VOLUME 2 HIGHWAY STRUCTURES: DESIGN (SUB-STRUCTURES AND SPECIAL STRUCTURES), MATERIALS

SECTION 2 SPECIAL STRUCTURES

PART 1

BD 26/04

DESIGN OF LIGHTING COLUMNS

SUMMARY

This Standard has been updated to incorporate the provisions of BS EN 40 and supersedes BD 26/99.

INSTRUCTIONS FOR USE

This is a revised document to be incorporated into the Manual.

1. Remove existing contents page for Volume 2 and insert new contents page for Volume 2 dated November 2004.

2. Remove BD 26/99 from Volume 2, Section 2 and archive as necessary.

3. Insert BD 26/04 into Volume 2, Section 2, Part 1.

4. Archive this sheet as appropriate.

Note: A quarterly index with a full set of Volume Contents Pages is available separately from The Stationery Office Ltd.

THE HIGHWAYS AGENCY

SCOTTISH EXECUTIVE

WELSH ASSEMBLY GOVERNMENT
LLYWODRAETH CYNULLIAD CYMRU

THE DEPARTMENT FOR REGIONAL DEVELOPMENT
NORTHERN IRELAND

Design of Lighting Columns

Summary: This Standard has been updated to incorporate the provisions for BS EN 40 and supersedes BD 26/99.

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REGISTRATION OF AMENDMENTS

Amend No	Page No	Signature & Date of incorporation of amendments	Amend No	Page No	Signature & Date of incorporation of amendments

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SECTION 2 SPECIAL STRUCTURES

PART 1

BD 26/04

DESIGN OF LIGHTING COLUMNS

Contents

1. INTRODUCTION

General

1.1 This Standard covers the use of BS EN 40 for the design of lighting columns made from steel, aluminium, and concrete. It sets out the Overseeing Organisation's particular requirements where these differ from, or are additional to those given in the British Standard. In addition, the Standard gives the requirements for lighting columns made essentially from glass fibre reinforced plastic (GFRP). Since this material is being introduced in lighting columns for the first time it may become necessary in due course to review the requirements, on the basis of their performance in service.

1.2 This Standard updates and supersedes BD 26/99. The major changes are as follows:

(i) This Standard has been updated to incorporate the provisions of BS EN 40.

1.3 Where lighting columns are procured through a contract incorporating the Specification for Highway Works (MCHW 1) products conforming to equivalent standards or specifications of other member states of the European Economic Area will be acceptable in accordance with the terms of the 104 and 105 Series of Clauses of that Specification. Any contract not containing these Clauses must contain a suitable clause of mutual recognition having the same effect regarding which advice should be sought.

1.4 Where this Standard requires tests to be carried out the results of tests undertaken by a body or laboratory in a member state of the European Economic Area will be accepted provided that the body or laboratory offers suitable and satisfactory evidence of technical and professional competence and independence. This requirement will be satisfied if the body or laboratory is accredited in a member state of the European Economic Area in accordance with the relevant parts of the EN45000 series of standards for the tests carried out.

Scope

1.5 This Standard sets out the design requirements for lighting columns and wall mounted brackets made from steel, aluminium, concrete and glass fibre reinforced plastic for use on trunk roads including motorways. The requirements for breakaway columns are outside the scope of this Standard.

Implementation

1.6 This Standard should be used forthwith on all schemes for the construction and improvement of trunk roads, including motorways, currently being prepared, provided that, in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay progress. Design Organisations should confirm its application to particular schemes with the Overseeing Organisation. Where the Overseeing Organisation's contract documents are based on the Specification for Highway Works (MCHW 1) use of this Standard is mandatory. In Northern Ireland this Standard should be used on all schemes for the construction and improvement to roads designated by the Overseeing Authority.

2. DIMENSIONAL LIMITATIONS

2.1 The dimensional limitations for the types of lighting columns covered by this Standard shall be:

For steel, aluminium and concrete columns:

(i) post top columns
< 20 m nominal height

(ii) columns with brackets
< 18 m nominal height

(iii) bracket projections
- not exceeding the lesser of 0.25 x nominal height or 3 metres

For glass fibre reinforced plastic columns:

(i) post top columns
< 10 m nominal height

(ii) columns with brackets
< 10 m nominal height

(iii) bracket projections
< 1.5 m

3. USE OF BRITISH STANDARDS AND STANDARDS ISSUED BY THE OVERSEEING ORGANISATIONS

3.1 The design, manufacture and installation of lighting columns shall comply with the relevant requirements of BS EN 40 as amended by this Standard and by the Specification for Highway Works (MCHW 1), hereinafter called the Specification.

3.2 The specific Overseeing Organisation's procedures for the Technical Approval of lighting columns for use on motorways and other trunk roads are given in BD2 (DMRB 1.1).

For lighting columns at very exposed sites, the Technical Approval procedures for high masts, given in BD2 (DMRB 1.1) shall apply.

Within the United Kingdom, very exposed sites are defined as:

(a) sites at high altitude, above 250m;

(b) sites closer than 5km from the coast; and

(c) sites subject to significant local funnelling.

Deflection Criteria

3.3 The horizontal deflection of each lantern connection shall conform to class 2 as specified in BS EN 40-3-3: Table 3.

Fatigue Criteria for Steel Columns

3.4 The rules set out in 3.5 to 3.12 shall be used for steel lighting columns 9m and above in height. 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 3.2. Columns of materials other than steel are not covered by the rules in this Standard.

In all cases the procedures to be used shall be agreed between the designer, the client and the technical approval authority, see 3.2 above.

3.5 Fatigue damage is most likely to occur at or adjacent to welds or near sharp corners creating stress concentrations; particularly vulnerable locations are:

- shoulder joints:
  - at the weld throat;
  - in the parent metal adjacent to weld;
- flange plates:
  - at the weld throat between the column and flange;
  - in the parent metal adjacent to the weld;
- door openings:
  - at welded attachments;
  - at poorly finished cut edges.

3.6 Generally, 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.

For reinforcement at door openings the geometric constraints set out in 3.10 shall be met, and stress ranges around door openings need not be calculated. However if these constraints are not met then the requirements of 3.7 shall be followed.

3.7 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 σ_r, given by:

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

where:

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;

β is the dynamic response factor (Clause 3.2.4 of BS 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 BS 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.

3.8 This stress range shall be less than that obtained from 3.9, appropriate to the class of detail being considered and for a number of cycles N given by:

$$N = 10^6 N_f L$$

where:

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

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

3.9 For a design life of 25 years, the maximum allowable stress range is given in figure 1.1(a) or figure 1.1(b) appropriate to the class of detail under consideration and dependent on the frequency N_f (Hz). The method of defining the S-N curves given in figure 1.1(a) and figure 1.1(b) is by two numbers joined by a hyphen. The first number is the reference strength at 2×10^6 cycles and the second is the m value which is a constant applicable to values of N up to 5×10^6 cycles. The basis of the curves in Figures 1.1(a) and 1.1(b) is given in 3.13.

Note: For a design life of L years figure 1.1 may be used by adopting an effective frequency N_{fe} as the horizontal scale given by:

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

3.10 Fatigue is critically dependent on geometrical configurations and fabrication.

The following geometric and fabrication constraints on cross sections of metal lighting columns shall be satisfied, in order to use the classes of details as provided in 3.11.

(a) Shoulders as shown in figure 1.2 shall have an angle of inclination to the axis of the columns, α, between the following limits:

$12° < \alpha < 35°$

(b) The shoulder weld A as shown in figure 1.2 shall have a throat size 10% greater than the thickness of the adjacent shaft material, t_s.

(c) The column/flange plate weld 1A, 2/1 and 2/2 shown in figures A1, A2 and A3 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

(d) The thickness of the base material t_b shall be not less than the thickness of the adjacent shaft material, t_s.

(e) To ensure that weld detail 6 (see figure A.7) 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.

(f) Stiffened and unstiffened door openings shall comply with the constraints shown in figure 1.3. 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.

3.11 Guidance on classes of typical weld details incorporating stress concentration factors, K_f, which comply with the constraints of 3.10 are given in figures A.1 to A.8 for welds made using normal commercial practice, e.g. manual welds without NDT or other testing. 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.

3.12 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 – 2 standard deviations. Such tests shall be subject to agreement between the parties concerned.

3.13 Figures 1.1(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 σ_o = details category (50, … 120 …)
m = slope of curve (3 for Figure 1.1(a) and 4 for Figure 1.1(b))
σ_R = stress range

(b) The number of cycles relate to the frequency by the equation in 3.8:

$N = 10^6 N_f L$

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

$N = 25.10^6 N_f$

(d) Thus, the relationship between σ_R and N_f (the plots of Figures 1.1(a) and 1.1 (b)) is:

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

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

e.g. for class detail 120: 4

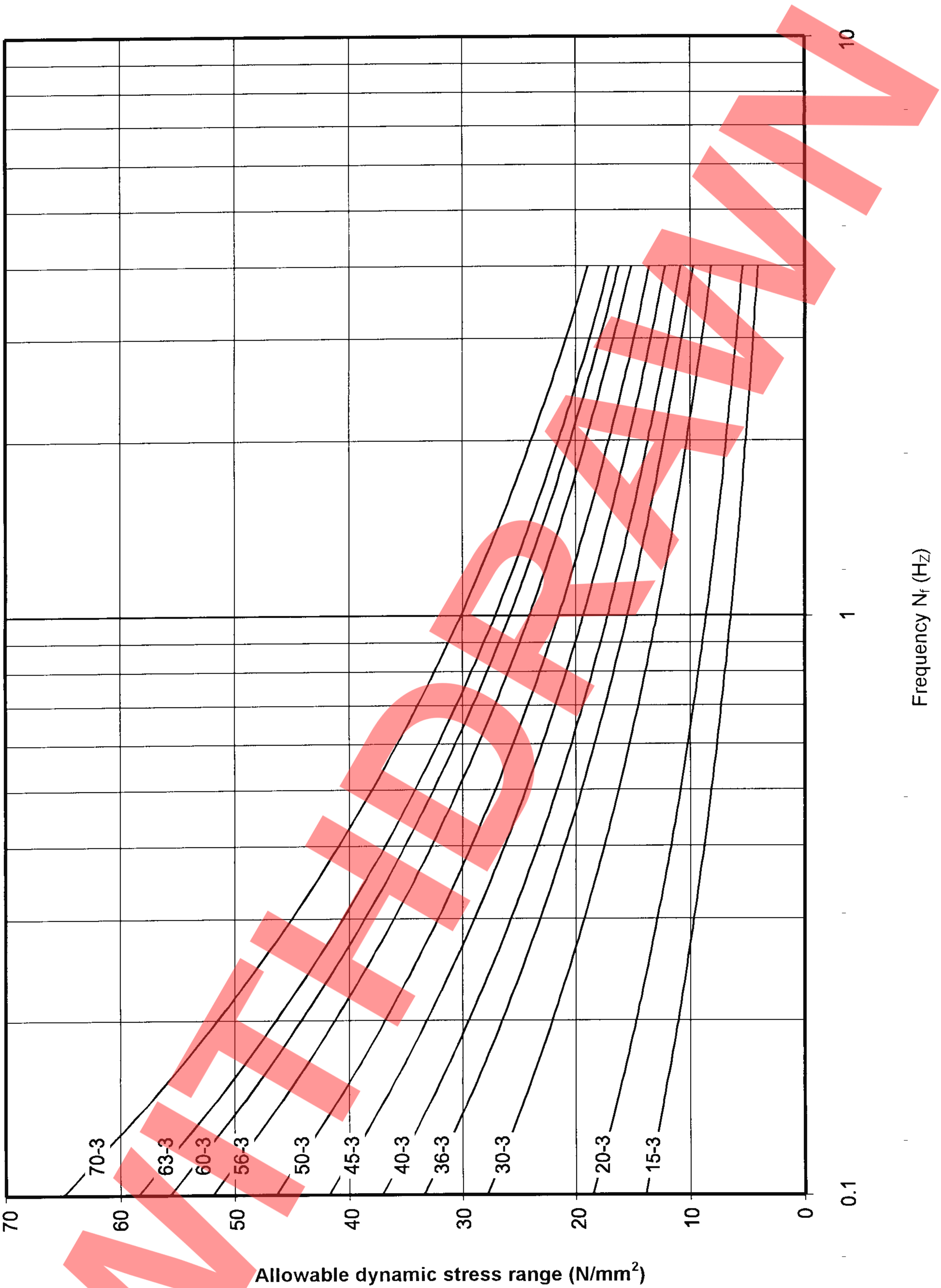
σ_o = 120
m = 4

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

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

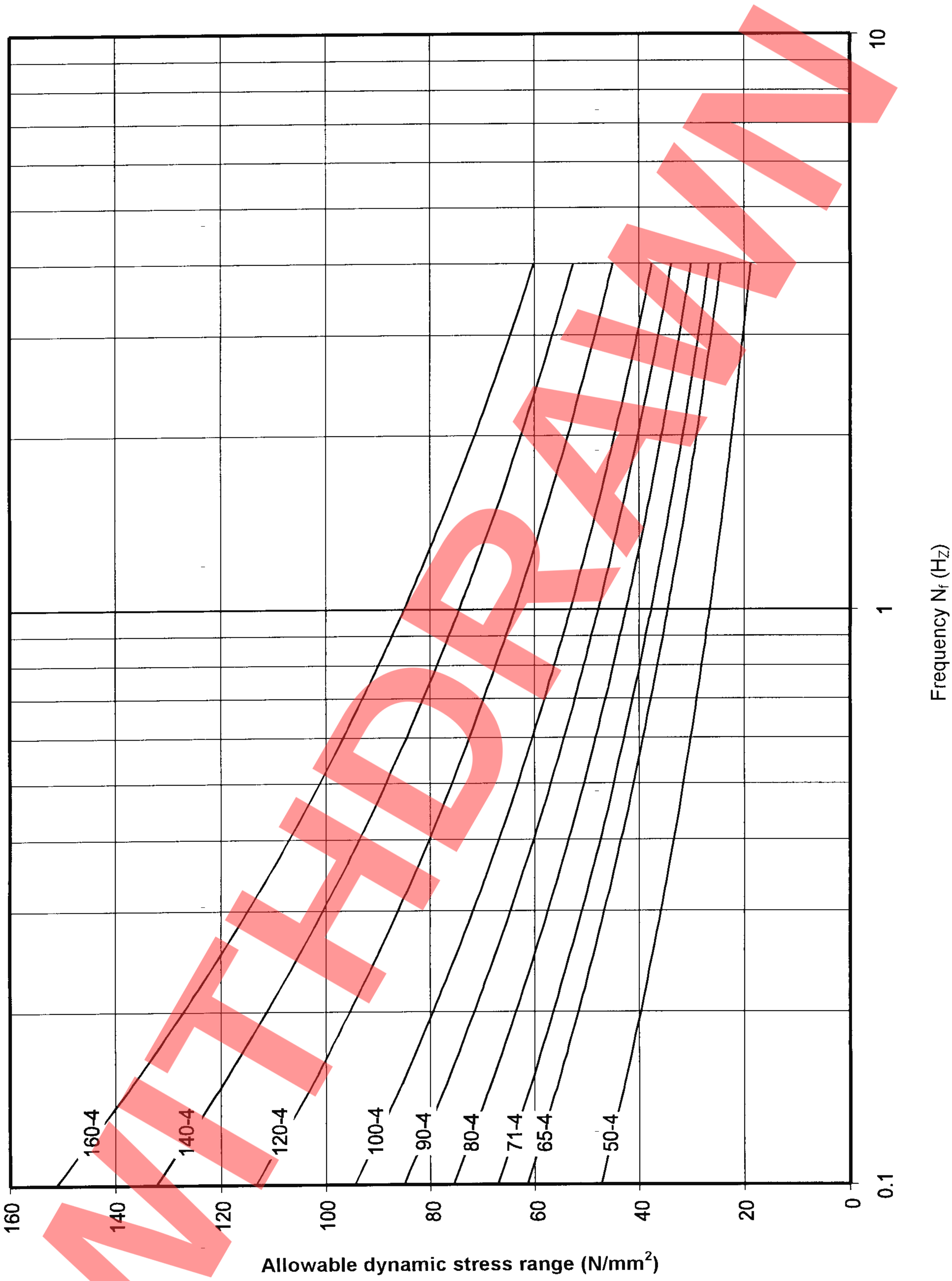
Determination of Shape Coefficients

3.14 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 B of this Standard.



Note: For basis of curves see 3.13

Figure 1.1(a) Fatigue of lighting 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 3.13

Figure 1.1(b) Fatigue of lighting column stress range limit for class of weld detail based on a 25 year design life requirement (m=4)

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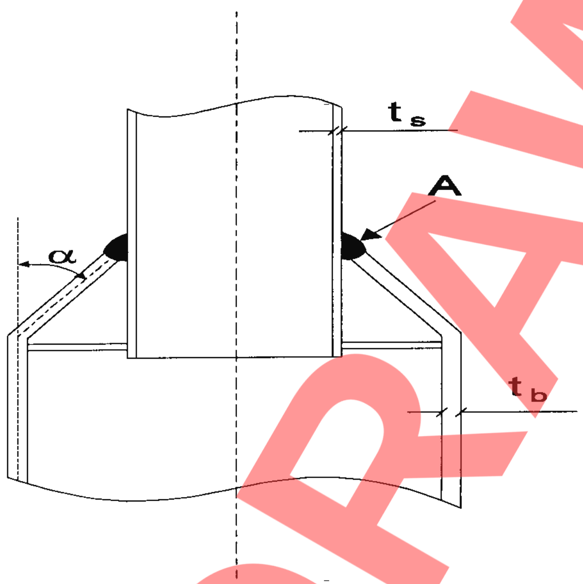

Figure 1.2 Shoulder joint

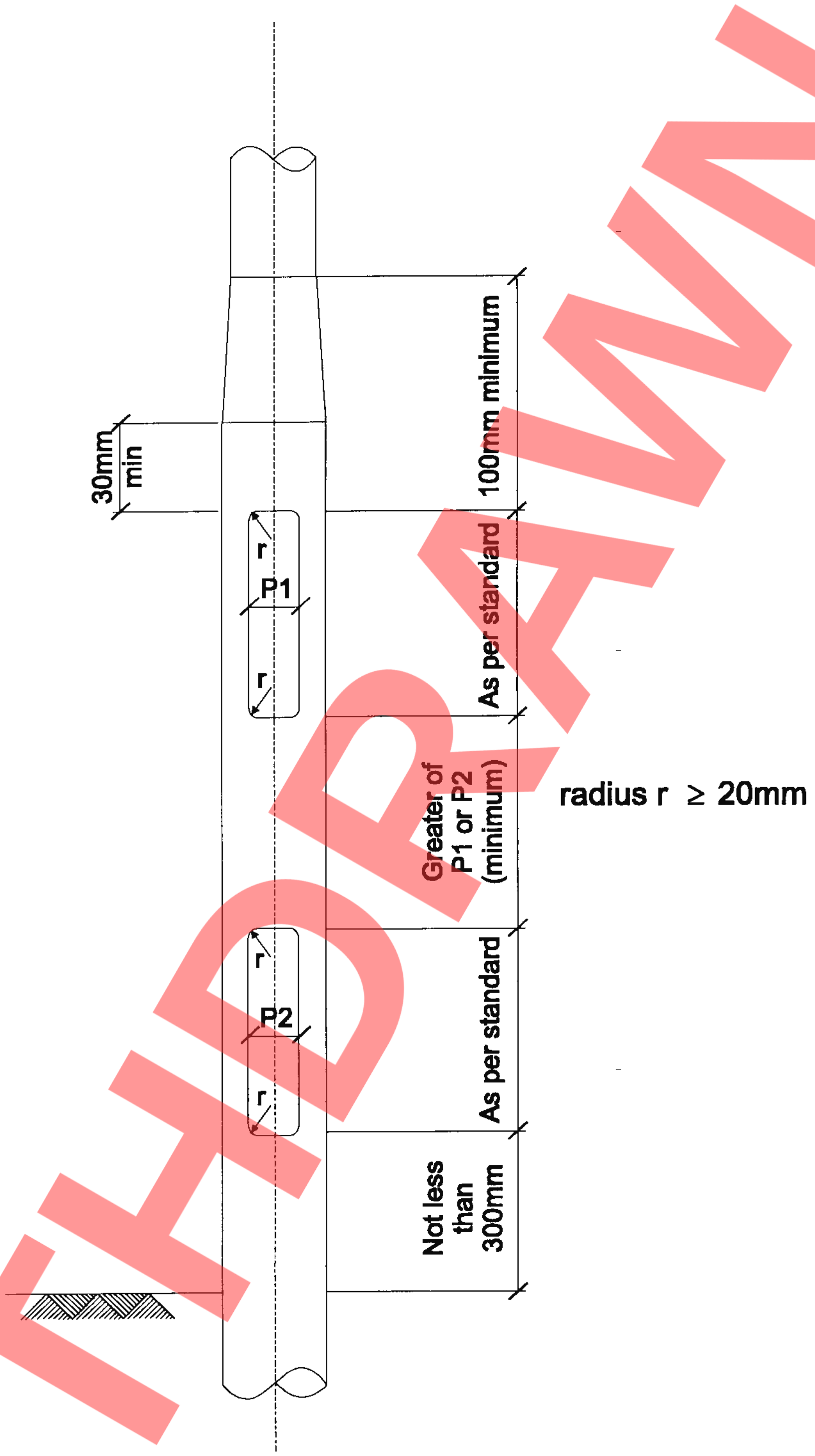

Figure 1.3 Door Openings

4. GLASS FIBRE REINFORCED PLASTIC (GFRP) LIGHTING COLUMNS

Design

4.1 Loading. Design loads and moments shall be determined in accordance with BS EN 40-3-1 and BS EN 40-7 as amended by this Standard.

4.2 The factor β for the dynamic behaviour of the GFRP column shall be determined by reference to Figure 2 given in this Standard.

Verification of Structural Design

General

4.3 The structural design of GFRP columns shall be verified either by calculations or by testing. The test results take precedence in all cases.

Calculations

4.4 Design calculations for GFRP columns shall be in accordance with the requirements for metal columns in BS EN 40-3-3 but with the following amendments:

(a) Bending strength. The coefficient ϕ_1 in BS EN 40-3-3, equation 2 shall be multiplied by a factor K, where K shall not be less than 1 and shall be calculated as follows:

for circular cross-sections:

$$K = \left[2\left[1+\delta_{12}\left(\frac{E_2}{E_1}\right)^{½}\right]\left(\frac{E_2}{E_1}\right)^{½}\left(\frac{G}{E_1}\right)\right]^{½}$$

and for octagonal sections:

$$K = 0.5\left[\left[\left(\frac{E_2}{E_1}\right)^{½}+\delta_{12}\left[\frac{E_2}{E_1}\right]+2\left(1-\delta_{12}\left[\frac{E_2}{E_1}\right]\right)\left(\frac{G}{E_1}\right)\right]\right]$$

Where:

E_1 = Young's modulus in the longitudinal direction (kN/m²)
E_2 = Young's modulus in the transverse direction (kN/m²)
G = in-plane shear modulus (kN/m²)
δ_{12} = Poisson's ratio when loaded in the longitudinal direction with associated transverse strains

$$\delta_{21} = \delta_{12}\frac{E_2}{E_1}$$

(b) Torsional strength. The coefficient ϕ_2 in BS EN 40-3-3, equation 3 shall take the value of:

$$\phi_2 = \left[\frac{0.533(1+\delta_{12})}{(1-\delta_{12}\delta_{21})}\cdot\frac{G}{\tau_u}\cdot\frac{t}{R}\right]^{\frac{3}{2}}$$

Subject to a maximum value of 1 where:

δ_{12}, δ_{21} and G should be as defined above
t = wall thickness of column (mm)
R = radius to external face of circular column or radius of circle inscribed through apexes of an octagon (mm)
τ_u = shear strength of the column material (kN/m²)

4.5 The mechanical properties of the GFRP 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, δ_{12}. A statistical assessment shall be made of the results to determine 95% confidence limits of the values to be used.

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4.6 The partial material factor γ_m for GFRP shall be taken as 1.75.

4.7 Deflection. The calculated horizontal and vertical deflections of the lantern connection due to the load effects specified in BS EN 40-3-1, shall not be greater than 0.065 (w + h) and 0.05w respectively where ‘h’ is the nominal height of the lighting column (in m) and ‘w’ is the bracket projection (in m) both as defined in BS EN 40-1.

Testing

4.8 The method to be used for verifying the design shall be that specified in BS EN 40-3-2. The acceptance criteria for deflection shall be as in 4.7 above. The residual deflection after removal of the design load shall not exceed 4% of the deflection due to this load.

Use of Other Materials

4.9 All other materials incorporated in the GFRP columns shall comply with the Specification and the relevant parts of BS EN 40.

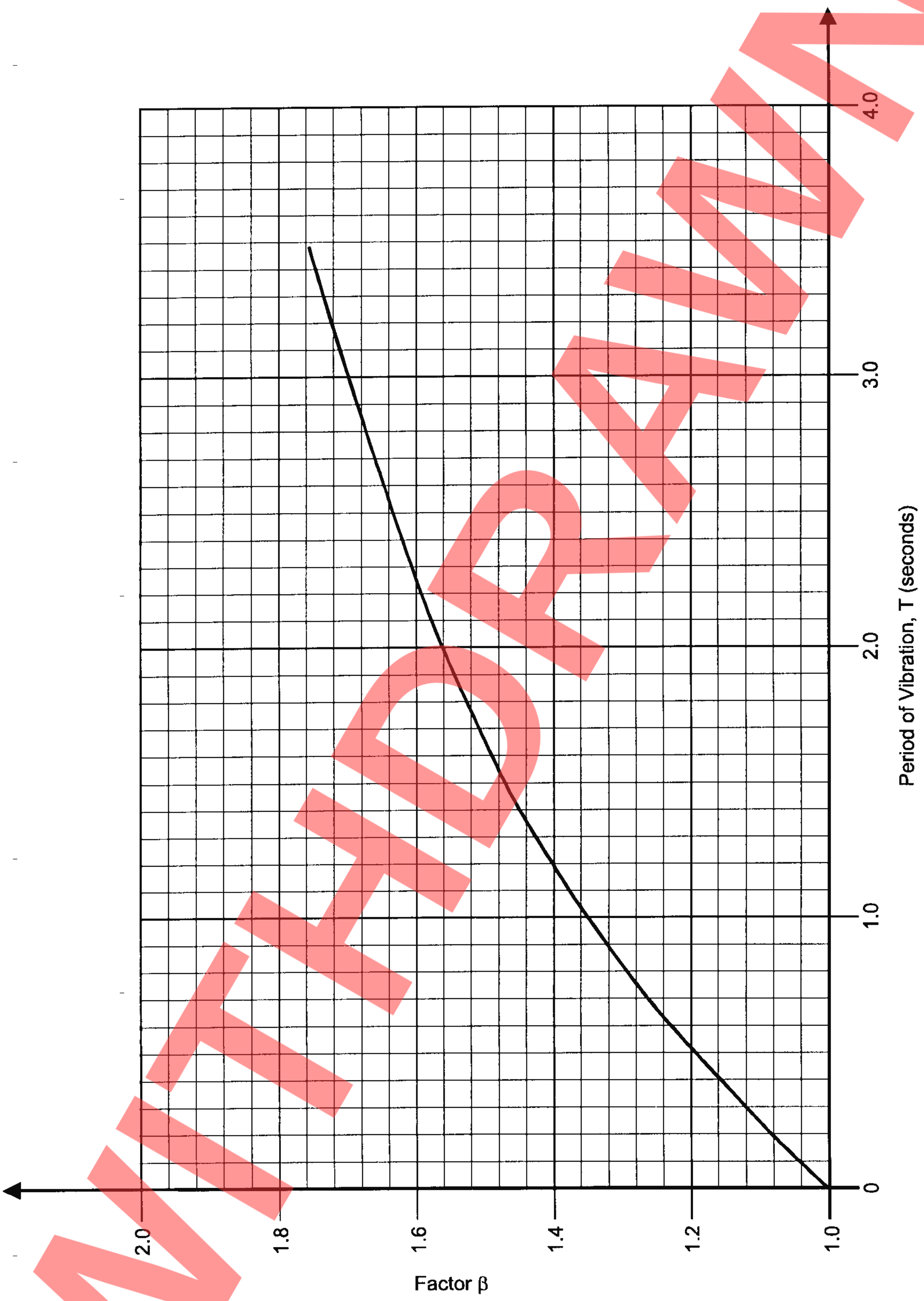

Figure 2

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5. DOORS IN LIGHTING COLUMNS

5.1 Door openings of the sizes given below shall be specified when providing information for Appendix 13/1 of the Specification.

5.2 Alternative door openings selected from the sizes given in BS EN 40-2 may be used, providing they are shown to be adequate for the size of equipment to be housed and maintained, in the column.

5.3 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 which shall be sufficiently robust, to support the door in severe gale conditions.

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	-

6. WALL MOUNTED BRACKETS

6.1 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 8.9.

6.2 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.

7. ATTACHMENTS TO LIGHTING COLUMNS

7.1. All lighting columns shall be designed for the attachment given in paragraph 7.2.

7.2 Design provision for lighting columns to carry an attachment:

(i) The attachment shall be taken as a sign, rectangular in elevation, with a surface area of 0.3 m^2.

(ii) The eccentricity from the centre line of the column to the centre of area of the sign shall be taken as 300 mm.

(iii) The height above ground level at the column to the centre of area of the sign shall be taken as 2500 mm.

(iv) The orientation of the sign shall be selected to produce the most adverse effects for the design condition being considered.

7.3 The forces due to dead and wind loads on sign and bracket projecting from the column shall be determined in accordance with BS EN 40-3-1. The shape coefficient of the sign shall be taken as 1.8.

7.4 Where larger signs, waste paper containers, flower baskets etc, are to be attached, the column shall be designed to resist the additional loadings which shall be described in Appendix 13/1 of the Specifications. Where appropriate the additional loadings shall be calculated in accordance with paragraph 7.3.

7.5 Lighting columns designed to carry attachments greater than that defined in 7.1 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 listed in the Column and Bracket Data sheet, proforma of which is included in Appendix 13/2 of the Notes for Guidance on the Specification for Highway Works (MCHW 2). All other requirements for the identifying mark shall be as required in the Specification.

8. FLANGE PLATES AND FOUNDATIONS

Foundations - General

8.1 Foundations shall be designed in accordance with paragraphs 8.2 to 8.3 as appropriate. The structural concrete shall be designed in accordance with BS 5400: Part 4.

Foundations for Planted Columns

8.2 The design of the foundation to determine its diameter and planted depth shall be carried out in accordance with the design procedure given in BS EN 40-2 for the three types of soils listed in National Appendix B.

In the design procedure given in BS EN 40-2 the design loadings as given in Clause B.5 of National Appendix B shall be deleted and amended as follows:

"B.5 Design loadings

The design loadings given in BS EN 40-2 shall be adopted for calculating planting depths".

8.3 Using the above designs, the requirements for the planted columns, including the diameter, planted depth and the type of backfill, to suit the type of soil, shall be selected at each column location.

Foundation for Columns with Flange Plates

8.4 The design of foundation shall be based on the design method given in BS 8004. 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 design moment M_i and the design moment as given in BS EN 40-3-1.

F_{fd} shall be the greater of the impact design shear force F_i and the design horizontal force calculated to BS EN 40-3-1 provisions.

M_i and F_i are derived as follows:

$M_i = k_{si}\, M_R$
$F_i = k_{si}\, M_R$

where the ultimate moment of resistence of the actual column at the base level, M_R, together with an equivalent ultimate shear force, F_R, are derived as follows:

M_R shall be calculated in accordance with BS EN 40-3-3, where:

$$M_R = M_{up} = \frac{\sigma_s\, \phi_1\, Z_p}{\gamma_m \times 10^3}\ (\text{in N.m})$$

An upper bound to the equivalent ultimate shear force may be taken as:

$$F_R = \frac{M_{up}}{0.5}\ (\text{in N}) \quad [= 2\, M_R]$$

where M_{up}, σ_s, ϕ_1, Z_p and γ_m are all as defined in BS EN 40-3-3.

The soil impact factor, k_{si} is based on the three types of soil listed in BS EN 40-2 National Appendix B and is derived as follows:

Soil Type	Soil Impact Factor k_{si}
Good	0.2
Average	0.3
Poor	0.5

Note: Where the soil type is unknown, k_{si} shall be taken as 0.5 for Poor soil type.

Fixing of Columns with Flange Plates to Foundation or Bridge Deck

8.5 A column with 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.

8.6 The flange plate shall be fixed by an attachment system, usually in the form of holding down bolts which connect with an anchorage. Anchorages of expanding type shall not be used. The design of attachment systems and anchorages shall be such that removal and replacement of damaged lighting columns may be readily achieved. This shall be achieved by providing an internally threaded component in the anchorage to receive the holding down bolt.

8.7 Typical arrangements are shown in Figure 3, which applies to both plates supported on bedding material and plates supported on levelling nuts only, without effective bedding.

Design of Welds

8.8 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 8.4 above.

8.9 Welds shall be deemed to meet these requirements provided the throat thickness of the top weld is not less than k x 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 3 and for the outer filler weld of detail A in Figure 3, k = 1.0 for a full penetration butt weld.

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

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

Design of Flange Plate

8.10 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 calculated in 8.4 above, and in general shall be checked about bending parallel to one side (axis u-u) and on the diagonal (axis v-v), see Figure 3.

8.11 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 3. The more general case is covered in Annex C.

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

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

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

8.13 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 0.25D in diameter (refer to Figure 3, detail B), M_p is given by:

$$M_p = \frac{(\sqrt{2}\,c - 0.63\,D) \times t_f^2}{4} \times \frac{\sigma_f}{\gamma_m \times 10^3 \times \gamma_{f3}} \quad \text{(in N.m)}$$

where: γ_m = 1.15; γ_{f3} = 1.0; c = the width of the flange plate (in mm); t_f = the thickness of the flange plate (in mm); σ_f = the yield stress in the flange plate (in N/mm²); and D is as defined in 8.12. Where the centrally located hole and the column base are the same diameter (refer to Figure 3, Detail A), M_p shall be calculated in accordance with the procedure given in Annex C.

8.14 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 Holding Down Bolts

8.15 The tensile stress in holding down bolts may be taken as:

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

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 3.

8.16 The shear stress in the bolts may be taken as:

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

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.

8.17 Bolts in tension and shear shall comply with:

$$\left\{\left(\frac{\sigma}{\sigma_t}\right)^2 + 2\left(\frac{\tau}{\sigma_q}\right)^2\right\}^{1/2} \le \frac{1}{\gamma_m\,\gamma_{f3}}$$

where: γ_m is taken as 1.30; γ_{f3} is taken as 1.00; σ_t is the lesser of: (i) 0.7 x minimum ultimate tensile stress; or (ii) either the yield stress or the stress at permanent set of 0.2%, as appropriate; σ_q = yield stress of bolts (factored by 0.85 in the case of black bolts).

8.18 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° cone recommended in "Holding down systems for steel stanchions" CS/BCSA/ Constrado, 1980.

Bearing Stresses under Flange Plates

8.19 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:

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

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

8.20 The bearing stresses in any bedding mortar under the flange plates shall not exceed 20 N/mm². The maximum bearing stresses on the concrete under a flange plate shall be in accordance with the requirements of BS 5400: Part 4 as implemented by BD 24 (DMRB 1.3).

8.21 For foundations on masonry, the guidance given in BS 5628: Part 1 should be followed with regard to bearing stresses.

8.22 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

8.23 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 $\sigma\,A_{et}$ (in N); and

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

where σ, τ, A_{et} and A_{eq} are all as derived above.

The capacity of the anchorage shall be derived in accordance with Sections 8.1 to 8.4 above, together with Figure 6 of BS EN 40-2, and relevant parts of BS 8004.

8.24 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

8.25 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 ISO 7093, provided the hole or width of the slotted hole does not exceed d_o + 4mm where d_o is the diameter of the holding down bolts.

For slotted holes, which provide flange plate rotations of up to ± 5° as shown in Figure 4, 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 ISO 7093 may be used provided the width of the slotted holes does not exceed d_o + 6mm.

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.

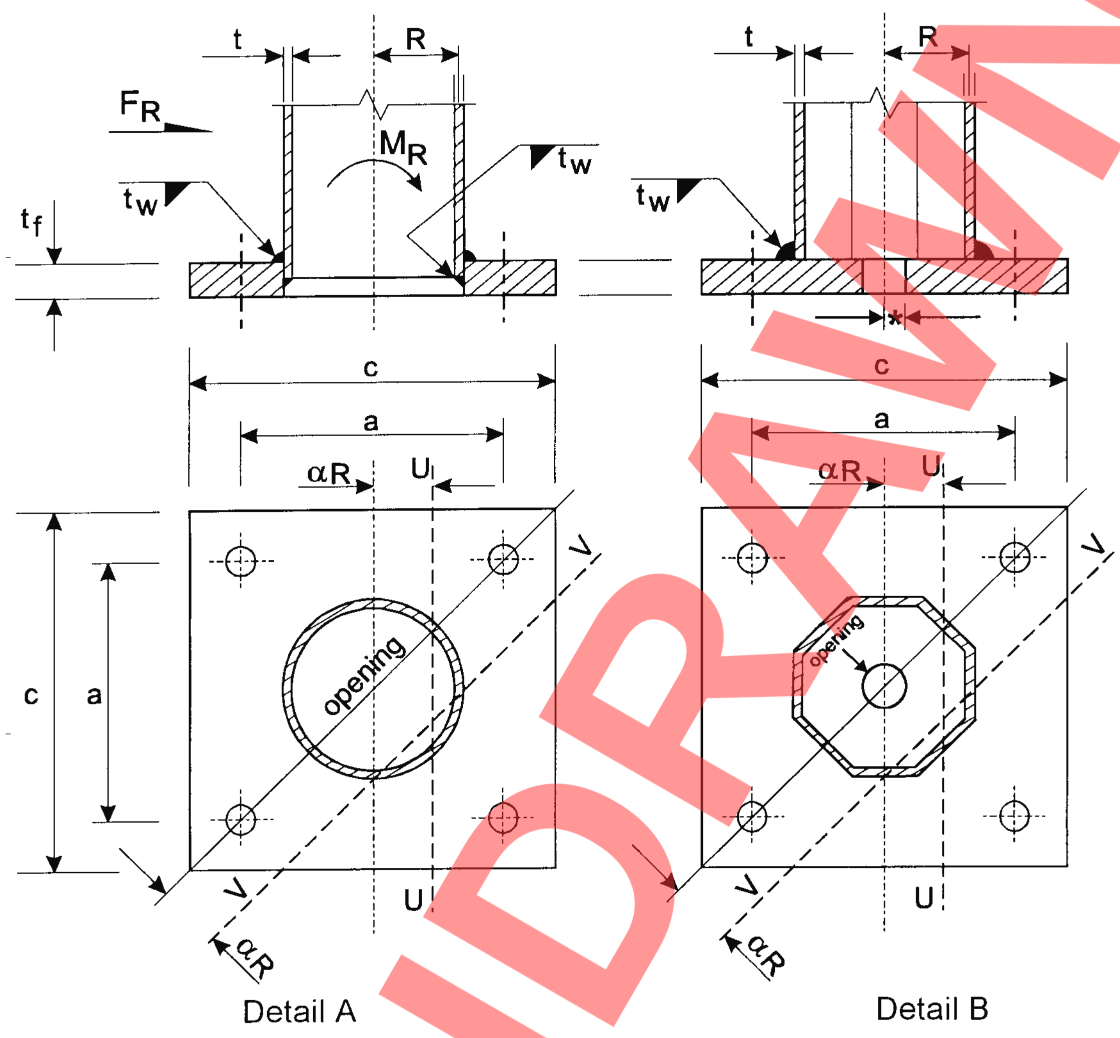

Figure 3 Typical arrangement of flange plate

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 BS EN 40-3-3 (Figure 3).

3: '*': Radius of centrally located hole shall not exceed 0.25R.

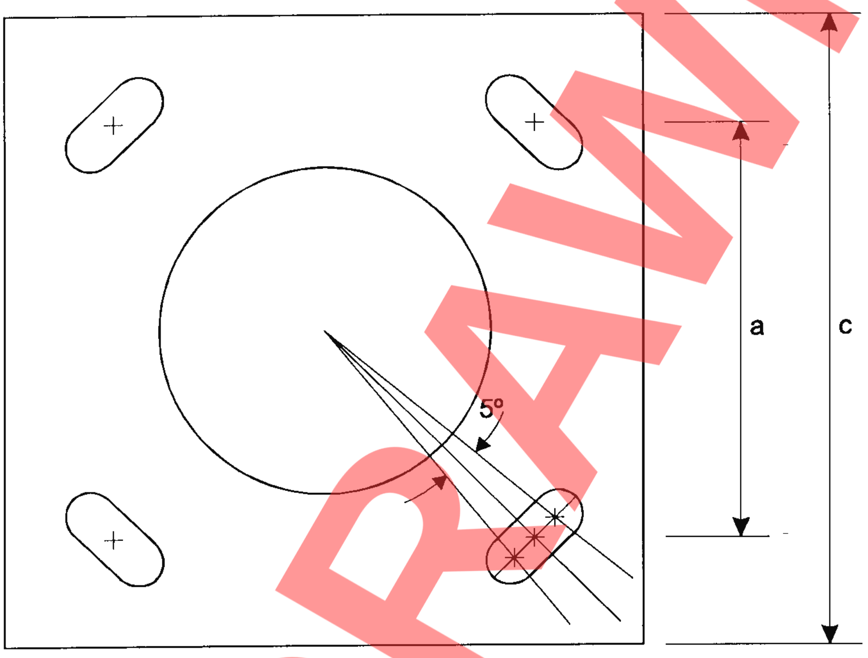

Figure 4 Slotted holes arrangement

9. REFERENCES

9.1 British Standards Institution

BS EN 40: Lighting Columns:

Part 1: - Definitions and terms

Part 2: - General requirements and dimensions

Part 3-1: - Design and verification - Specification for characteristic loads

Part 3-1: - Design and verification - Verification by testing

Part 3-1: - Design and verification - Verification by calculation

Part 4: - Specification for reinforced and prestressed concrete lighting columns

Part 5: - Specification for steel lighting columns

Part 6: - Specification for aluminium lighting columns

BS 6399-2: - Loading for buildings. Code of practice for wind loads

BS 8004: - Code of Practice for Foundations

BS 5400: Steel, Concrete and Composite Bridges:

Part 3: - Code of Practice for design of steel bridges

Part 4: - Code of Practice for design of concrete bridges

BS 5628: - Code of Practice for use of masonry Part 1: Structural use of unreinforced masonry

DD ENV 1993-1-1: 1992: Eurocode 3: Design of Steel Structures: Part 1.1: General Rules and Rules for Buildings

9.2 Design Manual for Roads and Bridges

Volume 1: Section 1 Approval Procedures

BD 2 - Technical Approval of Highway Structures on Motorways and other Trunk Roads;

Part I - General Procedures (DMRB 1.1)

Part IV - Procedures for lighting Columns.

(DMRB 1.1)

Volume 1: Section 3 General Design

BD 13 - Design of Steel Bridges: Use of BS 5400: Part 3: 1982. (DMRB 1.3)

BD 24 - Design of Concrete Highway Bridges and Structures: Use of BS 5400: Part 4: 1990 (DMRB 1.3)

9.3 Manual of Contract Documents for Highway Works. (MCHW)

Volume 1: Specification for Highway Works (MCHW 1)

Volume 2: Notes for Guidance on the Specification for Highway Works (MCHW 2)

9.4 International Standards Organisation

ISO 7093 – Plain washers – Large series – Product grades A and C, First edition – 1983-09-15

9.5 Other Publications

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.

10. ENQUIRIES

All technical enquiries or comments on this Standard should be sent in writing as appropriate to:

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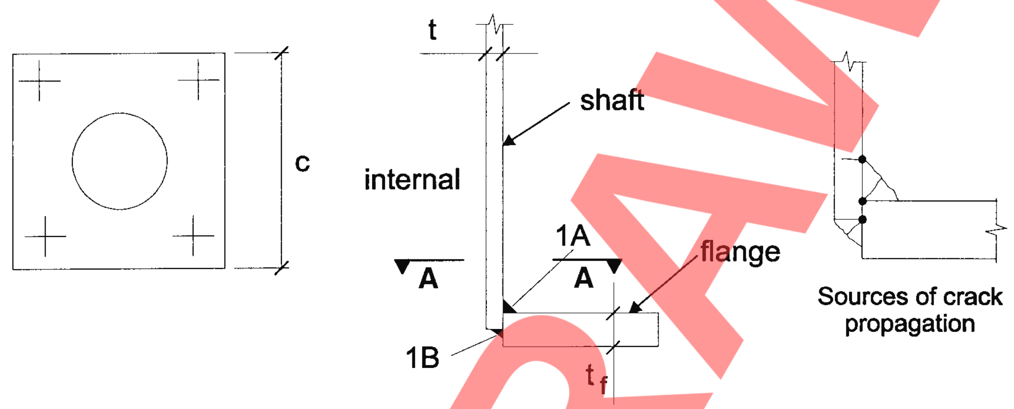
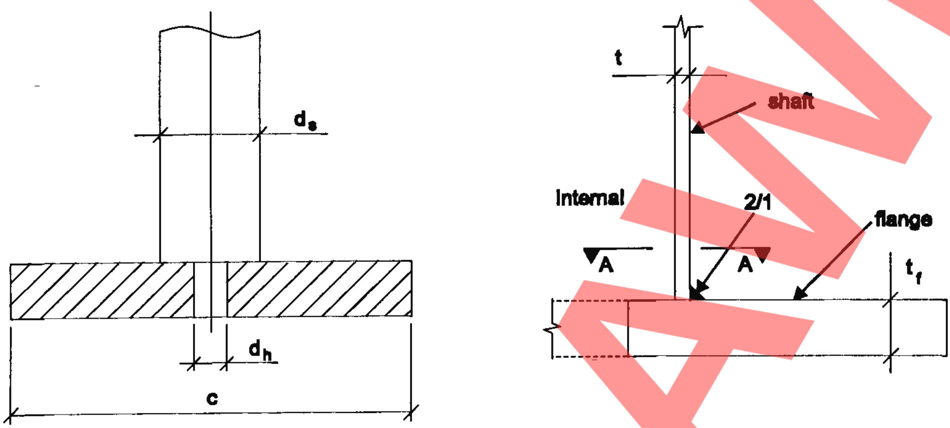
ANNEX A GUIDANCE FOR WELD CLASSIFICATION



Weld	Section to be checked	Class of Parent Metal	$K_f = K_t K_b K_h$
1A 1B	A-A	$\boxed{30K_f - 4}$ (1)	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_b = \left(\frac{\sqrt{t_f}}{5}\right)^3$ but $\nless$ 0.33 and $\ngtr$ 1.45 $K_h = 1.0$

(1) Provided weld 1A is designed for transfer of the total load and weld 1B is for sealing only. Otherwise a detailed stress analysis shall be undertaken and the resulting stress concentration factors used.
(2) No fatigue check need be undertaken on the weld throat provided the criteria of 3.10 are met.

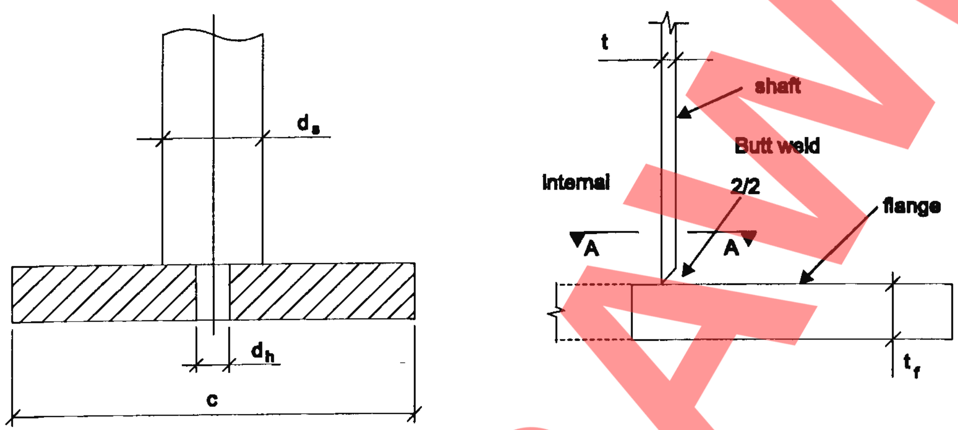
Figure A.1 Weld detail type 1



Weld	Section to be checked	Class of Parent Metal	$K_f = K_t K_b K_h$
2/1	A-A	$\boxed{30K_f - 4}$	$K_t = \sqrt{\frac{2}{\sqrt{t}}}$ $K_b = \left(\frac{\sqrt{t_f}}{5}\right)^3$ but $\nless$ 0.33 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 3.10 are met.

Figure A.2 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	$\boxed{30K_f - 4}$	$K_t = \sqrt{\dfrac{2}{\sqrt{t}}}$ $K_b = \left(\dfrac{\sqrt{t_f}}{5}\right)^3$ but $\not<$ 0.33 and $\not>$ 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 3.10 are met.

Figure A.3 Weld detail type 2/2

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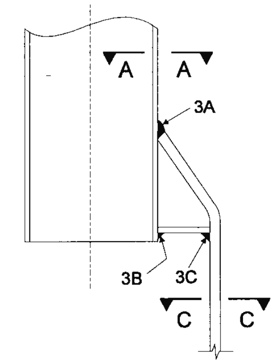
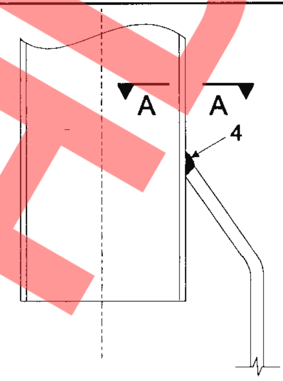


Weld	Section to be checked	Class	
		Parent Metal	Weld Throat
3A	A-A	90 - 4 [1]	See [2]
3B	No check necessary if criteria of 3.10 are met	90 - 4 [1]	See [2]
3C	C-C		

(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 3.10 are met.

Figure A.4 Weld detail type 3



Note: Joint detail not recommended for other than lightly loaded short columns.

Weld	Section to be checked	Class	
		Parent Metal	Weld Throat
4	A-A	71 - 4 [1]	See [2]

(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 3.10 are met.

Figure A.5 Weld detail type 4

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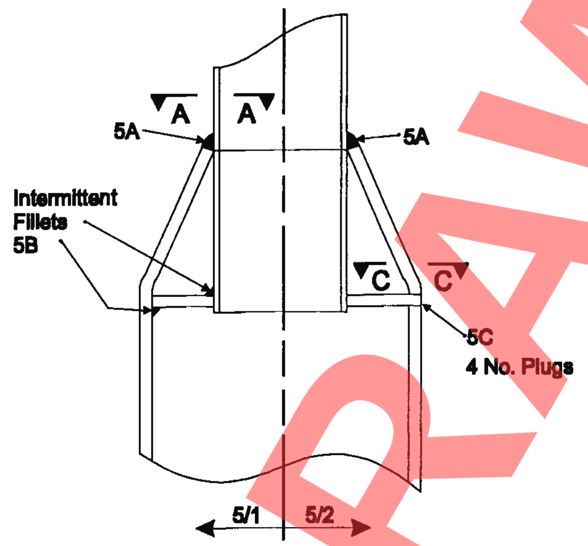
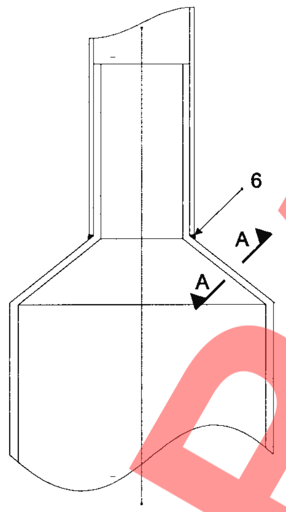


Weld	Section to be checked	Class	
		Parent Metal	Weld Throat
5A	A-A	$\boxed{90 - 4}$ (1)	See (2)
5B	No check necessary if criteria of 3.10 are met	$\boxed{120 - 4}$	See (2)
5C	C-C. Plugs not ground smooth	$\boxed{90 - 4}$	-
	C-C. Plugs ground smooth	$\boxed{120 - 4}$	-

(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 3.10 are met.

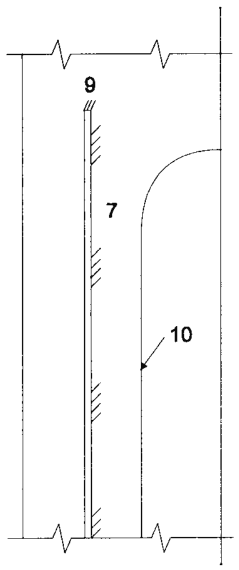
Figure A.6 Weld detail type 5

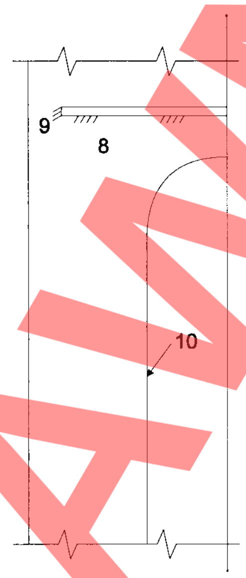


Weld	Section to be Checked	Class		
		Upper Tube Parent Metal	Lower Tube Parent Metal	Weld Throat
6	A - A	N / A (1)	71 - 4	See (2)

(1) Assumes tight fit between tubes for load transfer by shear.
(2) No fatigue check need be undertaken on the weld throat.
(3) Refer to 3.10(e) regarding the detailing of this joint.

Figure A.7 Weld detail type 6





Welded stiffener adjacent to opening parallel to column

Welded stiffener adjacent to opening transverse to column

Weld/Detail	Detail to be Checked	Class [1]
7	Intermediate weld	80 - 3
8	Intermediate weld	71 - 3
9	End Weld	50 - 3
10	Flame cut edge	112 - 4

(1) No fatigue stress calculations need be undertaken provided the geometric and fabrication constraints of 3.10 have been met. Otherwise the above classification should be adopted in conjunction with a detailed stress analysis incorporating appropriate stress concentration factors.

Figure A.8 Weld detail types 7 to 10

ANNEX B DETERMINATION OF SHAPE COEFFICIENTS BY TESTING

B.1 Shape Coefficients for Columns

General

B.1.1 Properly conducted wind tunnel tests on column and bracket shall only be undertaken when shape coefficients are not available from BS EN 40-3-1 or in recognised International Standards. Adoption of values from these standards or from wind tunnel tests shall 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.

B.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.

B.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° 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 Technical Approval Authority set out in B.1.1.

B.2 Shape Coefficients for Lanterns and Brackets

B.2.1 The shape and lift coefficients for lanterns may be determined from wind tunnel tests as required by BS EN 40-3-1. These tests shall be carried out on a full scale lantern shape 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 lanterns, eg. photo-electric control units, gear component extensions etc, these shall be included in the test specimen.

B.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 B.2.3. All shape coefficients shall be based on the projected area of the lantern normal to the air flow.

B.2.3 Forces on a lantern shall be measured at increments of rotation of approximately 1° between the limit of ± 10° to the horizontal. BS EN 40-3-1 requires the maximum value between ± 5° to the horizontal but a more conservative value shall 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 shall also be taken into account. Where an increase in shape coefficient obtained with a rotation within the limits of ± 10° then this value shall be adopted.

ANNEX C DETAILED DESIGN OF FLANGE PLATES

C.1 General

C.1.1 The procedure given in Chapter 8 for the design of flange plates assumes circular or octagonal columns connected to square flange plates and supported by four holding down bolts symmetrically disposed. Where these constraints are not satisfied the following procedure shall be used.

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.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, defined in 8.4 and the equivalent shear force, F_R. The connection may be achieved by welds of leg length, t_w as shown in Figure 3, 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:

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

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

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

τ_R the resultant weld stress shall be taken as:

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

where R = mean radius of cross section (in mm);
t_w = fillet weld leg length (in mm).

C.3 Capacity of Welds

C.3.1 The stress in the fillet welds, τ_R, shall not exceed the weld capacity τ_D given by:

$$\tau_D = \frac{k\,(\sigma_y + 455)}{2\,\gamma_m\,\gamma_{f3}\sqrt{3}} \text{ (in N/mm}^2\text{)}$$

where σ_y is the yield stress of the column section (σ_s) or the flange plate (σ_f) whichever is the lesser;
γ_m is taken as 1.20;
γ_{f3} is taken as 1.00;
k = 0.9 for side fillets; or
1.4 for end fillets in end connections; or
1.0 for all other welds.

(Where inner fillets and outer fillets are used together k may be aggregated, e.g. k = 2.8 for detail A in Figure 3 since both are effectively end fillets for an end connection.)

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 calculated in 8.4, 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:

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

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

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where R = mean radius of the column cross section (in mm);
a = spacing of the bolts (in mm);
and α relates to the position considered for maximum bending in the plate. In lieu of more thorough analysis α may be based on the centroid of the welds on the tensile side, i.e. α 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 C1, detail A) M_p is given by:

$$M_p = \left(c - 2R\sqrt{1-\alpha^2}\right)\frac{t_f^2}{4}\,\frac{\sigma_f}{\gamma_m\,\gamma_{f3}\,10^3}\ \text{(in N.m) for u-u axis;}$$

and

$$M_p = \left(c\sqrt{2} - 2R(\alpha + \sqrt{1-\alpha^2})\right)\frac{t_f^2}{4}\,\frac{\sigma_f}{\gamma_m\,\gamma_{f3} \times 10^3}\ \text{(in N.m) for v-v axis}$$

where γ_m is taken as 1.15;
γ_{f3} is taken as 1.00;
c = the width of the flange plate (in mm);
t_f = the thickness of the flange plate (in mm);
σ_f = the yield stress of the flange plate (in N/mm²).

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:

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

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 x a) should not be taken as greater than (a + αR + 0.5c) for axis u-u, nor greater than 0.7(a + 0.7αR + 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 8.16, combined shear and tension in 8.17 and capacity of the anchorage from 8.18

C.6 Check on Bearing Stress Below the Flange Plate

C.6.1 The bearing stress given in 8.19 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 v-v, on either a plastic or elastic basis as required. The maximum calculated bearing stress shall not exceed the value determined in accordance with 8.20.

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

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

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

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

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

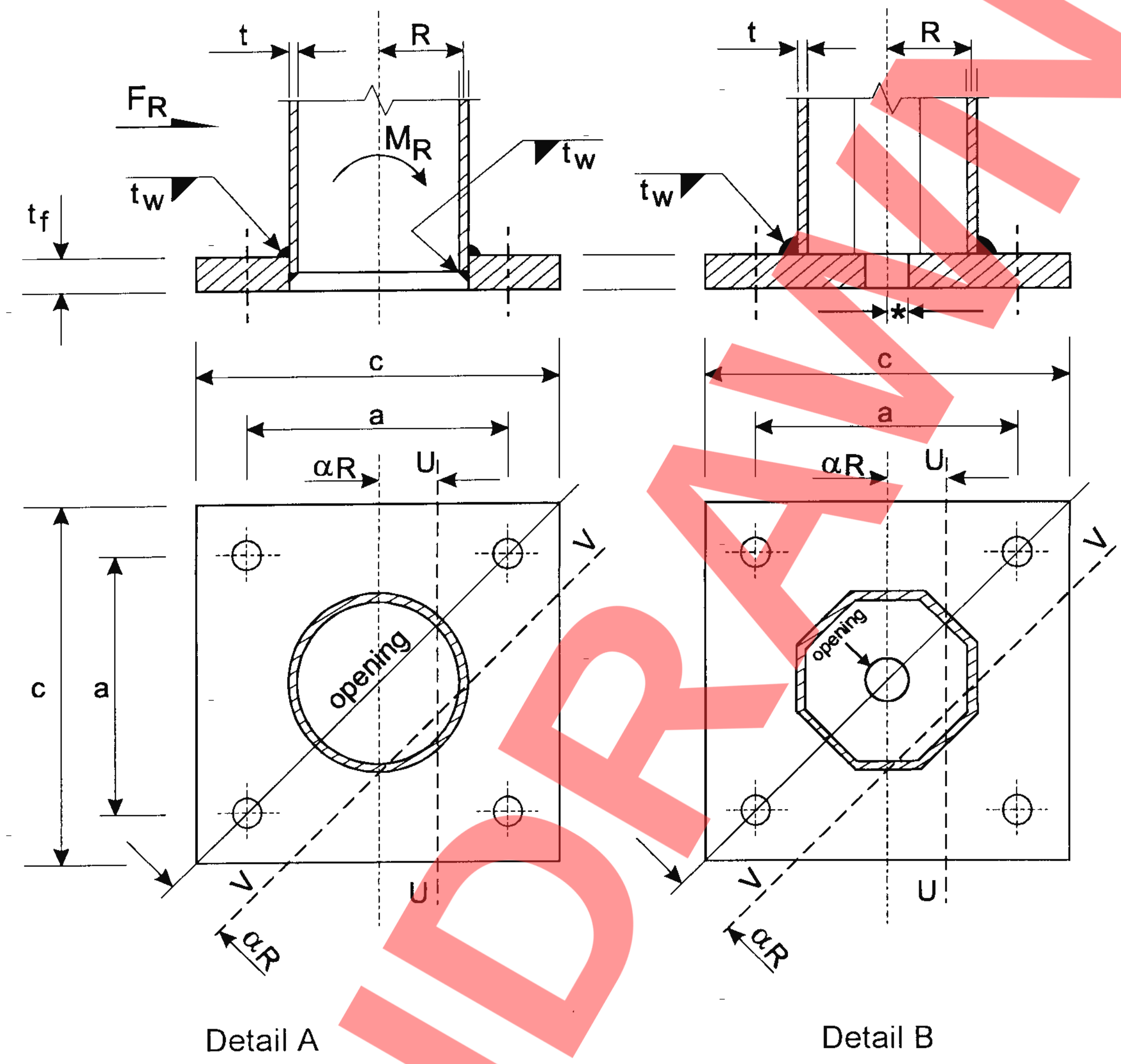

Figure C.1 Typical arrangement of flange plate

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 BS EN 40-3-3 (Figure 3).

3: ‘*’: Radius of centrally located hole shall not exceed 0.25R.