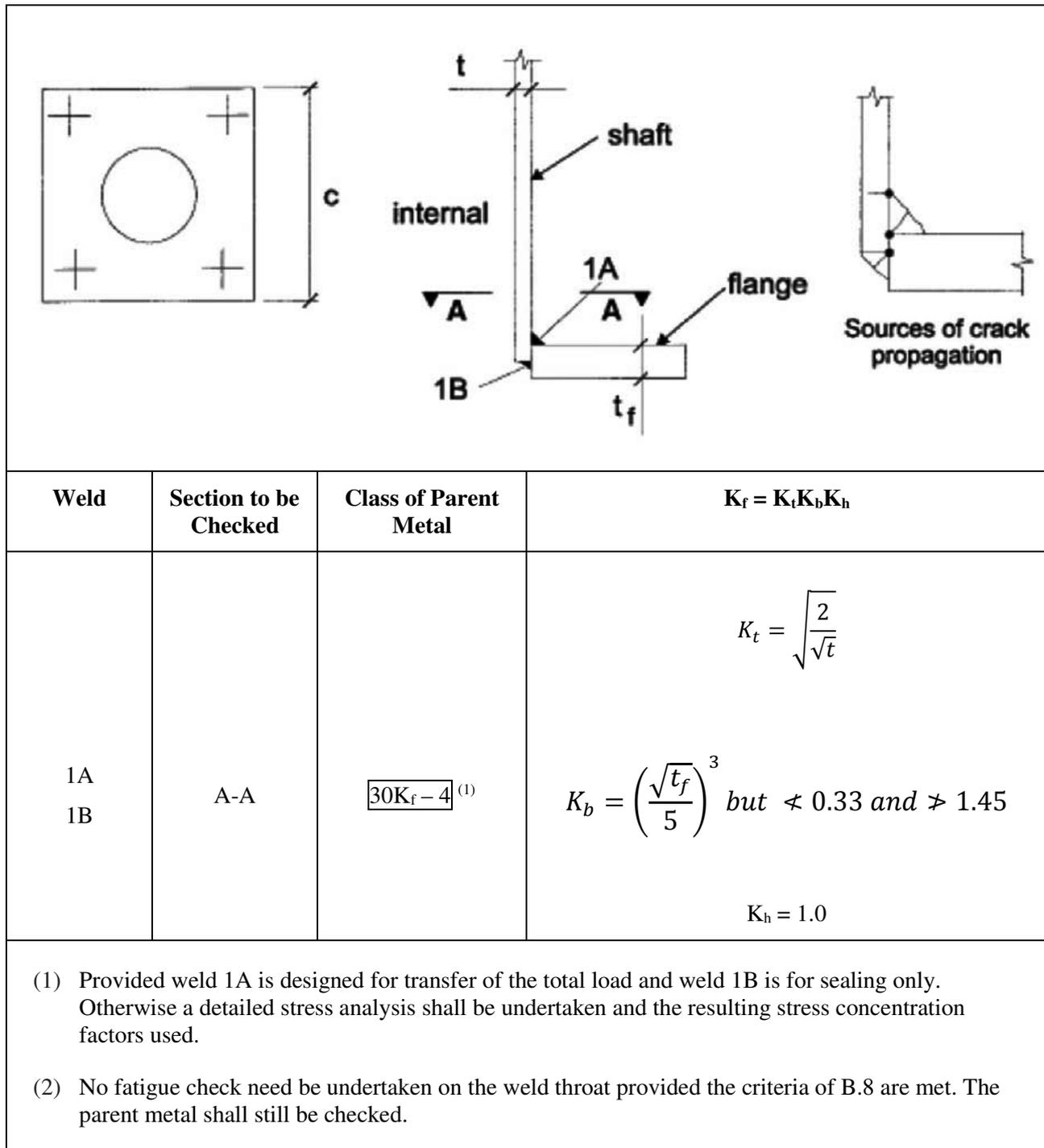
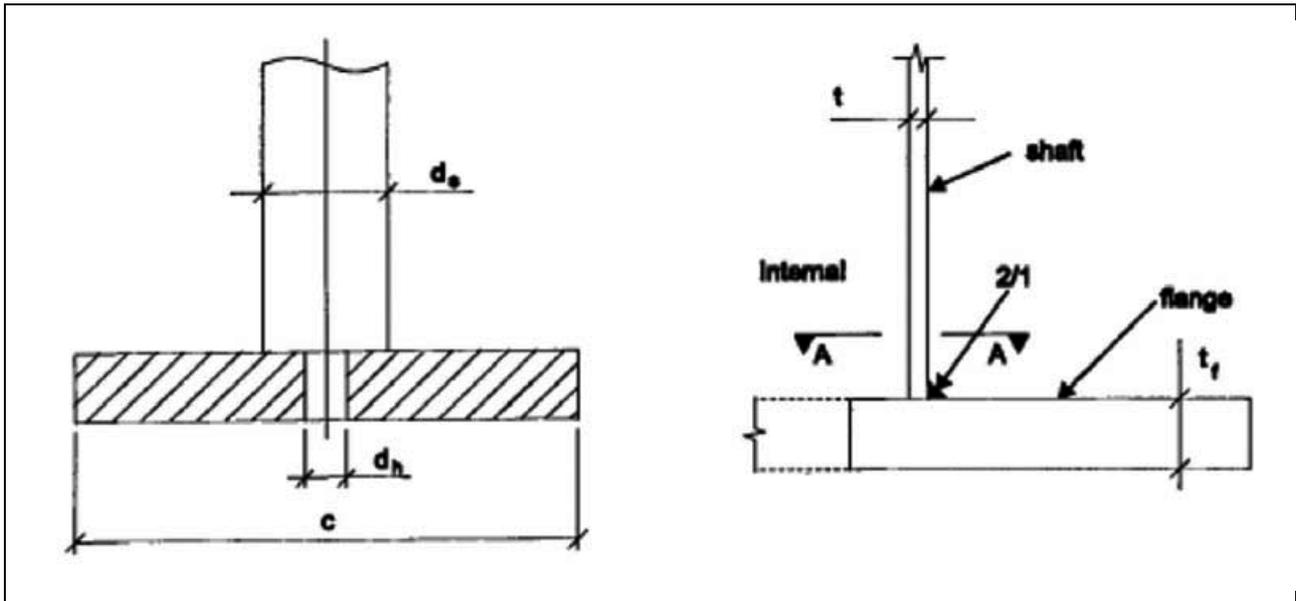


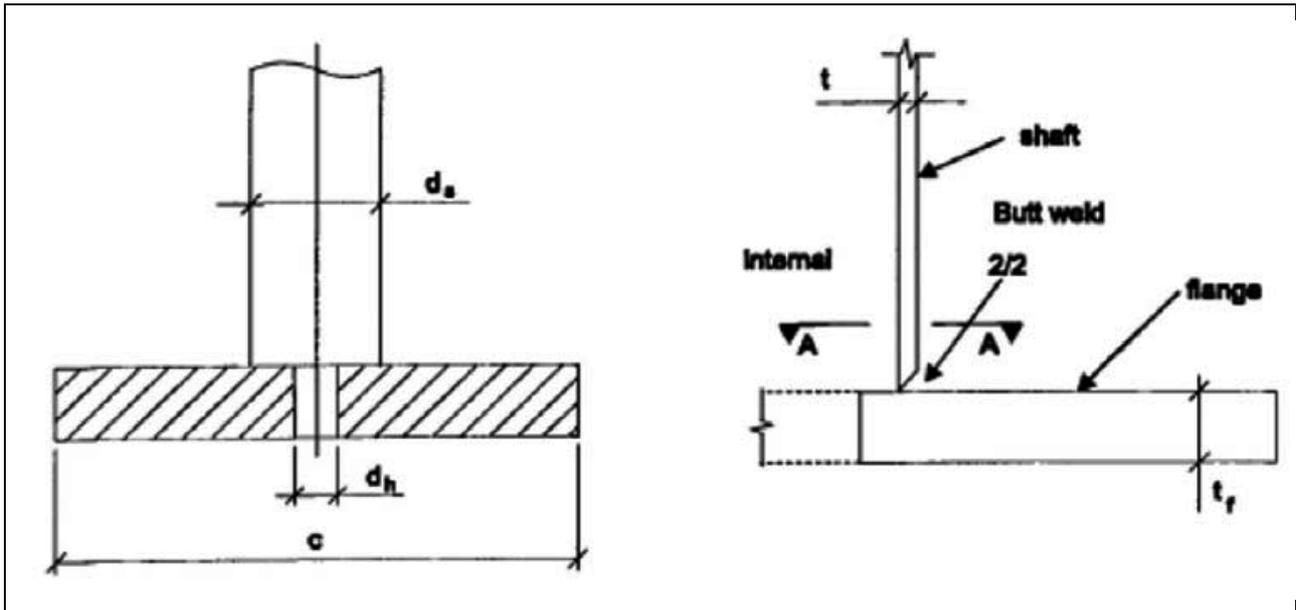
Figure B.2: Weld Detail Type 1



Weld	Section to be Checked	Class of Parent Metal	$K_f = K_t K_b K_h$
2/1	A-A	$30K_f - 4$ <sup>(1)</sup>	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_b = \left(\frac{\sqrt{t_f}}{5}\right)^3 \text{ but } \leq 0.33 \text{ and } \geq 1.45$ $K_h = 2/[1 + (d_h/(d_s - 2t))^3]$

Note: No fatigue check need be undertaken on the weld throat provided the criteria of B.8 are met. The parent metal shall still be checked.

Figure B.3: Weld Detail Type 2/1



Weld	Section to be Checked	Class of Parent Metal	$K_f = K_t K_b K_h$
2/2	A-A	$30K_f - 4$ <sup>(1)</sup>	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_b = \left(\frac{\sqrt{t_f}}{5}\right)^3 \text{ but } \nless 0.33 \text{ and } \ngtr 1.45$ $K_h = 2/[1 + (d_h/(d_s - 2t))^3]$

Note: No fatigue check need be undertaken on the weld throat provided the criteria of B.8 are met. The parent metal shall still be checked.

Figure B.4: Weld Detail Type 2/2

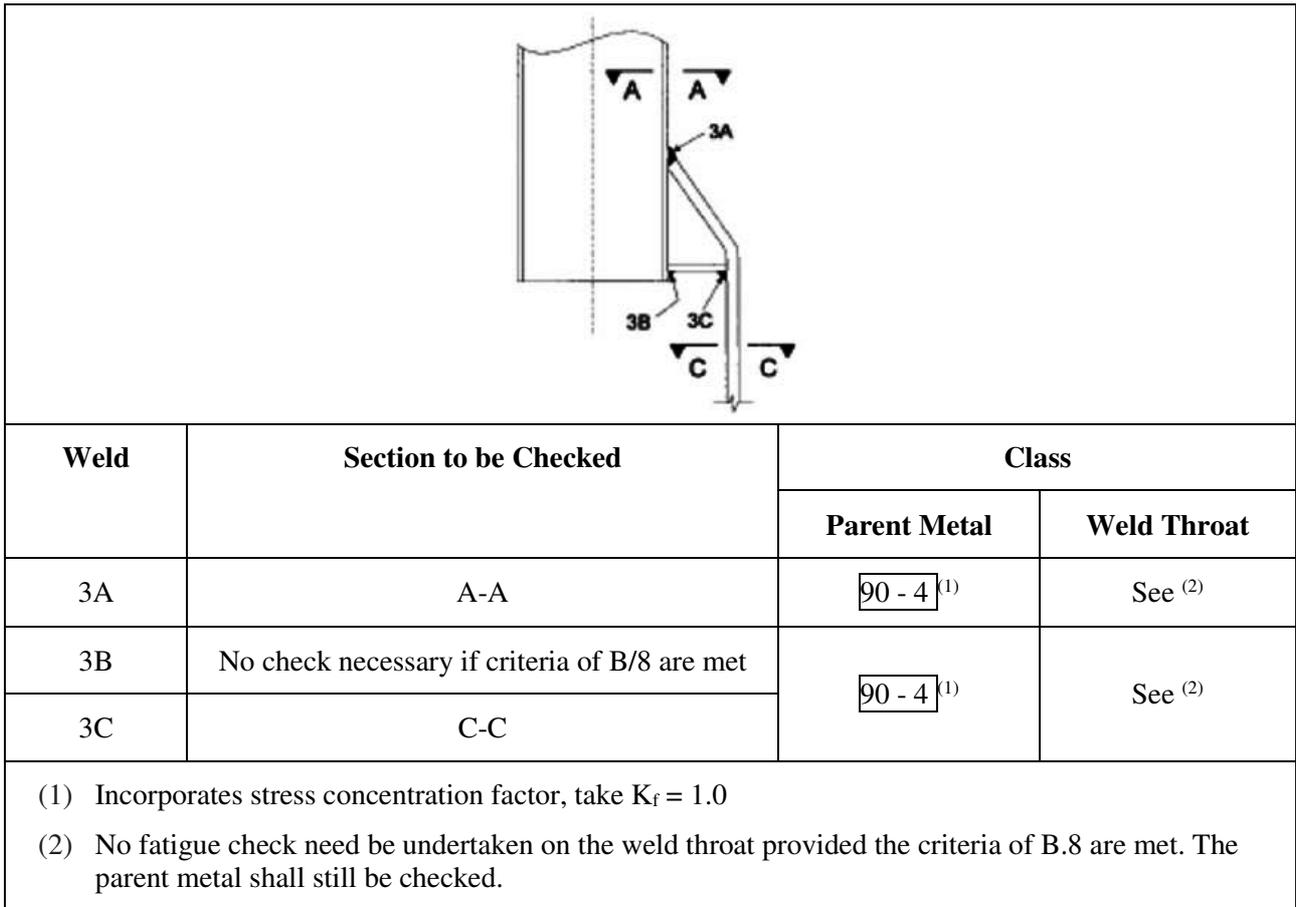


Figure B.5: Weld Detail Type 3

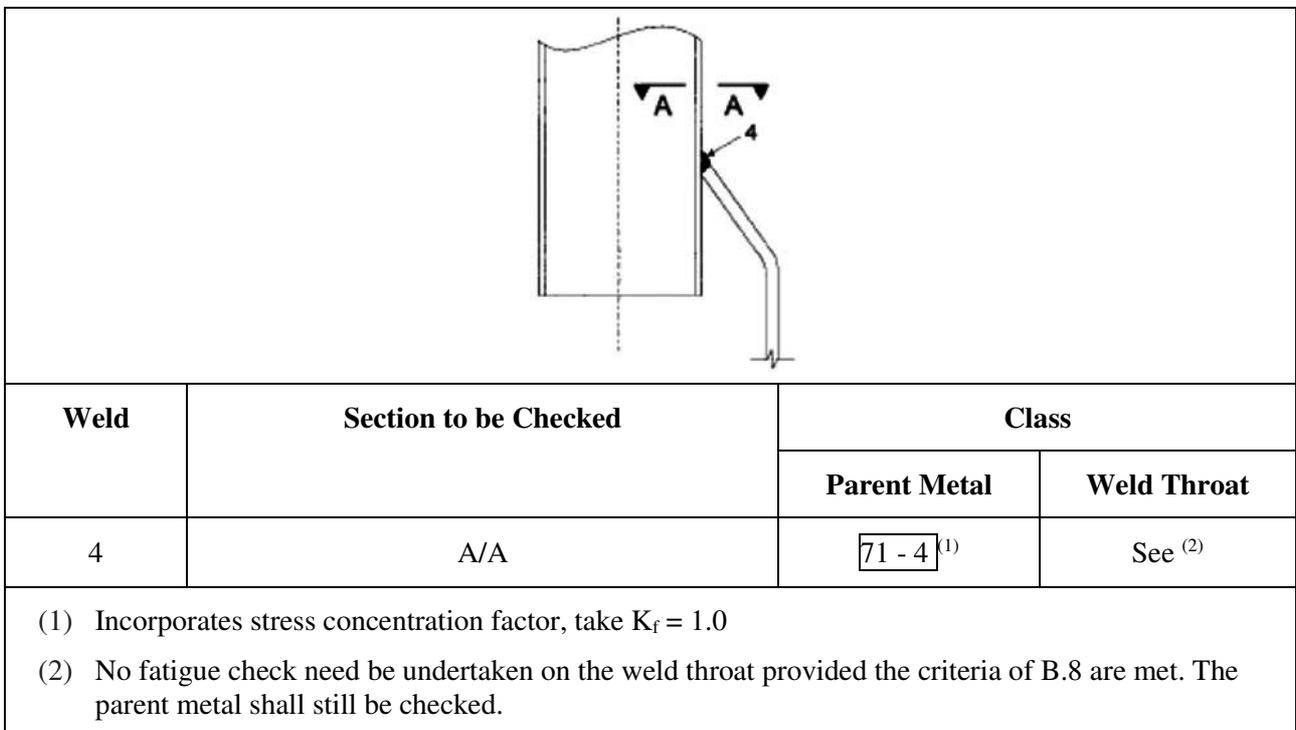
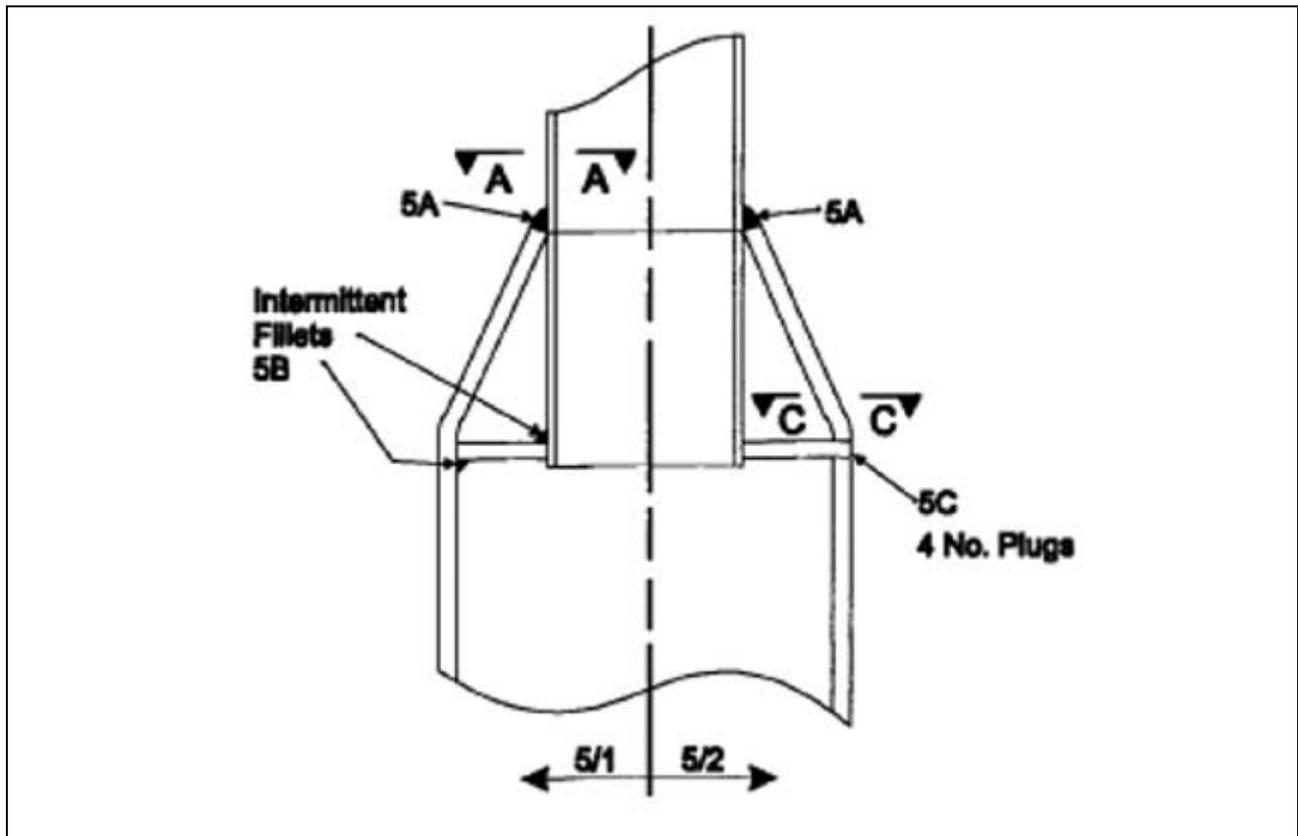


Figure B.6: Weld Detail Type 4

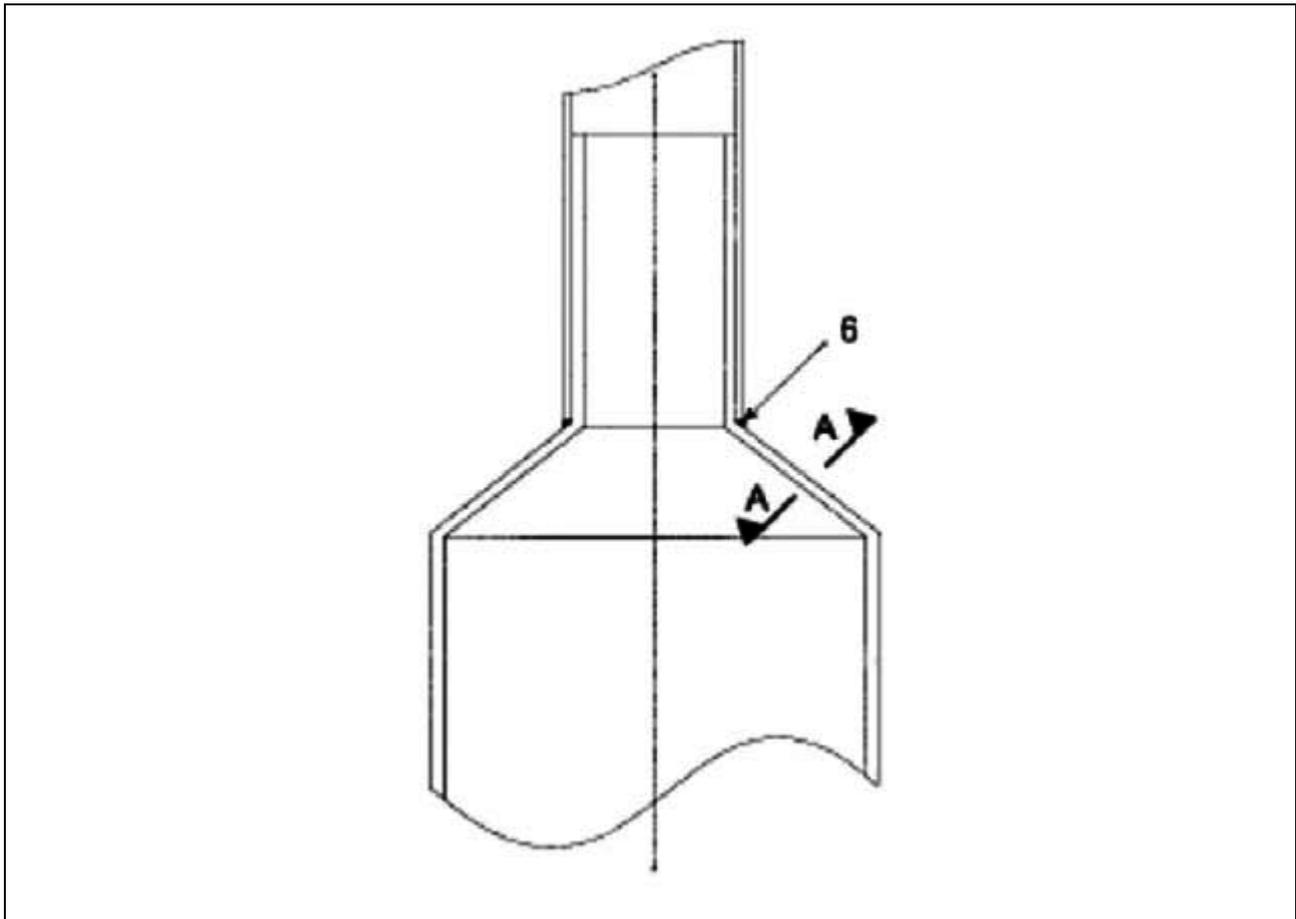


Weld	Section to be Checked	Class	
		Parent Metal	Weld Throat
5A	A-A	90 - 4 <sup>(1)</sup>	See <sup>(2)</sup>
5B	No check necessary if criteria of B/8 are met	120 - 4	See <sup>(2)</sup>
5C	C-C. Plugs not ground smooth	90 - 4	-
	C-C. Plugs ground smooth	120 - 4	-

(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 B.8 are met. The parent metal shall still be checked.

**Figure B.7: Weld Detail Type 5**



Weld	Section to be Checked	Class		
		Upper Tube Parent Metal	Lower Tube Parent Metal	Weld Throat
6	A-A	N/A <sup>(1)</sup>	71 = 4	See <sup>(2)</sup>

(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 shall still be checked.  
 (3) Refer to B.8(e) regarding the detailing of this joint.

Figure B.8: Weld Detail Type 6

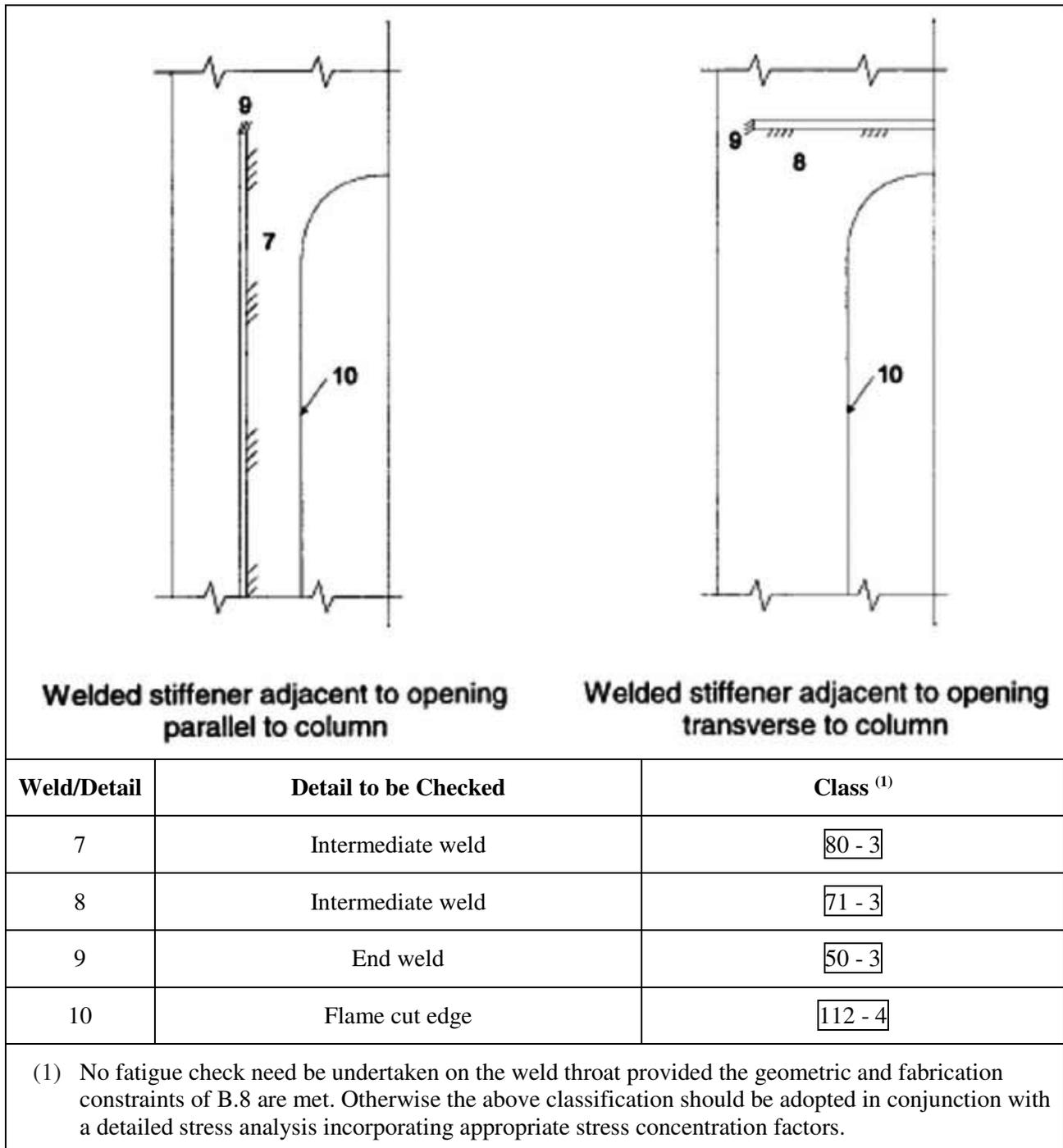


Figure B.9: Weld Detail Types 7 to 10

## APPENDIX C: DETAILED DESIGN OF FLANGE PLATES

### C.1. General

- C.1.1 The procedure given in Chapter 10 for the design of flange plates assumes circular or octagonal columns connected to square flange plates with a centrally located hole not exceeding 0.30D in diameter and supported by four holding down bolts symmetrically disposed. The following general procedure may be used for square plates with centrally located holes either not exceeding 0.30D in diameter, or of diameter equal to that of the column (see Figure C.1). This procedure provides design criteria for the welds, the plate, the holding down bolts and the bearing stresses.
- C.1.2 In addition a conservative assumption has been made for the position of the axis of bending. The procedure given herein provides a more accurate derivation of the maximum bending moment on the plate to be used in design.
- C.1.3 For flange plates not complying with the constraints of C.1.1 other suitable design methods, or full scale load tests may be adopted, subject to the approval of the National Roads Authority.

### C.2. Derivation of Weld Stresses

- C.2.1 The connection between the column and flange plate shall be capable of developing the ultimate moment of resistance,  $M_R$ , as derived from IS EN 40-3-3 and the equivalent shear force,  $F_R$ . The connection may be achieved by welds of leg length,  $t_w$  as shown in Figure 5, detail A or B.

NOTE: In the case of 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. Alternatively, a full penetration butt weld may be used which will automatically satisfy these requirements.

- C.2.2 The stress in the fillet welds due to moment of resistance  $M_R$  may be taken as:

$$\tau_1 = \frac{M_R 10^3}{\pi R^2 (0.7 t_w)} \text{ (in } N/mm^2 \text{)}$$

The shear stress in the fillet welds due to the equivalent shear force  $F_R$  may be taken as:

$$\tau_2 = \frac{F_R}{2\pi R (0.7 t_w)} = \frac{M_R}{\pi R (0.7 t_w)} \text{ (in } N/mm^2 \text{)}$$

where  $R$  = means radius of cross section (in mm);

$t_w$  = fillet weld leg length (in mm)

### C.3. Capacity of Welds

- C.3.1 The design resistance of the fillet welds shall be in accordance with IS EN 1993-1-8 and the associated Irish National Annex.

## C.4. Design of Flange Plates

### C.4.1 Derivation of Bending Moments in Flange Plates

C.4.1.1 The flange plate shall be designed to resist at least the effect of  $1.2 M_R$  at the base of the column where  $M_R$  is as derived from IS EN 40-3-3, and shall be checked about bending parallel to one side (axis u-u) and on the diagonal (axis v-v) see Figure C.1.

C.4.1.2 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 may be taken as:

$$M_{u-u} = 0.6M_R \left[ 1 - \alpha \frac{2R}{a} \right] \text{ (in N.m)}$$

$$M_{v-v} = 0.6M_R \left[ 1 - \alpha \frac{2R}{a\sqrt{2}} \right] \text{ (in N.m)}$$

where  $R$  = mean radius of the column cross section (in mm);

$a$  = spacing of the bolts (in mm);

and  $\alpha$  relates to the position considered for maximum bending in the plate. In lieu of more thorough analysis  $\alpha$  may be based on the centroid of the welds on the tensile side, i.e.  $\alpha$  may be taken as 0.63.

### C.4.2 Bending Capacity of Flange Plate

C.4.2.1 The maximum moment in the flange plate,  $M$ , shall not exceed the plastic moment capacity of the flange plate,  $M_P$ . For a square flange plate where the centrally located hole is the same diameter as the column base (refer to Figure C.1, detail A)  $M_P$  is given by:

$$M_p = \left( c\sqrt{2} - 2R(\alpha + \sqrt{1 - \alpha^2}) \right) \frac{t_f^2}{4} \frac{f_f}{\gamma_m 10^3} \text{ (in N.m) for u - u axis;}$$

and

$$M_p = \left( c\sqrt{2} - 2R(\alpha + \sqrt{1 - \alpha^2}) \right) \frac{t_f^2}{4} \frac{f_f}{\gamma_m 10^3} \text{ (in N.m) for v - v axis;}$$

where  $\gamma_m$  is taken as 1.15;

$c$  = the width of the flange plate (in mm);

$t_f$  = the thickness of the flange plate (in mm);

$f_f$  = the yield stress of the flange plate (in N/mm<sup>2</sup>).

## C.5. Design of Holding Down Bolts

### C.5.1 Derivation of Stresses in Bolts

C.5.1.1 The tensile stress in the holding down bolts may be taken as:

$$\sigma = \frac{1.2M_R \times 10^3}{n_t a A_{et}} \text{ (N/mm}^2\text{)}$$

where  $n_t$  is related to the number of bolts resisting tension and the assumed axis of bending and may be taken as:

$0.5n_b$  for bending about axis u-u; see Figure C.1; or

$0.25n_b$  for bending about axis v-v; see Figure C.1;

$A_{et}$  = the tensile stress area in the thread of the bolt obtained from the appropriate standard;

$a$  = the bolt spacing;

$n_b$  = total number of bolts fixing the flange plate.

NOTE: In general  $(n_t \times a)$  should 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 in 5 above.

The shear stress in the bolts may be taken to be that derived in 10.14, combined shear and tension in 10.15 and capacity of the anchorage from 10.16

### C.6. Check on Bearing Stress Below the Flange Plate

C.6.1 The bearing stress given in 10.17 assumes bending about the v-v axis. In general it will be necessary to derive the bearing stress on the foundation medium for both the assumed bending modes u-u and vv, on either a plastic or elastic basis as required. The maximum calculated bearing stress shall not exceed the value determined in accordance with 10.18.

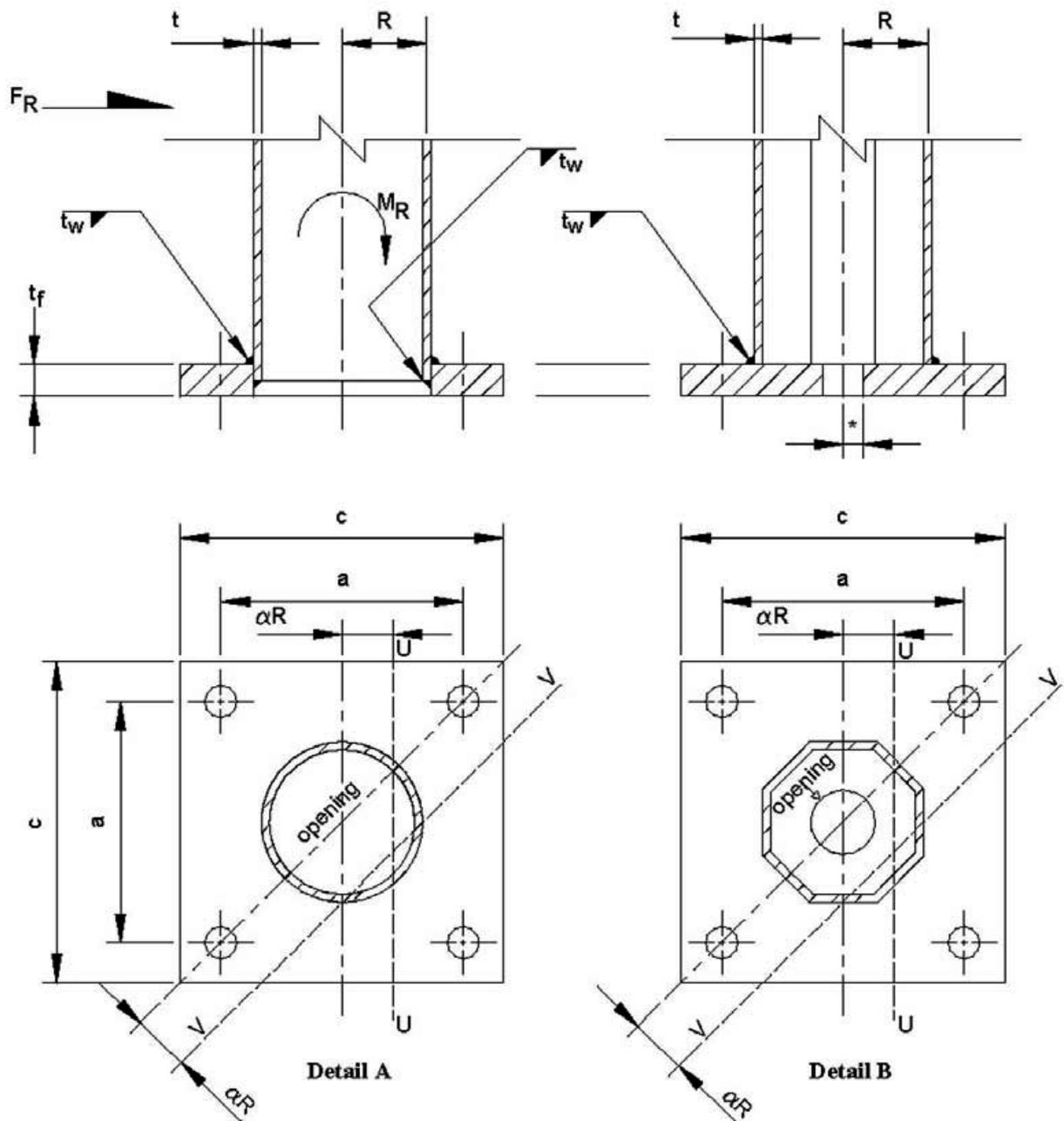
C.6.2 On a plastic basis, the maximum bearing stress for bending about u-u may be taken as:

$$\frac{3M_R \times 10^3}{c(0.5c - \alpha R)(0.75a + 0.5\alpha + 0.5c)} (N/mm^2)$$

C.6.3 On a plastic basis, the maximum bearing stress for bending about v-v may be taken as:

$$\frac{3M_R \times 10^3}{c(0.7c - \alpha R)^2(0.5c + 0.7\alpha R)} (N/mm^2)$$

where  $M_R$ ,  $c$ ,  $a$ ,  $R$  and  $\alpha$  are all as defined in C.4.1 above.



**Figure C.1: Typical Arrangement of Flange Plate**

- NOTE:
1. Details A and B are typical only and may be used with circular or octagonal columns if required.
  2.  $R$  = mean radius as defined in IS EN 40-3-3 (Figure 3).
  3. '\*': Radius of centrally located hole shall not exceed  $0.3R$ .

## APPENDIX D: DETERMINATION OF SHAPE COEFFICIENTS BY TESTING

### D.1. Shape Coefficients for Columns

#### General

- D.1.1 Properly conducted wind tunnel tests on columns and brackets shall only be undertaken when shape coefficients are not available from IS EN 40-3-1 or from recognised International Standards. Adoption of values from these standards or from wind tunnel tests shall be agreed with the National Roads Authority. Particular care should be taken to ensure that the values of shape coefficients relate to cross-sections of members of infinite length.
- D.1.2 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 shall be measured in the direction of the air flow and the direction normal to the air flow.
- D.1.3 Previous wind tunnel tests have indicated that small angular rotations of specimens can cause considerable differences in shape coefficients. The specimens shall therefore be turned in the wind tunnel and measurements taken at angular increments. In the region of each shape coefficient the measurements shall be reduced to approximately  $1^\circ$  of rotation. Comparisons shall be made with the values of similar sections given in recognized International Standards as part of the adoption and agreement procedure with the National Roads Authority set out in 4.

### D.2. Shape Coefficients for Lanterns, Cameras, Signs and Brackets

- D.2.1 The shape and lift coefficients for lanterns, cameras and signs may be determined from wind tunnel tests as required by IS EN 40-3-1. These tests shall be carried out on a full scale shape of the element in a tunnel sufficiently large to reduce side effects to an insignificant level. The surface condition of the specimen shall accurately represent that of the production version. Where optional attachments will be made to the element, e.g. photo-electric control units, gear component extensions etc., these shall be included in the test specimen.
- D.2.2 When carrying out wind tunnel test, forces both in the direction of the air flow and in the direction normal to the air flow shall be measured, as shape and lift coefficients are required for all the directions required in D.2.3. All shape coefficients shall be based on the projected area of the element normal to the air flow.
- D.2.3 Forces on an element shall be measured at increments of rotation of approximately  $1^\circ$  between the limit of  $\pm 10^\circ$  to the horizontal. IS EN 40-3-1 requires the maximum value between  $\pm 5^\circ$  to the horizontal but a more conservative value shall be adopted where large increases of coefficients are obtained between  $5^\circ$  and  $10^\circ$  to the horizontal. During testing the effects of small plan rotations about the point of fixing shall also be taken into account. Where an increase in shape coefficient obtained with a rotation within the limits of  $\pm 10^\circ$  then this value shall be adopted.





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