VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 11

BD 56/96

THE ASSESSMENT OF STEEL HIGHWAY BRIDGES AND STRUCTURES

SUMMARY

This Standard gives requirements for the assessment of existing steel highway bridges and structures on motorways and other trunk roads.

INSTRUCTIONS FOR USE

BD 56/96 is a new document in the Design Manual for Roads and Bridges.

1. Insert BD 56/96 into Volume 3, Section 4, at Part 11.

2. Archive this sheet as appropriate.

Note: A quarterly Index with a full set of Volume Contents Pages is available separately from HMSO.

THE HIGHWAYS AGENCY

THE SCOTTISH OFFICE DEVELOPMENT DEPARTMENT

THE WELSH OFFICE
Y SWYDDFA GYMREIG

THE DEPARTMENT OF THE ENVIRONMENT FOR
NORTHERN IRELAND

The Assessment of Steel Highway Bridges and Structures

Summary: This Standard gives requirements for the assessment of existing steel highway bridges and structures on motorways and other trunk roads.

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REGISTRATION OF AMENDMENTS

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VOLUME 3 HIGHWAY STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 11

BD 56/96

THE ASSESSMENT OF STEEL HIGHWAY BRIDGES AND STRUCTURES

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Chapter

November 1996

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1. INTRODUCTION

General

1.1 This Standard, which for assessment purposes replaces BD 13 (DMRB 1.3), gives requirements for the assessment of existing steel structures and structural elements, and shall be used in conjunction with BD 21 (DMRB 3.4.3).

1.2 Annex A of this Standard contains the relevant assessment clauses and appendices which have been presented as additions and amendments to the design clauses and appendices in BS 5400: Part 3 as amended by the Appendix A of BD 13 (DMRB 1.3). These additions and amendments have been specifically developed to suit assessment conditions and, therefore, must not be used in new design or construction.

1.3 BA 56 (DMRB 3.4.12) accompanies this Standard, giving the necessary background information and also guidance on the application of this Standard. It is recommended that the Advice Note should be used in conjunction with this Standard.

1.4 Where there is no assessment addition to a clause in Annex A, the existing design clause as amended by this Standard shall be applicable for assessment.

Scope

1.5 This Standard gives requirements for the assessment of existing steel highway bridges and structures on motorways and other trunk roads. For use in Northern Ireland, this Standard will be applicable to those roads so designated by the Overseeing Organisation.

Implementation

1.6 This Standard shall be used forthwith for the assessment of steel highway bridges and structures. The requirements shall be applied to assessments already in progress provided that, in the opinion of the Overseeing Organisation, this would not result in significant additional expense or delay. Its application to particular assessments shall be confirmed with the Overseeing Organisation.

2. ASSESSMENT OF STRENGTH

General

2.1 The objective of this Standard is to produce a more realistic assessment of the strength of steel elements than has previously been possible using the requirements of the existing design code. This in part is achieved by taking advantage of the information available to an assessing engineer in respect of the material strength, geometrical properties and imperfections etc which can only be predicted at the design stage.

2.2 Many of the criteria given in the design code are based on experimental evidence which in some cases have been either conservatively interpreted for use in design or updated by later evidence allowing a less conservative interpretation. For assessment purposes such criteria have been reviewed and amended where appropriate.

2.3 The assessment additions will enable any combination of the following aspects to be dealt with in assessment:

a) measured imperfections and sizes different from those assumed in the code;

b) structural steelwork components and connections not fabricated or erected in accordance with the Specification for Highway Works (MCHW 1);

c) structural steel material not complying with the relevant standards;

d) general configurations and shape limitations not complying with the limitation in BS 5400 Part 3;

e) restraint stiffnesses and strengths not complying with BS 5400 Part 3; and

f) outmoded forms of construction not complying with BS 5400 Part 3.

Global Analysis

2.4 Plastic analysis of load effects at ultimate limit state for beams and slabs may be permitted provided that the components are both compact and stocky and that serviceability criteria are met.
Requirements are given when plastic global analysis is to be used.

Partial Factor for Loads, γ_{fL}

2.5 Assessment loads Q_A^* shall be obtained by multiplying the nominal loads, Q_K by γ_{fL}, the partial safety factor for loads. The relevant values of γ_{fL} are given in BD 21 (DMRB 3.4.3). The assessment load effects, S_A^* shall be obtained by the relation:

$$S_A^* = \gamma_{f3} \text{ (effects of } Q_A^*)$$

where γ_{f3} is a factor that takes account of inaccurate assessment of the effects of loading, unforeseen stress distribution in the structure and variations in dimensional accuracy in construction. γ_{f3} shall be taken as 1.1 for the ultimate limit state and 1.0 for the serviceabilty limit state.

Partial Safety Factor for Materials, γ_m

2.6 An important feature of the design code is the application of the partial safety factor for material strength, γ_m, to the characteristic values. In assessment, a reduced value of γ_m may be used as an alternative based on the results of laboratory tests.

Limit State

2.7 Although BD 21 (DMRB 3.4.3) specifies that assessments shall be carried out at the ultimate limit state, this Standard requires that serviceability limit state checks be carried out in a number of cases. However certain seviceability checks required by the design rules may be waived when permanent deformations are acceptable.

Fatigue

2.8 In assessment, fatigue analysis is not normally necessary. Where the configuration of the bridge is such that fatigue assessment is essential the loading and the method of analysis shall be as given in BS 5400 Part 10 as implemented by BD 9 (DMRB 1.3)

Condition Factor in BD 21

2.9 While the application of the condition factor F_c in paragraph 3.18 of BD 21 (DMRB 3.4.3) is not affected in principle by the requirements of this Standard, care should be taken to ensure that the estimated values of F_c do not allow for deficiencies of the materials in a structure which are separately allowed for by using the amended values of γ_m.

3. USE OF ANNEX A, BS 5400 PART 3 AND BD 13

3.1 Annex A is presented in the form of new clauses, amendments or add-ons to the design clauses in BS 5400 Part 3 as amended by BD 13 (DMRB 1.3). For the benefit of engineers existing clauses in BS 5400 Part 3 as amended by BD 13 (DMRB 1.3) have been included on the facing pages of the document. To facilitate this, the appropriate sections of BS 5400 Part 3 have been highlighted. The page numbers centred above the footer are the page numbers of BS 5400 Part 3.

3.2 Some clauses in BS 5400 Part 3 and Appendix A of BD 13 (DMRB 1.3) are expressed in a mandatory form using the word 'shall', whereas some other clauses are expressed in the form of recomendations using the word 'should'. However, even the latter requirements shall be considered as mandatory. In Annex A the word 'shall' is used throughout.

3.3 Where reference is made to any part of BS 5400, this shall be taken as a reference to that part as implemented by the Overseeing Organisation.

4. REFERENCES

1. Design Manual for Roads and Bridges (DMRB)

Volume 1: Section 3: General Design

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

BD 37 Loads for Highway Bridges. (DMRB 1.3)

BD 9 Implementation of BS 5400: Part 10: 1980, Code of Practice for Fatigue. (DMRB 1.3)

Volume 3: Section 4: Assessment

BD 21 The Assessment of Highway Bridges and Structures. (DMRB 3.4.3)

BA 56 The Assessment of Steel Highway Bridges and Structures. (DMRB 3.4.12)

BD 61 The Assessment of Composite Highway Bridges and Structures. (DMRB 3.4.16)

BA 61 The Assessment of Composite Highway Bridges and Structures. (DMRB 3.4.17)

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

Volume 1: Specification for Highway Works (MCHW 1)

3. British Standards Institution

BS 5400: Part 3: 1982. Steel, Concrete and Composite Bridges. Code of Practice for Design of Steel Bridges.

BS 5400: Part 10: 1980. Steel, Concrete and Composite Bridges. Code of Practice for Fatigue.

4. The Bridge Inspection Guide, 1984 (HMSO)

5. ENQUIRIES

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

The Chief Highway Engineer The Highways Agency St Christopher House Southwark Street London SE1 0TE	T A ROCHESTER Chief Highway Engineer

The Deputy Chief Engineer National Roads Directorate The Scottish Office Development Department Floor 2C, Victoria Quay Edinburgh EH6 6QQ	N B MACKENZIE Deputy Chief Engineer

The Director of Highways Welsh Office Crown Buildings Cathays Park Cardiff CF1 3NQ	K THOMAS Director of Highways

The Director of Roads Service Department of the Environment for Northern Ireland Roads Service Headquarters Clarence Court 10-18 Adelaide Street Belfast BT2 8GB	W J McCOUBREY Director of Roads Service

AMENDMENTS TO BS 5400 : PART 3 : 1982 AND APPENDIX A OF BD13/90

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Tables

Figures

Add after Clause G.18:

BS 5400 : Part 3 : 1982

Foreword

BS 5400 is a document combining codes of practice to cover the design and construction of steel, concrete and composite bridges and specifications for loads, materials and workmanship. It comprises the following Parts:

Part 1	General statement
Part 2	Specification for loads
Part 3	Code of practice for design of steel bridges
Part 4	Code of practice for design of concrete bridges
Part 5	Code of practice for design of composite bridges
Part 6	Specification for materials and workmanship, steel
Part 7	Specification for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 8	Recommendations for materials and workmanship, concrete, reinforcement and prestressing tendons
Part 9*	Bridge bearings Section 9.1 Code of practice for design of bridge bearings Section 9.2 Specification for materials, manufacture and installation of bridge bearings
Part 10	Code of practice for fatigue

In the drafting of BS 5400 important changes have been made in respect of loading and environmental assumptions, design philosophy, load factors, service stresses and structural analysis. Furthermore, recourse has been made to recent theoretical and experimental research and several design studies have been made on components and on complete bridges.

It is to be expected that as design experience of different bridge types is accumulated, further modifications will be required.

The relationship between Part 3 and Part 5. The design of composite bridges requires the combined use of Part 5 and Part 3 of BS 5400.

Part 5 was published in 1979, the major decisions on scope and approach having been taken some years previously. It is natural therefore that some differences will exist between Part 3 and Part 5.

Part 3 has been drafted on the assumption that for the design of steelwork in bridges with either steel or concrete decks the methods of global analysis and all the procedures for satisfying the limit state criteria will be as prescribed in this Part. For beams Part 3 may be used without any modification in conjunction with those provisions of Part 5 that are applicable to the properties of the composite slab and its connection to the steel section.

Part 5 also contains optional provisions for increased redistribution of longitudinal moments in compact members or for plastic analysis of continuous beams for the ultimate limit state, which could prove economical in some instances. These procedures require special consideration of increased transverse deformations of the slab, which is not covered in Part 5, and of stability of the bottom flange, which is not covered in Part 3: they should not be used unless proper account is taken of these considerations.

It will be noted that more serviceability checks are required for composite than for steel bridges. This difference is due to the special characteristics of composite construction, such as the large shape factor of certain composite sections; the addition of stresses in a two-phase structure (bare steel/wet concrete and composite); and the effects of shrinkage and temperature on the girders and on the shear connectors.

The method given in 4.1.3(a) of Part 5 should not be used, when the relationship between loading and load effects is non linear, and the values of γ_m for structural steel given in table 1 of Part 5 should not be used and reference made to table 2 of Part 3.

It is intended to revise Parts 3 and 5 to coordinate them fully after there has been sufficient experience of their application.

*In course of preparation.

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British Standard

Steel, concrete and composite bridges

Part 3. Code of practice for design of steel bridges

1. Scope

This Part of this British Standard gives recommendations for the design of structural steelwork in bridges.

After stating general recommendations, procedures are given for the design of steelwork components, assemblies and connections. Such procedures are applicable to steelwork which is to be fabricated and erected in accordance with Part 6. Recommendations for design against fatigue are contained in Part 10. Recommendations for design of concrete components and shear connectors for their interaction with steelwork are contained in Part 5. The partial factors of safety given are appropriate only for bridges designed to this standard.

Hybrid construction, using materials of different yield stress, is not generally within the scope of this Part.

Parapets, safety fences and other ancillary items are not within the scope of this Part.

2. References

The titles of the standards publications referred to in this Part of this standard are listed in the inside back cover.

3. Definitions and symbols

3.1 Definitions. For the purposes of this Part of this British Standard the definitions given in Part 1 apply. For the sake of clarity the factors which together comprise the partial safety factor for loads are restated as follows.

design loads. Design loads are the loads obtained by multiplying the nominal loads by γ_{fL}, the partial safety factor for loads. γ_{fL} is a function of two individual factors, γ_{f1} and γ_{f2}, which take account of the following:

γ_{f1}, the possible unfavourable deviations in load from those considered in deriving their nominal values

γ_{f2}, the reduced probability that, with combinations of load, the individual loads would all be at their nominal values.

The relevant values of the function γ_{fL} $(=\gamma_{f1}\gamma_{f2})$ are given in Part 2. The factor γ_{f3} takes account of inaccurate assessment of effects of loading, unforeseen stress distribution in the structure, variation in dimensional accuracy achieved in construction and the importance of the limit state being considered.

The values of γ_{f3} are given in 4.3.3.

3.2 Symbols

3.2.1 ***General.*** The symbols used in this Part of this British Standard are as given in 3.2.2 and 3.2.3. Main symbols are given in 3.2.2 and subscripts are given in 3.2.3. Symbols are further clarified, as appropriate, in the text. Some additional symbols are used in the appendices.

3.2.2 ***Main symbols***

A	Cross-sectional area
A_o	Area of box enclosed by perimeter
a	Length of panel; longitudinal spacing of transverse stiffeners
B	Overall width; spacing of beams
b	Width of panel; width of element; transverse spacing of longitudinal stiffeners
C	Weld shrinkage coefficient
C_v	Charpy energy absorption
c	Centres of bearings
D	Overall depth; overall diameter
d	Depth of element; diameter of element
E	Modulus of elasticity
e	Eccentricity; offset; exponential function = 2.7183
F	Internal force
F_p	Prestress force
f	Rotational flexibility; force per unit length or width
G	Shear modulus of elasticity
g	Throat thickness of weld; gauge of holes
H	Prying force
h	Height of element
I	Second moment of area
i	Factor, as defined in text
J	Torsional constant
j	Width of bearing pad contact
K	Buckling coefficient
k	Factor, as defined in text
L	Span; overall length
ℓ	Length of element
M	Moment
m	Ratio, as defined in text
N	Number of friction interfaces
n	Number, as defined in text
P	Applied force
p	Penetration of weld
Q	Shear force in diaphragms and battens
q	Shear flow
R	Reaction; radius of curvature
r	Radius of gyration
S	Shape factor
s	Spacing
T	Torque or torsional moment
t	Thickness of plate or section
U	Temperature
V	Shear force in webs

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BD 56/94 THE ASSESSMENT OF STEEL HIGHWAY BRIDGES AND STRUCTURES

ANNEX A AMENDMENTS TO BS 5400: PART 3: 1982 AND APPENDIX A OF BD 13/90

1. Scope

Delete the existing text and substitute the following:

This Standard shall be used for the assessment of steel highway bridges and their structural components. The assessment additions contained in this document extend the existing Code to cater for the majority of existing steel highway bridges.

In assessment, hybrid construction shall be dealt with by taking due account of the different levels of yield stress in all aspects of the assessment.

2. References

Add at end:

Additional references are given in Appendix Z of the accompanying Advice Note BA 56 (DMRB 3.4.11). These relate to specific references called up in the added text, as well as listing other references useful for the general interpretation of Part 3 in the context of assessment.

u, v	Factors, as defined in text
W	Total uniform load
w	Width of element
x	Distance longitudinally along member
y	Distance from neutral axis or centroid
Z	Section modulus
α	Ratio of stiffener/flange area
α_{LT}	Coefficient for torsional restraint
β	Slope of web to vertical
γ_{fL}, γ_{f3}	Partial load factors
$\gamma_m, \gamma_{m1}, \gamma_{m2}$	Partial material factors
Δ	Imperfection to be assumed
δ	Flexibility (deflection per unit force)
θ	Rotation (rad)
θ_d	Slope (degrees)
λ	Slenderness parameter
λ_{LT}	Slenderness parameter for lateral torsional buckling
μ	Slip factor
ν	Poisson's ratio
π	3.1416
ρ	Proportion, as defined in text
Σ	Sum
σ	Direct stress
σ_y	Nominal yield stress (N/mm^2)
τ	Shear stress
τ_o	Shear stress at onset of tension field action
τ_y	Shear yield stress
ϕ	Panel aspect ratio
ψ	Effective breadth ratio
η, ξ	Factors, as defined in text

3.2.3 *Subscripts*

a	axial
b	cross beams; bending; bearing; battens
B	box; beam
c	cantilever; compressive
d	diaphragm
D	design resistance
e	effective; equivalent
E	Euler buckling
f	flange
g	gusset
h	horizontal; hole
i	buckling; instability
j	compact member
k	stocky member
ℓ	limiting
n	integer value to be taken, 1 to n
o	stiffener outstand
p	plastic; lacing
q	shear
R	reference value
s	stiffener
t	tensile
T	torsion
u	U-frame
v	vertical
w	web; weld
x	about X-X axis
y	about Y-Y axis
z	centroid of plate
1	normal; longitudinal; primary
2	transverse; horizontal; secondary or 1 to 5 distinguishing subscript, as defined in text

4. Design objectives

4.1 General. The objectives of design shall be those stated in Part 1.

4.2 Limit states

4.2.1 *Ultimate limit state.* All structural steelwork should be checked for compliance with the recommendations of this Part in relation to the ultimate limit state.

4.2.2 *Serviceability limit state.* Structural steelwork should be considered to have reached the serviceability limit state if either:

(a) deformation has occurred in one or more components or connections such as to cause either excessive permanent deflection, or damage to finishes or to protective coatings; or

(b) buckling of one or more elements has occurred to such an extent that the maximum stress exceeds the yield stress, and excessive deformation occurs due to spread of plasticity.

Except where otherwise stated in this Part, structural steelwork may be deemed to satisfy the serviceability limit state if it has been designed in accordance with the provisions of this Part for the ultimate limit state.

A list of clauses requiring a separate check at the serviceability limit state is given in table 1.

Table 1. Clauses requiring serviceability check

Clause	Items and consideration requiring serviceability check
9.2.3.1	Flange panel when maximum longitudinal stress is more than 1.67 times the mean longitudinal stress
9.5.5	Whole beam cross section, without redistribution, when yielding of tension flange occurs at a lower loading than the buckling or yielding of the compression flange and redistribution of tension flange stresses is assumed for the ultimate limit state
9.9.8	Unsymmetric sections
9.10.3.3	Stiffened flanges subjected to local bending when local bending stresses are neglected for the ultimate limit state
12.2.3	Compression members in trusses that are not compact, or certain compression members having length to width ratios of less than 12 for chord members and less than 24 for web members
14.5.4.1.2	Connections made with HSFG bolts when the capacity used for the ultimate limit state is based on the shear or bearing capacities of the HSFG bolts and the lower of these is in excess of the friction capacity

Add new Clause 3.2.4:

3.2.4 Symbols used in assessment

The main symbols and subscripts used for assessment are generally in accordance with **3.2.3** and **3.2.4**. However, some additional symbols are needed for assessment which are defined in the text as they occur.

4.1 General

Add at end:

The loading for assessment of existing bridges shall be in accordance with BD 21 (DMRB 3.4.3).

The objectives and procedures for assessment of existing bridges shall be in accordance with BD 21 (DMRB 3.4.3). The compliance criteria for assessment of structural steelwork in existing bridges shall be in accordance with this part as supplemented by the clause additions for assessment. Where other documents are used for derivation of load effects, analysis or other objectives, the principles and requirements of this Part shall still be applied unless specifically stated otherwise. Further guidance notes on assessment objectives and procedures are given in the accompanying Advice Note BA 56 (DMRB 3.4.11).

Inspections for assessment shall follow the recommendations of Appendix I as well as "The Bridge Inspection Guide" (HMSO 1984).

4.2.2 Serviceability limit state

In Table 1, against 9.2.3.1 delete '1.67' and insert ' $1/\psi_R$*'.*

Add the following NOTE under Table 1:

NOTE 1. Clauses **14.2.3** and **14.5.4.1.2**
When calculated deflections due to bolt slip do not cause unserviceability as defined in **4.2.2**, the serviceability limit criterion need not apply.

NOTE 2. GENERAL
Further cases, where additional serviceability checks are required according to individual assessment circumstances, are set out in the relevant clauses.

4.2.3 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

4.3 Partial safety factors to be used

4.3.1 ***General.*** For a satisfactory design of the structure, the provisions given in Part 1 should be met using the format set out in **4.3.2** of this Part.

4.3.2 ***Safety factor format.*** Stresses shall be calculated from the effects of $\gamma_{fL} Q_k$.

The safety factor format to be used in applying this Part is:

$$(\text{the effects of } \gamma_{fL} Q_k) \leqslant \frac{1}{\gamma_{f3}\gamma_{m1}\gamma_{m2}}\left[\begin{pmatrix}\text{function } \sigma_y, \text{ and other}\\ \text{geometric variables}\end{pmatrix}\right]$$

where

γ_{fL} γ_{f3} and Q_k are as described in Part 1

γ_{m1} is the partial factor on the characteristic yield stress σ_y

γ_{m2} is the partial factor for modelling uncertainties and other variables in the formulae for design resistance.

4.3.3 ***Values of partial safety factors.*** The values of partial safety factors are as follows:

γ_{fL} the values of γ_{fL} are given in Part 2 for each type and combination of loading

γ_{f3} the factor γ_{f3} in this Part shall be taken as 1.1 for the ultimate limit state and 1.0 for the serviceability limit state

γ_{m1}, γ_{m2} for the sake of simplicity the expressions for strength in this Part contain a single factor $\gamma_m (= \gamma_{m1}\gamma_{m2})$. Values of the factor to be used where γ_m is explicitly shown in the design strength equations in this Part are given in table 2.

Table 2. Partial safety factors, $\gamma_m = \gamma_{m1}\gamma_{m2}$

(a) Ultimate limit state

Where explicitly expressed in a strength requirement in this Part, γ_m shall be taken as 1.05, except in the following clauses for which the appropriate value of γ_m is tabulated

Structural component and behaviour	Clauses	γ_m
Bending resistance of beams related to the limiting compressive stress except for a composite beam section where the steel and concrete composite flange is in compression for which γ_m=1.05	9.9.1.2, 9.9.1.3(a), 9.9.3.1, 9.9.5.3(a), 9.10.1.1(a)	1.20
Buckling resistance of stiffeners	9.10.2.3(a) and (b), 9.11.5.2, 9.13.5.3, 9.13.6, 9.14.4.3, 9.17.6.7, 9.17.7.3.2, 9.17.8	1.20
Fasteners in tension	14.5.3.2, 14.5.3.3, 14.5.3.5	1.20
Fasteners in shear	14.5.3.4	1.10
Friction capacity of HSFG bolts	14.5.4.2	1.30
Welds	14.6.3.11.1, 14.6.3.11.2, 14.6.3.11.3	1.20

(b) Serviceability limit state

Where explicitly expressed in a strength requirement in this Part, γ_m shall be taken as 1.00, except in the following clause for which the appropriate value of γ_m is tabulated.

Structural component and behaviour	Clause	γ_m
Friction capacity of HSFG bolts	14.5.4.2	1.20

NOTE. Any other clause making cross-reference to any of the above clauses should incorporate the appropriate γ_m factor tabulated.

4.3 Partial safety factors to be used

Add the following text under (a) in Table 2:

Structural Components and Behaviour	Clauses	γ_m
Compression members	10.6.1.1 10.6.3	$0.95 + \frac{1.8}{(L/r + 5)}$ but not greater than 1.05

4.3.3 Values of partial safety factors

Add at end:

Where alternative methods of calculating strength or resistance are used the value of resistance shall be taken as:

[the predicted resistance]/ $\gamma_m \gamma_{f3}$

where

γ_m in Table 2 is replaced by:

$$\gamma_m = (1.05 + 26.5\, m_{cv}^{\ 2})\, m_{mean}$$

m_{mean} = $m_{tests} + m_{st}.k$

m_{cv} = m_{st}/m_{tests}

m_{tests} = The mean value of the ratios for each test between the resistance predicted using the proposed method and the measured resistance

m_{st} = The standard deviation of the ratios for each test between the resistance predicted using the proposed method and the measured resistance

k = A correction factor obtained from Table 4.3A in which n is the number of tests.

n	**2***	**3***	**4***	**5**	**6**	**7**	**8**	**9**	**10**	**11**
k	4.47	1.69	1.18	0.95	0.82	0.73	0.67	0.62	0.58	0.55

n	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**
k	0.52	0.49	0.47	0.45	0.44	0.42	0.41	0.40	0.39	0.38

n	**22**	**23**	**24**	**25**	**31**	**41**	**61**	**121**	**∞**
k	0.37	0.36	0.35	0.34	0.31	0.26	0.21	0.15	0.00

NOTE: * The use of less than five tests is not recommended.

Table 4.3A. Sample standard deviation correction factor k

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4.4 Structural support. Provision should be made in the design for the transmission of vertical, longitudinal and lateral forces to the bearings and supporting structures.

4.5 Corrosion resistance and protection

4.5.1 ***General.*** The basis for the design of components contained in this Part makes no allowance for any loss of material due to corrosion. All steelwork should be designed and detailed to minimize the risk of corrosion.

All parts should be accessible for inspection, cleaning and painting, or should be effectively sealed against corrosion. Where these methods are not possible, either the surface of the steel should be given a system of protective coating selected with due regard to the design life of the part, together with an additional thickness of steel in accordance with 4.5.5.1, or the steel used should have corrosion resistant properties suitable for the design environment. Road decks, whether of steel or concrete, should be waterproofed and so designed as to protect supporting steelwork from corrosive attack by salt from the road surface.

4.5.2 ***Provision of drainage.*** Drainage should be provided wherever water may collect and should be designed to carry the water to a point clear of the underside of adjacent parts of the structure.

4.5.3 ***Sealing.*** Box members and other hollow sections without access for internal inspection and maintenance should be effectively sealed against corrosion (e.g. by continuous welding).

Box members and other hollow sections accessible for maintenance should be provided with internal protection against corrosion unless measures are taken to ensure that they are airtight.

Box members designed to be completely airtight should be checked for structural adequacy under internal and external pressure due to changes in the temperature of the air inside the enclosed space and to changes in the pressure of the external atmosphere.

4.5.4 ***Narrow gaps and spaces.*** To permit inspection and maintenance, the clear space between parts not in contact should not be less than one-sixth of the width of the face of the smaller part, or 10 mm, whichever is the greater. Alternatively, steel packing should be inserted to fill the space. Where this is impracticable then a sealant should be used.

4.5.5 ***Thickness of steel with inaccessible surfaces***

4.5.5.1 Where required by 4.5.1, inaccessible surfaces of steel should be provided with an extra thickness based on an estimate of probable corrosion. For a design life of 120 years, this provision may be considered to be met if the following extra thickness is provided at each inaccessible surface:

(a) at industrial or marine sites, 6 mm;
(b) at other inland sites, 4 mm;
(c) where free drainage cannot be ensured, 1 mm, in addition to the excess under (a) and (b).

NOTE. This provision need not apply to surfaces in contact, the edges of which are effectively sealed against corrosion.

4.5.5.2 Sealed hollow sections should not be less than 5 mm thick. For such components with a design life not greater than 50 years this value may be reduced to 4 mm.

4.6 Clearance gauges. Specified clearance gauges should be maintained without encroachment by any part of the structure under the action of load combination 1, specified in 4.4.1 of Part 2 of BS 5400: 1978, for the serviceability limit state.

5. Limitations on construction and workmanship

5.1 Workmanship. The design rules given in this Part are appropriate only to bridges fabricated and erected in accordance with Part 6.

5.2 Robustness. Components should be sufficiently robust to facilitate handling and prevent accidental damage in service. Consideration should also be given to the possibility of vibration due to aerodynamic excitation of exposed slender members.

5.3 Handling and transport. Components should be designed with due regard to the limitations on shop and site handling capacity and to transport restrictions on bulk and/or weight.

5.4 Composite steel/concrete construction. Where steel construction is used in conjunction with concrete, composite action may be assumed provided that the design is in accordance with Part 5.

5.5 Built-up members. The elements of any member built-up from parts should be joined by connections providing the rigidity assumed in design and sufficient to transmit all appropriate internal and external forces in accordance with the relevant clauses of this Part.

5.6 Diaphragms and fixings required during construction. When, in addition to any diaphragms required for the proper functioning of the completed structure, diaphragms, bracings, brackets and cleats are provided to facilitate fabrication, transport and erection, the effects of such components on the adequacy of the structure in service (particularly in relation to fatigue) should be considered.

5.7 Camber. The structure may need to be cambered in order to satisfy the provisions of 4.6 or to achieve a satisfactory appearance of the bridge. In this connection a sagging deflection of a nominally straight soffit of 1/800th of the span should not be exceeded. The cambered shape of the structure under the action of the actual dead and superimposed dead loads should be as specified or approved by the Engineer.

5.8 End connections of beams. Where the end of a beam is required to be free to rotate, due allowance for the movement should be made in the detailed design. Where the end of a beam is restrained by its connection to adjacent parts of the structure, account should be taken of the resulting moment when designing the end of the beam, the connection and the adjacent parts of the structure.

5.9 Support cross beams. Where a deck is supported on cross beams, no part of the deck should rest directly on a pier or abutment but should be supported by cross beams in the appropriate positions.

4.5.1 General

Add at end:

Where an existing bridge has inaccessible surfaces and does not comply with 4.5.5.1 or 4.5.5.2 as appropriate an assessment of any existing corrosion losses at the inaccessible surfaces shall be made in accordance with Appendix I and either allowance be made in strength assessment for existing and future losses in accordance with 8.7 or remedial action taken to reinforce the damaged part and to reliably protect it against corrosion.

No allowance need normally be made for corrosion in carrying out the global analysis to determine the moments and forces in the structures.

4.5.2 Provision of drainage

Add at end:

In assessment of existing sealed box members and other hollow sections checks shall be undertaken to determine whether water has collected in them. If necessary water shall be drained.

4.5.3 Sealing

The first two paragraphs are not applicable to assessment.

4.5.4 Narrow gaps and spaces

Not applicable to assessment.

5.1 Workmanship

Add at end:

In the assessment of existing structures allowance shall be made for geometric and other imperfections in accordance with 8.5.

5.2 Robustness

Not applicable to assessment.

5.4 Composite steel/concrete construction

Add at end:

In assessment of bridges of such construction composite action shall be assumed only when the strength of the shear connection between the materials complies with the relevant ultimate limit state provisions of BD 16 (DMRB 1.3) and the strength of the concrete parts using BD 44 (DMRB 3.4). No composite action shall be assumed when the details of shear connection are not known except as described in 8.8.

Where plastic methods of global analysis are used in assessment (see 7.4) of structures in which all or some of the members are of composite construction the assessor shall satisfy himself that the concrete has a sufficient strain plateau to permit the development and sustaining of yield line plastic moments in reinforced concrete slabs, or plastic moments in composite beams in which the concrete is in compression. Bridges constructed with concrete complying with the workmanship clauses of the Specification for Highway Works (MCHW 1) will satisfy this requirement.

5.5 Built-up members

Add at end:

In assessment of such members which do not comply with the design standard the connections of their elements shall comply with section 14.

5.6 Diaphragms and fixings required during construction

Add at end:

In assessment of existing structures the possible effects of any residual defect or other permanent change resulting from the removal of temporary attachments shall be taken into account.

5.7 Camber

Not applicable to assessment.

5.9 Support cross beams

Add at end:

Where in an existing bridge a deck is supported on cross beams and also directly on one or more supports at a pier or abutment due account shall be taken in the analysis of the total support system and any restraints which it provides.

6. Properties of materials

6.1 Performance. The mechanical properties of materials required by the Engineer are to be specified in accordance with Part 6.

6.2 Nominal yield stress. The nominal yield stress σ_y, for steel supplied to a standard grade complying with the requirements of BS EN 10 025 or BS 4360 and tested in accordance with those standards, should be taken as:

Steel grade in		Yield strength σ_y, for thickness (in mm)		
BS EN 10 025	BS 4360	up to and including 63	over 63 up to and including 100	over 100 up to and including 150
		N/mm²	N/mm²	N/mm²
Fe 360	40	225	215	195
Fe 430	43	265	245	225
Fe 510	50	355	325	295
	55	450	400	–
	WR 50	345		–

When steel to specifications other than BS EN 10 025 or BS 4360 is used the nominal yield stress should be taken as:

$$\left(1 - \frac{\rho_t}{100}\right)(\sigma_{ym} - k_2 \times \text{standard deviations from } \sigma_{ym})$$

where

ρ_t is the percentage tolerance below the specified thickness permitted by the relevant British Standard for material for the relevant thickness

σ_{ym} is the mean yield stress of material of the relevant thickness.

k_2 is the coefficient as given in Table 7 of BS 2846:Part 3:1975, using the confidence level $(1 - \alpha) = 0.95$ and the proportion of the population $P = 0.95$.

6.3 Ultimate tensile stress. The ultimate tensile stress of steel plates and sections of steel not complying with the requirements of BS EN 10 025 or BS 4360 should be not less than $1.4\sigma_y$ when $\sigma_y < 390$ N/mm², nor less than $1.2\sigma_y$ when $\sigma_y \geq 390$ N/mm²,

where

σ_y is the nominal yield stress of the material.

6.4 Ductility. Steel used in a bridge designed in accordance with this Part should have a ductility not less than that corresponding to an elongation of 15%, based on the standard gauge length of 200 mm. Steels complying with the requirements of BS EN 10 025 or BS 4360 satisfy this recommendation.

NOTE. If ductility is tested with other gauge lengths, the observed value should be converted to an equivalent ductility for a 200 mm gauge length in accordance with BS 3894.

Where use is made of plastic methods of design which permit redistribution of bending effects, the ductility of the steel used should not be less than that specified in BS EN 10 025 or BS 4360.

6.5 Notch toughness

6.5.1 ***General.*** Any part which is to be subjected to applied tensile stress either during erection or in service should be in accordance with the simple provisions of 6.5.4, or with the fuller energy absorption provisions of 6.5.5.

In applying either 6.5.4 or 6.5.5, the design minimum temperature, U (in °C), determined in accordance with 6.5.2, should be used, and the part should be classified in accordance with 6.5.3.

Special consideration is required where the geometrical shape is such as to induce large concentrations of stress (see 6.5.6).

6.5.2 ***Design minimum temperature.*** The design minimum temperature, U (in °C), to be used when applying either 6.5.4 or 6.5.5 should be as follows:

(a) in a part of which the primary function is to resist thermal movement, $U = U_e - 5$;

(b) in all other parts, $U = U_e$;

where

U_e is the minimum effective bridge temperature given in Part 2 (in °C).

6.5.3 ***Classification.*** Each part subjected to applied tensile stress should be classified as type 1 or type 2 as follows.

(a) *Type 1.* Any part which is subjected to applied principal tensile stress at the ultimate limit state (ignoring geometric stress concentrations) greater than 100 N/mm², and which, in addition, has either

(1) any welded connection or attachment; or

(2) welded repair of surface defects and has not been subsequently inspected by crack detection of at least a 10% random sample; or

(3) punched holes which have not been subsequently reamed.

(b) *Type 2.* All parts subjected to applied tensile stress and which are not of type 1.

6.5.4 ***Simple provisions.*** To simplify selection, limiting thicknesses for certain steels complying with the requirements of BS EN 10 025 or BS 4360, to give the required energy absorption, are given in table 3. For the use of greater thicknesses of these steels, the provisions of 6.5.5 should be met.

Unless dealt with in accordance with 6.5.5:

(a) no part of type 1 shall be thicker than the appropriate limit given in table 3;

(b) no part of type 2 shall be thicker than double the appropriate limit given in table 3 nor exceed the maximum thickness specified in BS EN 10 025 or BS 4360.

6.1 Performance.

Not applicable to assessment.

6.2 Nominal yield stress

Delete the contents of the existing Clause and substitute as follows:

The nominal yield stress for assessment of existing bridges shall be derived in accordance with Appendix H.

6.3 Ultimate tensile stress

Add at end:

The ultimate tensile stress of tension elements and their connections of steel not complying with BS EN 10 025, BS 4360, BS 15 or BS 968 shall be established from mill test certificates or by tests on samples of the materials of the elements. The values of the ultimate tensile stress for assessment of existing structures shall be derived from the test data as for the values of yield stress in accordance with Appendix H.

Where a plastic method of analysis is used in assessment in accordance with **7.4** the steel in the parts assumed to have plastic capacity shall either comply with BS EN 10 025, BS 4360, BS 15 or BS 968 or shall have ultimate tensile stress not less than 1.4 σ_y when σ_y < 390 N/mm^2 nor less than 1.2 σ_y when $\sigma_y \geq$ 390 N/mm^2

where σ_y is the nominal yield stress derived in accordance with Appendix H.

6.4 Ductility

Add at end:

Where a plastic method of analysis is used in accordance with **7.4** or where the plastic moment capacity of a compact section is utilised or redistribution of tension flange stresses is assumed, the ductility of the steel shall be not less than the equivalent to an elongation of 19% on a gauge length 5.65 $\sqrt{S_o}$, where S_o is the original cross sectional area of the test piece. In addition where a plastic method of analysis is used the strain at the ultimate tensile stress shall be at least 20 times the strain corresponding to the yield stress.

6.5.2 Design minimum temperature

Add at end:

For existing bridges the assessment minimum temperature shall be taken as the design minimum temperature.

6.5.4 Simple provisions

Add at end:

In assessment of existing bridges components of B.S. EN 10 025 or B.S. 4360 steels having thicknesses not greater than the limiting thicknesses given in Table 3 may be deemed to provide the required energy absorption for Type 1. The limiting thicknesses for Type 2 may be taken as double the thicknesses given in Table 3.

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6.5.5 *Energy absorption.* Unless the simple provisions of 6.5.4 are adopted, the energy value C_v for steel used to resist applied tensile stress should not be less than:

(a) 18 joules or

(b) for type 1, $\left(\frac{\sigma_y}{355}\right)\left(\frac{t}{2}\right)$ (in joules) when $\sigma_y \le 355$ N/mm²

$\left(\frac{\sigma_y}{355}\right)^2\left(\frac{t}{2}\right)$ (in joules) when $\sigma_y > 355$ N/mm²

for type 2, $\left(\frac{\sigma_y}{355}\right)\left(\frac{t}{4}\right)$ (in joules) when $\sigma_y \le 355$ N/mm²

$\left(\frac{\sigma_y}{355}\right)^2\left(\frac{t}{4}\right)$ (in joules) when $\sigma_y > 355$ N/mm²

whichever is the greater

where

C_v is the energy value in impact tests carried out at the design minimum temperature U (see 6.5.2) in accordance with BS EN 10 025 or BS 4360.

σ_y is the nominal yield stress appropriate to the thickness.

t is the thickness of the part (in mm).

6.5.6 ***Stress concentrations.*** The provisions of 6.5.4 and 6.5.5 do not apply where severe geometric stress concentrations occur. In such cases the material should be such that:

$$C_v \ge \frac{\sigma_y}{1000}\left(0.75 + \frac{\sigma}{\sigma_y}\,k\right)t \text{, but not less than 18 joules}$$

where

C_v, σ_y and t are as defined in 6.5.5

k is the stress concentration factor.

σ is the applied mean principal tensile stress at the ultimate limit state.

NOTE. The stress concentration factor is the ratio of the peak principal tensile stress to the mean principal tensile stress at any section. Typical stress concentration factors for openings and notches are given in Part 10. For example, at the junctions between plated diaphragms and corners of box girders, k may be taken as 3.5 for the adjacent flanges and webs. The influence of stress concentrations inherent in the make-up of welded joints, those around small holes and those for bolts or rivets may be considered to have been taken into account in 6.5.4 and 6.5.5.

Table 3. Limiting thickness of certain steels, complying with the requirements of BS EN 10 025 or BS 4360, for parts in tension.

(a) Plates, strip and wide flats.

Grade in BS EN 10 025 and BS 4360	U=0°C	U=-10°C	U=-20°C	U=-30°C	U=-40°C	U=-50°C
	Limiting thickness					
	mm	mm	mm	mm	mm	mm
Fe360B, Fe430B	0	0	0	0	0	0
Fe360C, Fe430C	75	45	0	0	0	0
Fe360D1, Fe360D2 Fe430D1, Fe430D2	150	125	75	45	0	0
40EE, 43EE	75*	75*	75*	75*	75*	75*
Fe510B	0	0	0	0	0	0
Fe510C	55	35	0	0	0	0
Fe510D1, Fe510D2	130	85	55	35	0	0
Fe510DD1, Fe510DD2	150	130	85	55	35	0
50EE	75*	75*	75*	75*	75*	55*
50F	40	40	40	40	40	40
55C	25	20	0	0	0	0
55EE	63**	63**	63**	63**	50**	35**
55F	40	40	40	40	40	40
WR50A	12	12	0	0	0	0
WR50B	50	35	0	0	0	0
WR50C	50	50	45	35 (U = -25°C)	0	0

* 50 mm for wide flats
** 30 mm for wide flats

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6.5.5 Energy Absorption

Add at end:

When Charpy tests have been undertaken at test temperature U_t differing from the assessment minimum temperature (U) the results shall be extrapolated using the temperature-impact-value relation in Note 4 to Table 3, where T is redefined as the appropriate temperature for the extrapolation, subject to the following limitations:

(a) linear interpolation may be used within the table,

(b) maximum effective C_v is 67 joules,

(c) If U_t-U exceeds 10°C, C_v is 0 joules,

(d) U-U_t not to exceed 30 C.

Table 3 Limiting thickness of certain steels, complying with the requirements of BS EN 10 025 or BS 4360, for parts in tension

(a) Plates, strip and wide flats

Delete '*' appended to the limiting thickness for grade 50EE and insert '**'

Delete '**' appended to the limiting thickness for grade 55EE and insert '*'.

Add new clause 6.5.7

6.5.7 Assessment of notch toughness

Where in the assessment of the adequacy of the bridge either the tensile components do not satisfy the provisions of **6.5.4** or the impact energy absorption values for the tensile components are unknown the notch toughness of the material may be determined by testing samples taken from non-critical parts of the components and compliance demonstrated with **6.5.5** or **6.5.6** as appropriate.

Where the notch toughness of the steel does not meet the requirements of **6.5.5** or **6.5.6** the service and loading history of the bridge may be taken into consideration with the agreement of the Overseeing Organisation. Further guidelines are given in the accompanying Advice Note.

(b) Sections (other than hollow sections)

Grade in BS EN 10 025 and BS 4360	U=0°C	U=-10°C	U=-20°C	U=-30°C	U=-40°C	U=-50°C
	Limiting thickness					
	mm	mm	mm	mm	mm	mm
Fe360B, Fe430B	0	0	0	0	0	0
Fe360C, Fe430C	75	45	0	0	0	0
Fe360D1, Fe360D2 Fe430D1, Fe430D2	100	100	75	45	0	0
40DD, 43DD	100	100	100	75	45	0
Fe510B	0	0	0	0	0	0
Fe510C	55	35	0	0	0	0
Fe510D1, Fe510D2	100	85	55	35	0	0
Fe510DD1, Fe510DD2	100	100	85	55	35	0
50E	100	100	100	85	55	35
55C	19	19	0	0	0	0
WR50A	12	12	0	0	0	0
WR50B	50	35	0	0	0	0
WR50C	50	50	45	35 (U = -25°C)	0	0

(c) Hollow sections

Grade in BS 4360	U=0°C	U=-10°C	U=-20°C	U=-30°C	U=-40°C	U=-50°C
	Limiting thickness					
	mm	mm	mm	mm	mm	mm
43C	40	40	0	0	0	0
43D	40	40	40	40	0	0
43EE	40	40	40	40	40	40
50C	40	35	0	0	0	0
50D	40	40	40	35	0	0
50EE	40	40	40	40	40	40
55C	25	20	0	0	0	0
55EE	25	25	25	25	25	25
55F	25	25	25	25	25	25
WR50A	12	12	0	0	0	0
WR50B	40	35	0	0	0	0
WR50C	40	40	40	35 (U = -25°C)	0	0

NOTE 1. All thicknesses given are for type 1 parts. Thicknesses may be doubled for type 2 parts but should not exceed the maximum thickness specified in BS EN 10 025 or BS 4360.

NOTE 2. Interpolation for limiting thicknesses for intermediate temperatures is permitted between data in adjacent columns except where one of the limiting thicknesses is shown as zero, then the use of that grade of material for the intermediate temperature is not permitted.

NOTE 3. Some of the thicknesses given are the limits set by the maximum thickness specified in BS EN 10 025 or BS 4360. In the case of sections, for which the maximum thicknesses for some grades are not specified, they are taken as those for plates. The option in BS 4360 for specifying the impact value for hollow section of grade 43C should be adopted.

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NOTE 4. Limiting thicknesses given are derived using 6.5.5 for type 1 parts and the following impact values:

Temperature (°C)	Impact Value (joules)
T + 30	67
T + 20	54
T + 10	40
T	27
T - 10	18

Where T is the test temperature given in BS EN 10 025 or BS 4360 for impact value of 27 joules, except for grades Fe510DD1 and Fe510DD2, where T is taken as -30°C.

6.6 Properties of steel. The following properties of steel should be assumed in design:

modulus of elasticity, $E = 205\,000$ N/mm^2
shear modulus, $G = 80\,000$ N/mm^2
Poisson's ratio, $\nu = 0.3$
coefficient of thermal expansion $= 12 \times 10^{-6}$/°C.

6.7 Modular ratio. For global analysis of bridges of composite construction the modular ratio may be based on the long term value of the elastic modulus for concrete unless stated otherwise in this Part. For stress analysis the modular ratio appropriate to the stage of construction and the type of loading should be adopted.

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7. Global analysis for load effects

7.1 General. The global analysis of the structure should be in accordance with Part 1 using an elastic method of analysis. For structures in which the load effects are not proportional to the loads and/or the secondary effects due to deformation are significant, the method of analysis should be suitable for treatment of non-linear behaviour.

7.2 Sectional properties. The sectional properties to be used in global analysis should generally be calculated for the gross section assuming the specified sizes. For beams or trusses on flexible supports or cable stayed, account should, however, be taken of the influence of shear lag on their stiffnesses. The effect of shear lag should also be taken into account in analysis of conditions during erection of continuous girders of box construction or with integral decks.

8. Stress analysis

8.1 Longitudinal stresses in beams. The distribution of longitudinal stress between the flanges and web or webs of a beam may be calculated on the assumption that plane sections remain plane, but using the effective widths of flanges and the effective thickness of a deep web in accordance with 8.2 and 9.4.2.5, respectively. No further account need be taken of deformation of plating out of its plane. In composite construction the area of concrete in a tensile zone shall be ignored (see 9.4.2.7).

Subject to the provisions of 9.5.4, plate panels in webs may be assumed to shed a proportion of their longitudinal stresses to the flanges.

8.2 Allowance for shear lag. Where, in order to meet the provisions of 9.2.3, the effect of in-plane shear flexibility (i.e. shear lag) is to be allowed for in calculating the stress in a flange, an equivalent flange may be assumed to have an effective breadth equal to the sum of the effective breadths of the portions of flange on each side of the web. The effective breadth, b_e, of each portion should be taken as:

(a) ψb for portions between webs

where

b is half the distance between centres of webs, measured along the mid-plane of the flange plate;

(b) $k\psi b$ for portions projecting beyond an outer web

where

b is the distance from the free edge of the projecting portion to the centre of the outer web, measured along the mid-plane of the flange plate

$k = (1 - 0.15b/L)$

and where in (a) and (b) and in tables 4 to 7,

ψ is the appropriate effective breadth ratio taken from tables 4, 5, 6 or 7 for uniformly distributed loads and which should be used for standard highway or railway loading as specified in Part 2, including wheel and axle loads

L is the span of a beam between centres of support, or in the case of a cantilever beam, between the support and the free end

α = 0 if there are no stiffeners on the flange within the width b in the span direction, otherwise

$$\alpha = \frac{\text{sectional area of flange stiffeners in width } b}{\text{sectional area of flange plate in width } b}$$

Values of ψ for intermediate values of b/L and α and for intermediate positions in the span may be obtained by linear interpolation.

The value of ψ at an interior support should be taken as the mean of the values obtained for adjacent spans.

For end spans of continuous beams the effective breadth ratios may be obtained by treating the end span as a propped cantilever of the same span.

For the purpose of calculating deflections of beams the values of ψ given in tables 4, 5, 6 and 7 and in appendix A for the quarter-span sections may be adopted for all sections in the span.

For intermediate structures, for point loads and for combinations of point and distributed loads, not specifically covered above, the effective breadth ratios ψ may be determined by the methods given in appendix A.

Table 4. Effective breadth ratio ψ for simply supported beams

$\frac{b}{L}$	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.98	0.97	0.98	0.96	0.84	0.77
0.10	0.95	0.89	0.93	0.86	0.70	0.60
0.20	0.81	0.67	0.77	0.62	0.52	0.38
0.30	0.66	0.47	0.61	0.44	0.40	0.28
0.40	0.50	0.35	0.46	0.32	0.32	0.22
0.50	0.38	0.28	0.36	0.25	0.27	0.18
0.75	0.22	0.17	0.20	0.16	0.17	0.12
1.00	0.16	0.12	0.15	0.11	0.12	0.09

Table 5. Effective breadth ratio ψ for interior spans of continuous beams

$\frac{b}{L}$	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.96	0.91	0.85	0.76	0.58	0.50
0.10	0.86	0.72	0.68	0.55	0.41	0.32
0.20	0.58	0.40	0.42	0.31	0.24	0.17
0.30	0.38	0.27	0.30	0.20	0.15	0.11
0.40	0.24	0.18	0.21	0.14	0.12	0.08
0.50	0.20	0.14	0.16	0.11	0.11	0.07
0.75	0.15	0.10	0.10	0.08	0.09	0.06
1.00	0.13	0.09	0.09	0.07	0.07	0.05

7.1 General

Add at end:

Alternatively, a plastic method of analysis may be used in the assessment of beams in accordance with **7.4**.

7.2 Sectional properties

Add at end:

When in assessment of existing bridges allowance is to be made for unintended composite action where permitted in **8.8** the appropriate composite properties shall be used in global analysis.

Add new Clause 7.3:

7.3 Construction in stages

When in assessment of existing bridges the actual construction sequence is known, that sequence shall be used in the analysis. When the construction sequence is not known or is uncertain, a practicable sequence shall be assumed which leads to maximum effects in the structural element being considered. More than one such sequence may be required for a bridge, each appropriate to different groups of structural elements.

Add new Clause 7.4:

7.4 Plastic methods of analysis

If a plastic method is employed it shall take account of all parts of the structure which can participate in the global response and shall be able to follow the progressive development of plastic hinges (in parts which are essentially linear in configuration) and of yield lines (in parts which are planar, including reinforced concrete slabs in composite structures). The method shall generally be in accordance with **8.2.1** in Part 1.

Additionally, a plastic method of analysis shall only be used if:

(a) The steel materials satisfy the appropriate requirements of **6.3** and **6.4**.

(b) The member cross sections satisfy the requirements of **9.3.8**.

(c) The structure is assessed in addition for the serviceability limit state using an elastic method of analysis.

(d) The assessment of supports, supporting structures, webs and connections is based on the most onerous of the load effects derived from plastic and elastic analysis respectively. Where a plastic method is used consideration shall be given to all adverse patterns of loading and potential failure mechanisms to determine those providing the least safety margins.

(e) Lateral restraint to plastic hinges is provided as required in **9.12.5**.

(f) Slenderness of members satisfy the requirements of **9.7.6**.

Table 6. Effective breadth ratio ψ for propped cantilever beams

$\frac{b}{L}$	Fixed end		Quarter-span near fixed end		Propped end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.62	0.54	1.00	1.00	0.79	0.70
0.10	0.45	0.38	1.00	1.00	0.63	0.52
0.20	0.27	0.21	0.92	0.76	0.44	0.32
0.30	0.18	0.14	0.72	0.53	0.33	0.23
0.40	0.13	0.10	0.46	0.35	0.24	0.16
0.50	0.11	0.08	0.31	0.25	0.19	0.13
0.75	0.10	0.07	0.21	0.16	0.12	0.09
1.00	0.09	0.06	0.19	0.15	0.08	0.07

Table 7. Effective breadth ratio ψ for cantilever beams

$\frac{b}{L}$	Fixed end		Quarter-span near fixed end		Free end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.82	0.76	1.00	1.00	0.92	0.86
0.10	0.68	0.61	1.00	1.00	0.84	0.77
0.20	0.52	0.44	1.00	1.00	0.70	0.60
0.30	0.42	0.35	0.95	0.90	0.60	0.48
0.40	0.35	0.28	0.88	0.75	0.52	0.38
0.50	0.30	0.25	0.76	0.62	0.40	0.33
0.75	0.22	0.18	0.52	0.38	0.34	0.23
1.00	0.18	0.14	0.38	0.27	0.27	0.18

8.3 Distortion and warping stresses in box girders. Stresses in a box girder due to transverse bending of the walls of the box and torsional and distortional warping, should, where required by **9.2.1.2**, be calculated by linear elastic analysis. The simplified procedure given in appendix B may be used for highway bridges designed for loading in accordance with Part 2.

8.4 Shear stresses. The design values of shear stress in webs of rolled or fabricated I, box or channel sections may be calculated in accordance with **9.5.1**. Shear stresses in other sections should be computed from the whole cross section having regard to the distribution of flexural stress across the section.

8.5 Imperfections

8.5.1 *Imperfections allowed for.* The design strengths given in this Part may be assumed to have made allowance for the following tolerances where these apply:

(a) bearing misalignment in plan, errors in level of single bearing or in the mean level of more than one bearing at any support, and bearing inclination within the tolerances given in Part 9*;

(b) imperfect flatness and straightness of compression members and of stiffened and unstiffened plate panels within the tolerances given in Part 6.

8.5.2 *Imperfections to be allowed for separately*

8.5.2.1 *Torsionally stiff girders.* Imperfections in common planarity of bearings should be allowed for in the analysis of torsional moments and reactions for torsionally stiff superstructures and should be compatible with tolerances, which the designer should specify, and the construction method and sequence used.

8.5.2.2 *Columns.* Where a column is supported on a rocker bearing, the following eccentricities should be added to any calculated eccentricity of the bearing reaction:

(a) For a flat-topped rocker bearing in contact with flat bearing surface beneath column: half the width of the flat bearing surface plus 10 mm;

(b) For a radiused rocker bearing: 3 mm if the bearing is attached to the end of the column during fabrication in a position that is nominally central and 10 mm in all other cases.

8.6 Residual stresses. The design strengths given in this Part may be deemed to allow for residual stresses due to rolling, handling and transportation, and from those arising from normal welding procedures.

In order to make allowance for the relaxation of residual stresses due to welding when estimating deflections during erection, or on first loading, the effective area A_e of a flange in tension should, for this purpose, be taken as:

$$A_e = A_f\left(1 - \frac{CA_w}{\sigma_y A_f}\right)$$

where

A_f is the gross-sectional area of flange in tension, inclusive of longitudinal stiffeners

A_w is the volume of longitudinal weld per unit length of flange

C is a weld shrinkage coefficient and may be taken as 7000 N/mm^2

σ_y is the nominal yield stress of the flange material.

9. Design of beams

9.1 General. Beams are defined as members with solid webs (or with openings in accordance with **9.3.3**), subjected primarily to bending, including members of rolled and hollow section, plate girders and box girders.

9.2 Limit states

9.2.1 *Ultimate limit state*

9.2.1.1 *General.* Beams should be designed to satisfy the provisions of clause **9** for the ultimate limit state.

9.2.1.2 *Effects to be considered.* The effects at the ultimate limit state should be obtained for relevant combinations of:

(a) flexure, shear and torsion due to any loads transverse to the longitudinal axis of the member;

(b) the effects of axial load;

(c) creep, shrinkage and differential temperature (see Part 5 for composite structures);

(d) settlement of supports.

* In course of preparation.

8.3 Distortion and warping stresses in box girders

Add at end:

Where the stiffness does not comply with such requirements (of Appendix B) distortional and warping stresses shall be calculated (where required by **9.2.1.2**) by means of analysis of a finite element plate model of the box girder and its diaphragms of sufficient extent to ensure that the effects calculated are insensitive to assumed end conditions.

8.5.1 Imperfections allowed for

Add at end:

For bridges which do not comply with the specification requirements of Parts 6 and 9, bearing misalignment, errors in level, bearing inclination, and imperfections in flatness and straightness shall be determined by inspections when required and as described in Appendix I and taken into account in strength assessments. Where these are within the tolerances set in Part 6 or Part 9 as appropriate, design strengths given in Part 3 may be used in assessment.

For parts having measured imperfections beyond the above tolerances their magnitude shall be taken into account in strength assessment where explicit provision is made in the assessment additions for doing so or their strength and stiffness shall be assumed to be zero where explicit provision is not made in the assessment additions. Alternatively remedial measures shall be taken. Values of imperfections less than the tolerances may be taken into account when this is significantly beneficial.

Where the imperfections are to be taken into account in assessment, they shall be assumed to be 1.2 times the measured imperfections to allow for inaccuracies of measurement. This factor of 1.2 is embodied into the relevant assessment additions, and should only be varied with the agreement of the Overseeing Organisation where the nature of the survey so warrants.

8.5.2.1 Torsionally stiff girders

Add at end:

For assessments imperfections in common planarity of bearings shall be assumed to be 1.2 times the tolerances specified for a bridge or 1.2 times the imperfections recorded in as-built information. In the absence of such specification or records the imperfections shall be measured during preliminary inspections as described in Appendix I and adopted in analysis of load effects. The factor of 1.2 is embodied into the relevant assessment additions, and should only be varied with the agreement of the Overseeing Organisation where the nature of the survey so warrants.

8.5.2.2 Columns

Add at end:

For assessment all eccentricities of rocker bearings to the axes of columns shall be measured during detailed inspections as described in Appendix I and the measured values allowed for in assessments of column strengths.

Add new Clause 8.5.2.3:

8.5.2.3 Other imperfections

Where inspection reveals detrimental imperfections or effects other than those described above then due allowance shall be made in the calculations for assessment in accordance with **8.5.1** and Appendix I.

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Table 6. Effective breadth ratio ψ for propped cantilever beams

$\frac{b}{L}$	Fixed end		Quarter-span near fixed end		Propped end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.62	0.54	1.00	1.00	0.79	0.70
0.10	0.45	0.38	1.00	1.00	0.63	0.52
0.20	0.27	0.21	0.92	0.76	0.44	0.32
0.30	0.18	0.14	0.72	0.53	0.33	0.23
0.40	0.13	0.10	0.46	0.35	0.24	0.16
0.50	0.11	0.08	0.31	0.25	0.19	0.13
0.75	0.10	0.07	0.21	0.16	0.12	0.09
1.00	0.09	0.06	0.19	0.15	0.08	0.07

Table 7. Effective breadth ratio ψ for cantilever beams

$\frac{b}{L}$	Fixed end		Quarter-span near fixed end		Free end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.82	0.76	1.00	1.00	0.92	0.86
0.10	0.68	0.61	1.00	1.00	0.84	0.77
0.20	0.52	0.44	1.00	1.00	0.70	0.60
0.30	0.42	0.35	0.95	0.90	0.60	0.48
0.40	0.35	0.28	0.88	0.75	0.52	0.38
0.50	0.30	0.25	0.76	0.62	0.40	0.33
0.75	0.22	0.18	0.52	0.38	0.34	0.23
1.00	0.18	0.14	0.38	0.27	0.27	0.18

8.3 Distortion and warping stresses in box girders. Stresses in a box girder due to transverse bending of the walls of the box and torsional and distortional warping, should, where required by 9.2.1.2, be calculated by linear elastic analysis. The simplified procedure given in appendix B may be used for highway bridges designed for loading in accordance with Part 2.

8.4 Shear stresses. The design values of shear stress in webs of rolled or fabricated I, box or channel sections may be calculated in accordance with **9.5.1**. Shear stresses in other sections should be computed from the whole cross section having regard to the distribution of flexural stress across the section.

8.5 Imperfections

8.5.1 ***Imperfections allowed for.*** The design strengths given in this Part may be assumed to have made allowance for the following tolerances where these apply:

(a) bearing misalignment in plan, errors in level of single bearing or in the mean level of more than one bearing at any support, and bearing inclination within the tolerances given in Part 9*;

(b) imperfect flatness and straightness of compression members and of stiffened and unstiffened plate panels within the tolerances given in Part 6.

8.5.2 ***Imperfections to be allowed for separately***

8.5.2.1 *Torsionally stiff girders.* Imperfections in common planarity of bearings should be allowed for in the analysis of torsional moments and reactions for torsionally stiff superstructures and should be compatible with tolerances, which the designer should specify, and the construction method and sequence used.

8.5.2.2 *Columns.* Where a column is supported on a rocker bearing, the following eccentricities should be added to any calculated eccentricity of the bearing reaction:

(a) For a flat-topped rocker bearing in contact with flat bearing surface beneath column: half the width of the flat bearing surface plus 10 mm;

(b) For a radiused rocker bearing: 3 mm if the bearing is attached to the end of the column during fabrication in a position that is nominally central and 10 mm in all other cases.

8.6 Residual stresses. The design strengths given in this Part may be deemed to allow for residual stresses due to rolling, handling and transportation, and from those arising from normal welding procedures.

In order to make allowance for the relaxation of residual stresses due to welding when estimating deflections during erection, or on first loading, the effective area A_e of a flange in tension should, for this purpose, be taken as:

$$A_e = A_t\left(1 - \frac{CA_w}{\sigma_y A_t}\right)$$

where

A_t is the gross-sectional area of flange in tension, inclusive of longitudinal stiffeners

A_w is the volume of longitudinal weld per unit length of flange

C is a weld shrinkage coefficient and may be taken as 7000 N/mm^2

σ_y is the nominal yield stress of the flange material.

9. Design of beams

9.1 General. Beams are defined as members with solid webs (or with openings in accordance with 9.3.3), subjected primarily to bending, including members of rolled and hollow section, plate girders and box girders.

9.2 Limit states

9.2.1 ***Ultimate limit state***

9.2.1.1 *General.* Beams should be designed to satisfy the provisions of clause 9 for the ultimate limit state.

9.2.1.2 *Effects to be considered.* The effects at the ultimate limit state should be obtained for relevant combinations of:

(a) flexure, shear and torsion due to any loads transverse to the longitudinal axis of the member;

(b) the effects of axial load;

(c) creep, shrinkage and differential temperature (see Part 5 for composite structures);

(d) settlement of supports.

* In course of preparation.

Add new Clause 8.7:

8.7 Variations in structural dimensions

Measured section sizes shall be used in assessment of strength of all critical sections, see Appendix I. Due account shall be taken of any existing or projected future losses of section due to corrosion in accordance with **4.5.5**. No changes are to be made to partial safety factors when using the measured dimensions.

Add new Clause 8.8:

8.8 Originally unintended composite action

8.8.1 General

Stiffnesses and strengths calculated for steel sections not originally intended as acting compositely can be enhanced by consideration of composite action with adjacent or surrounding structure with appropriate reference to Part 5 where conditions are achieved as given in **8.8.2** or **8.8.3**.

8.8.2 Cased beams or filler beams or jack arch decks

For cased beams and concrete filler beams the stress analysis shall be based on composite properties to Clauses **8.1, 8.2, 8.3, 8.4, 8.5.1** and **8.5.2** of Part 5 where there is no evidence of excessive corrosion, fretting action or cracking sufficient to adversely affect the achievement of composite action. Sections can be assumed to be compact where the compression flange and webs are cased on both sides, and any uncased section remaining is also compact. Where the requirements for resistance to longitudinal shear are not met then the beams shall be assessed on the basis of the properties of the steel section only, and may be assumed to be compact carrying the entire load.

Alternatively where attachments to the beams are sufficent to prevent relative longitudinal slip (such as rivet or bolt heads or other transverse elements) as demonstrated by push-out tests or by appropriate evidence then these may be assumed to transmit the longitudinal shear forces. For dense brickwork filler beam or jack arch decks global and stress analysis shall be based on composite properties provided that the bending resistance of the composite section shall not be taken as greater than 30% in excess of the calculated resistance of the steel section alone.

8.8.3 Concrete slab and steel beam decks

For concrete slab and steel beam decks global and stress analysis using composite properties can be used provided:

(a) At the steel/concrete interfaces there is no evidence of corrosion, fretting action, relative longitudinal slip or cracking sufficient to affect the required composite action.

(b) The attachments to the beams at the interface are sufficient to prevent relative slip (such as rivet or bolt heads or other transverse elements) as demonstrated by push-out tests or by appropriate evidence.

(c) Appropriate site testing is carried out to demonstrate that the live load : stiffness relationship is supportive of the composite action achieved, where the amount of composite action is required to increase the design strength by 30% or more. Normally loading approximately equivalent to the nominal calculated live load capacity of the steel section only shall be applied.

9.2.1.2 Effects to be considered

Add at end:

(e) the effects of restraint of distortional warping and transverse distortional bending.

9.2.1.3 *Effects that may be neglected.* The effects of shear lag and restraint of torsional and distortional warping may be neglected for the ultimate limit state.

Items (c) and (d) of **9.2.1.2** may be neglected at the ultimate limit state provided that:

(a) the section is compact throughout the span being considered in accordance with the provisions of **9.3.7**; and

(b) the member is not prone to lateral instability; this may be deemed to be satisfied when the slenderness parameter, λ_{LT} is less than $45\sqrt{355/\sigma_y}$ (see **9.7**).

9.2.2. ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

9.2.3 ***Serviceability limit state***

9.2.3.1 *General.* The serviceability limit state provisions should additionally be met where called for by the following.

(a) When the maximum longitudinal stress is more than 1.67 times the average longitudinal stress in the particular flange segment between adjacent webs, or in the overhang from an outside web as appropriate. For this purpose, a 'flange segment' is the whole flange width between adjacent webs, or the whole overhang, whether or not stiffened by longitudinal stiffeners.

NOTE. For plate girder construction this criterion may be deemed to be satisfied if the shear lag effective breadth ratio ψ, determined in accordance with 8.2, is greater than 0.6.

(b) When redistribution of stresses from the tension flange is made in accordance with **9.5.5**.

(c) When stiffened flanges are subjected to bending by local loads (see **9.10.3.3**).

(d) When required in **9.9.8** for unsymmetric beams.

9.2.3.2 *Effects to be considered.* Stresses at the serviceability limit state should be obtained for the relevant combinations given in 9.2.1.2 (a) to (d) together with:

(a) the effects of shear lag;

(b) the effects of restraint of torsional and distortional warping in a girder.

9.2.4 ***Composite beams.*** In the design of composite beams, the concrete, reinforcement and shear connectors should satisfy the limit state requirements of Parts 4 and 5.

9.3 Shape limitations

9.3.1 General. Figure 1 sets out the geometric notation used throughout this section.

In applying 9.3.2, 9.3.4 and 9.3.6 to sections that are not fully stressed by the applied loading, the nominal yield stress values σ_{ys} or σ_y may be taken as the lesser of: the nominal yield stress of the material or 1.5 times the maximum stress at the appropriate section calculated for the factored loading at the ultimate limit state condition,

where

σ_{ys} relates to the stiffener

σ_y relates to the flange, the web or the circular hollow section, as appropriate.

9.3.2 ***Flanges***

9.3.2.1 *Outstands in compression.* Unless a free edge of a plate or other outstand is stiffened, the ratio b_{fo}/t_{fo} should not exceed $12\sqrt{355/\sigma_y}$ or 16 whichever is the lesser.

When the edge of the outstand is stiffened the ratio b_{fo}/t_{fo} should not exceed $14\sqrt{355/\sigma_y}$

where

b_{fo} is the width of the outstand measured from the edge to the nearest line of rivets or bolts connecting it to the supporting part of the member, or to the toe of a root fillet of a rolled section, or, in the case of a welded construction, to the surface of the supporting part of the member, or, in the case of composite construction, to the outer line of shear connectors

t_{fo} is the mean thickness of the outstand

σ_y is as defined in 9.3.1.

9.3.2.2 *Outstands in tension.* The ratio b_{fo}/t_{fo} should not exceed 16, where b_{fo} and t_{fo} are as defined in **9.3.2.1**.

NOTE. Where a flange consists of several flange plates built-up and connected to each other only by welds at their edges, an outer flange plate should not be thicker than an inner plate and the above provision should be satisfied for all the flange plates. For the flange plate connected to the web, b_{fo} should be taken as given above, but for all the other flange plates b_{fo} should be taken as half the width between the welds connecting it to the adjacent inner plate.

9.3.3 ***Openings***

9.3.3.1 *General.* Any openings in webs or compression flanges should be framed and the stiffened section designed for local load effects, including secondary bending. Alternatively, openings in webs may be unstiffened provided that they meet the provisions of **9.3.3.2**.

All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.

9.3.3.2 *Unstiffened openings in webs.* Openings in a web may be unstiffened provided that:

(a) the overall greatest internal dimension does not exceed one-tenth the depth of the web, nor for longitudinally stiffened webs, one-third the depth of the panel containing the opening;

(b) the longitudinal distance between boundaries of adjacent openings is at least three times the maximum internal dimension;

(c) not more than one opening is provided at any cross section

Cut outs provided for transverse stiffeners should either have at least one-third of their perimeters welded to the stiffeners, or the stiffeners should be cleated to the web with at least two bolts or rivets per side of the connection or by full perimeter welding of the cleat.

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9.2.1.3 effects that shall be neglected.

In line 2, delete 'and distortional'.

9.2.3.1 General:

In item (a), delete '1.67' *and substitute* '*1/* ψ_R'

In item (a), delete NOTE and content.

Add the following at the end of item (a):

ψ_R is the restricting shear lag factor dependent on the governing load case, and the adopted partial factors of safety on the governing loads.

ψ_R shall be derived from:

$$\psi_R = \frac{1}{r_{f3} . r_m . r_{fL}}$$

Where

$$r_{f3} = \frac{\gamma_{f3ULS}}{\gamma_{f3SLS}}$$

$$r_m = \frac{\gamma_{m\ ULS}}{\gamma_{m\ SLS}}$$

$$r_{fl} = \frac{\left(\gamma_{fLDEAD}\alpha_{DEAD} + \gamma_{fLSUP} + \alpha_{SUP} + \gamma_{fLLIVE}\alpha_{LIVE} + \gamma_{fLO}\alpha_o\right)_{ULS}}{\left(\gamma_{fLDEAD}\alpha_{DEAD} + \gamma_{fLSUP} + \alpha_{SUP} + \gamma_{fLLIVE}\alpha_{LIVE} + \gamma_{fLO}\alpha_o\right)_{SLS}}$$

α_{DEAD}, α_{SUP}, α_{LIVE} α_o are the fractions of load effect due to dead load, superimposed load, live load and other loads, to the load effect due to the total load, respectively.

NOTE: For plate and box girder construction, serviceability limit state checks will not be required when 1/ ψ_R <1.30, i.e. the shear lag factor ψ is greater than 0.77 when determined in accordance with **8.2**.

9.2.3.2 Effects to be considered.

In line 3, delete '(d)' *and substitute* '(e)'.

In item (b), delete 'and distortional'.

9.2.1.3 *Effects that may be neglected.* The effects of shear lag and restraint of torsional and distortional warping may be neglected for the ultimate limit state.

Items (c) and (d) of **9.2.1.2** may be neglected at the ultimate limit state provided that:

(a) the section is compact throughout the span being considered in accordance with the provisions of **9.3.7**; and
(b) the member is not prone to lateral instability; this may be deemed to be satisfied when the slenderness parameter, λ_{LT} is less than $45\sqrt{355/\sigma_y}$ (see **9.7**).

9.2.2. *Fatigue.* The fatigue endurance should be in accordance with the recommendations of Part 10.

9.2.3 *Serviceability limit state*

9.2.3.1 *General.* The serviceability limit state provisions should additionally be met where called for by the following.

(a) When the maximum longitudinal stress is more than 1.67 times the average longitudinal stress in the particular flange segment between adjacent webs, or in the overhang from an outside web as appropriate. For this purpose, a 'flange segment' is the whole flange width between adjacent webs, or the whole overhang, whether or not stiffened by longitudinal stiffeners.

NOTE. For plate girder construction this criterion may be deemed to be satisfied if the shear lag effective breadth ratio ψ, determined in accordance with 8.2, is greater than 0.6.

(b) When redistribution of stresses from the tension flange is made in accordance with **9.5.5**.
(c) When stiffened flanges are subjected to bending by local loads (see **9.10.3.3**).
(d) When required in **9.9.8** for unsymmetric beams.

9.2.3.2 *Effects to be considered.* Stresses at the serviceability limit state should be obtained for the relevant combinations given in **9.2.1.2** (a) to (d) together with:

(a) the effects of shear lag;
(b) the effects of restraint of torsional and distortional warping in a girder.

9.2.4 *Composite beams.* In the design of composite beams, the concrete, reinforcement and shear connectors should satisfy the limit state requirements of Parts 4 and 5.

9.3 Shape limitations

9.3.1 General. Figure 1 sets out the geometric notation used throughout this section.

In applying **9.3.2**, **9.3.4** and **9.3.6** to sections that are not fully stressed by the applied loading, the nominal yield stress values σ_{ys} or σ_y may be taken as the lesser of: the nominal yield stress of the material or 1.5 times the maximum stress at the appropriate section calculated for the factored loading at the ultimate limit state condition,

where

σ_{ys} relates to the stiffener
σ_y relates to the flange, the web or the circular hollow section, as appropriate.

9.3.2 *Flanges*

9.3.2.1 *Outstands in compression.* Unless a free edge of a plate or other outstand is stiffened, the ratio b_{fo}/t_{fo} should not exceed $12\sqrt{355/\sigma_y}$ or 16 whichever is the lesser.

When the edge of the outstand is stiffened the ratio b_{fo}/t_{fo} should not exceed $14\sqrt{355/\sigma_y}$

where

b_{fo} is the width of the outstand measured from the edge to the nearest line of rivets or bolts connecting it to the supporting part of the member, or to the toe of a root fillet of a rolled section, or, in the case of a welded construction, to the surface of the supporting part of the member, or, in the case of composite construction, to the outer line of shear connectors
t_{fo} is the mean thickness of the outstand
σ_y is as defined in **9.3.1**.

9.3.2.2 *Outstands in tension.* The ratio b_{fo}/t_{fo} should not exceed 16, where b_{fo} and t_{fo} are as defined in **9.3.2.1**.

NOTE. Where a flange consists of several flange plates built-up and connected to each other only by welds at their edges, an outer flange plate should not be thicker than an inner plate and the above provision should be satisfied for all the flange plates. For the flange plate connected to the web, b_{fo} should be taken as given above, but for all the other flange plates b_{fo} should be taken as half the width between the welds connecting it to the adjacent inner plate.

9.3.3 *Openings*

9.3.3.1 *General.* Any openings in webs or compression flanges should be framed and the stiffened section designed for local load effects, including secondary bending. Alternatively, openings in webs may be unstiffened provided that they meet the provisions of **9.3.3.2**.

All corners should be rounded with a radius of at least one-quarter of the least dimension of the hole.

9.3.3.2 *Unstiffened openings in webs.* Openings in a web may be unstiffened provided that:

(a) the overall greatest internal dimension does not exceed one-tenth the depth of the web, nor for longitudinally stiffened webs, one-third the depth of the panel containing the opening;
(b) the longitudinal distance between boundaries of adjacent openings is at least three times the maximum internal dimension;
(c) not more than one opening is provided at any cross section

Cut outs provided for transverse stiffeners should either have at least one-third of their perimeters welded to the stiffeners, or the stiffeners should be cleated to the web with at least two bolts or rivets per side of the connection or by full perimeter welding of the cleat.

9.3.1 General

Delete the existing clause and substitute the following:-

Figures 1 and 1A sets out the geometric notation used throughout this section.

Where the proportions of flanges, stiffeners or hollow sections comply with the requirements of **9.3.2.1**, **9.3.4** and **9.3.6**, taking σ_{ys} or σ_y as the yield stress of the material as defined in **6.2**, the strength of the sections shall be determined as specified in the appropriate clauses of this standard relating to assessment of strength, using this value of σ_{ys} or σ_y where:

σ_{ys} relates to the stiffener

σ_y relates to the flange, the web or the circular hollow sections, as appropriate.

Where the proportions do not thus comply, a lower value of σ_{ys} or σ_y shall be determined such that compliance with **9.3.2.1**, **9.3.4** or **9.3.6** as appropriate is achieved, and this lower value of σ_{ys} or σ_y shall be used in all subsequent assessments of strength.

9.3.2.1 Outstands in compression.

On the third and fourth lines, delete 'or 16 whichever is the lesser'.

Add at end:

See 9.3.1 for assessment of non-complying outstands in compression.

9.3.2.2 Outstands in tension.

Not applicable to assessment.

9.3.3.1 General

Add at end:

Openings rounded with a radius of less than ¼ of the least dimension of the hole, or any openings not complying with any of the requirements of **9.3.3.2**, shall be inspected individually for evidence of cracking. They shall be assessed for the effects on fatigue life and brittle fracture propensity of stress concentrations. Detailed local analyses, e.g. finite element analysis, shall be carried out where appropriate.

9.3.4 *Stiffeners to webs and compression flanges*

9.3.4.1 *Open stiffeners to webs and compression flanges*

9.3.4.1.1 *General.* Open stiffeners should either be in accordance with 9.3.4.1.2, 9.3.4.1.3, 9.3.4.1.4 or 9.3.4.1.5, as appropriate, or be checked in accordance with appendix C.

9.3.4.1.2 *Flat stiffeners.* The stiffener proportions should be such that $\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed:

(a) 10: or
(b) a higher value obtained from figure 2, when

$\frac{b}{t}\sqrt{\frac{\sigma_y}{355}}$ is less than 31

where, as shown in figure 1,

h_s, t_s are the depth and thickness, respectively, of the stiffener
b is the spacing of stiffeners or distance between the stiffener and the beam flange/web boundary, as appropriate

NOTE. In the case of non-uniform spacing the average value on the two sides may be taken.

t is the plate thickness t_f or t_w
σ_{ys} and σ_y are as defined in 9.3.1.

9.3.4.1.3 *Bulb flat stiffeners.* Bulb flat stiffeners should comply with the requirements of Part 5 of BS 4848 and should be proportioned such that:

(a) either $\frac{\ell_s}{h_s}$ does not exceed $3\sqrt{\frac{355}{\sigma_{ys}}}$

(b) or $\frac{(b + k_s h_s)}{t}\sqrt{\frac{\sigma_y}{355}}$ does not exceed 30

where

ℓ_s is the span of the stiffener between supporting members
h_s is the overall depth of the bulb flat, as shown in figure 1
k_s = 0.4 for grade 50 steel in accordance with BS 4360, or
= 0.15 for grade 43 steel in accordance with BS 4360
b and t are as defined in 9.3.4.1.2
σ_{ys} and σ_y are as defined in 9.3.1.

9.3.4.1.4 *Angle stiffeners.* Angle stiffeners should comply with the requirements of Part 4 of BS 4848 and should be proportioned such that:

(a) b_s does not exceed h_s;

(b) $\frac{b_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 11;

(c) $\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed either:

(1) 7; or
(2) a higher value obtained from figure 3 when

$\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}}$ is less than 50

NOTE. When $\frac{b}{t}\sqrt{\frac{\sigma_y}{355}}$ is less than or equal to 30, there is no limitation on $\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$.

where

h_s, b_s and t_s are the depth, width and thickness of the angle, respectively, as shown in figure 1
b and t are as defined in 9.3.4.1.2
ℓ_s is as defined in 9.3.4.1.3
σ_{ys} and σ_y are as defined in 9.3.1.

9.3.4.1.5 *Tee stiffeners.* Tee stiffeners should be proportioned such that:

(a) $\frac{b_{so}}{t_{so}}\sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 10;

(b) $\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys} + \sigma_a}{355}}$ does not exceed 41;

(c) $\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 7 or a higher value obtained from:

(1) either figure 4(a) when $\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}}$ is less than 25;

(2) or figure 4(b) when both $\frac{b}{t}\sqrt{\frac{\sigma_{ys}}{355}}$ is less than 32, and $\frac{d_s}{b_s}$ does not exceed 4

where

b_s and t_s are as shown in figure 1
b and t are as defined in 9.3.4.1.2
ℓ_s is as defined in 9.3.4.1.3
t_{so} is the average thickness of the flange outstand width b_{so}
d_s is the effective stiffener depth, measured from the underside of the flange plate to the midplane of the equivalent uniform thickness of the flange of the tee (see figure 1)
σ_a is the longitudinal stress (positive when compressive) for the ultimate limit state at the centroid of the effective section of the stiffener, as given in 9.10.2.2 for flanges and 9.11.5.1 for webs, but may be taken conservatively as σ_{ys}
σ_{ys} and σ_y are as defined in 9.3.1.

9.3.4.2 *Closed stiffeners to webs and compression flanges.* Closed stiffeners should be proportioned such that:

(a) $\frac{d_{s1}}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$ does not exceed 29;

(b) $\frac{d_{s2}}{t_s}\sqrt{\frac{\sigma_{ys} + \sigma_a}{355}}$ does not exceed 41.

where

σ_a is as defined in 9.3.4.1.5
d_{s1}, d_{s2} are the widths of the walls of the stiffener as shown in figure 1
t_s is the thickness of the stiffener
σ_{ys} is as defined in 9.3.1.

9.3.4.1.1. General.

Add at end:

See 9.3.1 and Appendix S of the accompanying Advice Note for the assessment of non-complying stiffener configurations. Other shapes of stiffeners other than those specified shall be assessed on the basis of the nearest standard shape.

9.3.4.1.3 Bulb flat stiffeners

Add at end:

For assessment, the requirement in this subclause may be omitted.

9.3.4.1.4 Angle stiffeners

Add at end:

For assessment, the requirement in this subclause may be omitted.

9.3.4.1.5 Tee stiffeners

In item (c) (2), delete $\sqrt{\sigma_{ys} \ / \ 355}$ and substitute $\sqrt{\sigma_{y} \ / \ 355}$

9.3.4.2 Closed stiffeners to webs and compression flanges.

Add at end:

See 9.3.1 and Appendix S of the accompanying Advice Note BA 56 for the assessment of non-complying stiffener configurations. Shapes of stiffeners other than those specified shall be assessed on the basis of the nearest standard shape.

Add new Clause 9.3.4.3:

9.3.4.3 Combinations of closed and open stiffener

A stiffener fabricated from a combination of closed and open sections shall be proportioned such that individual components meet the requirements of **9.3.4.1** or **9.3.4.2** as appropriate.

When an element is not connected directly to the parent plate, no benefit shall be assumed from the restraining effect of the parent plate when using Appendix S or any other method.

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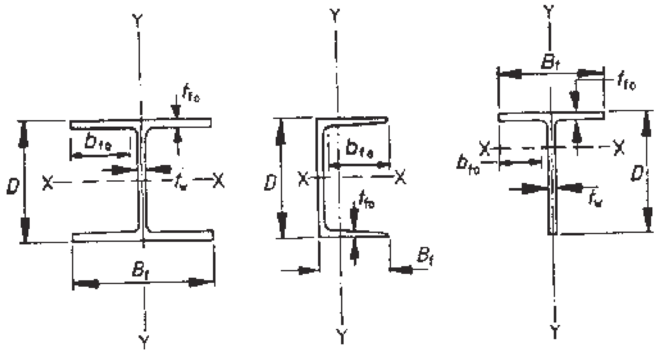
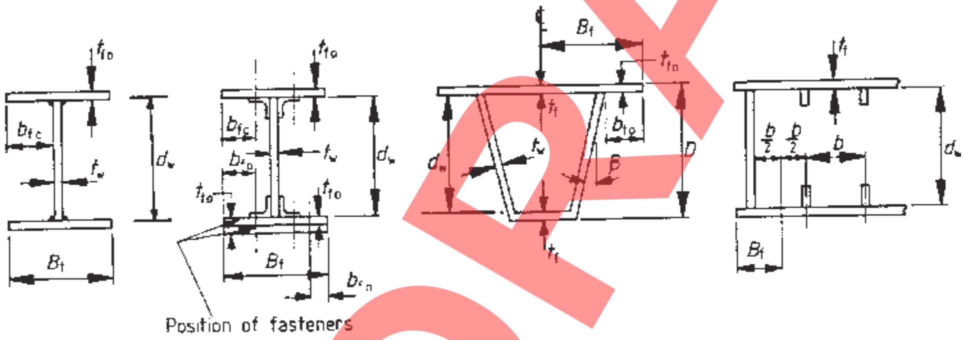
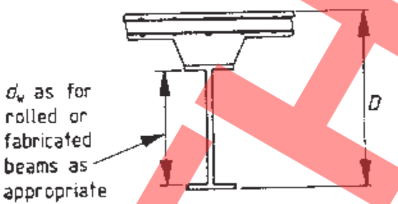


NOTE. For tapered flanges, t_{fo} should be taken as the mean thickness of the flange, as defined in BS 4.

(a) Rolled beam sections, $d_w = D$

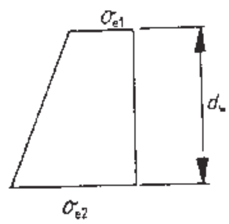


(b) Fabricated beam sections, d_w is as shown



NOTE. D is shown for concrete in compression. For concrete in tension D is taken to level of tension reinforcement.

(c) Composite beam section



(d) Stress distribution in web

Figure 1. Geometric notation for beams

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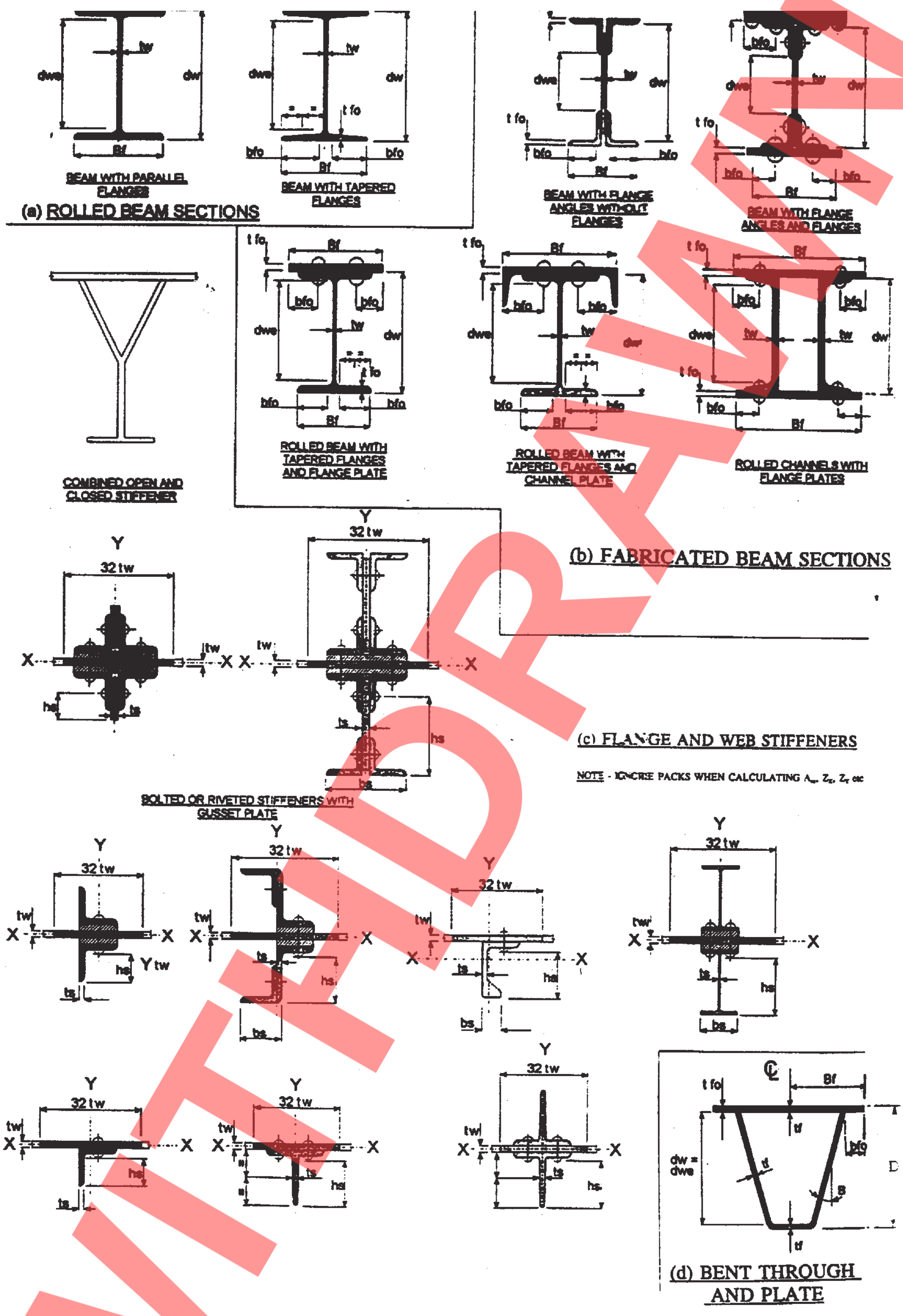
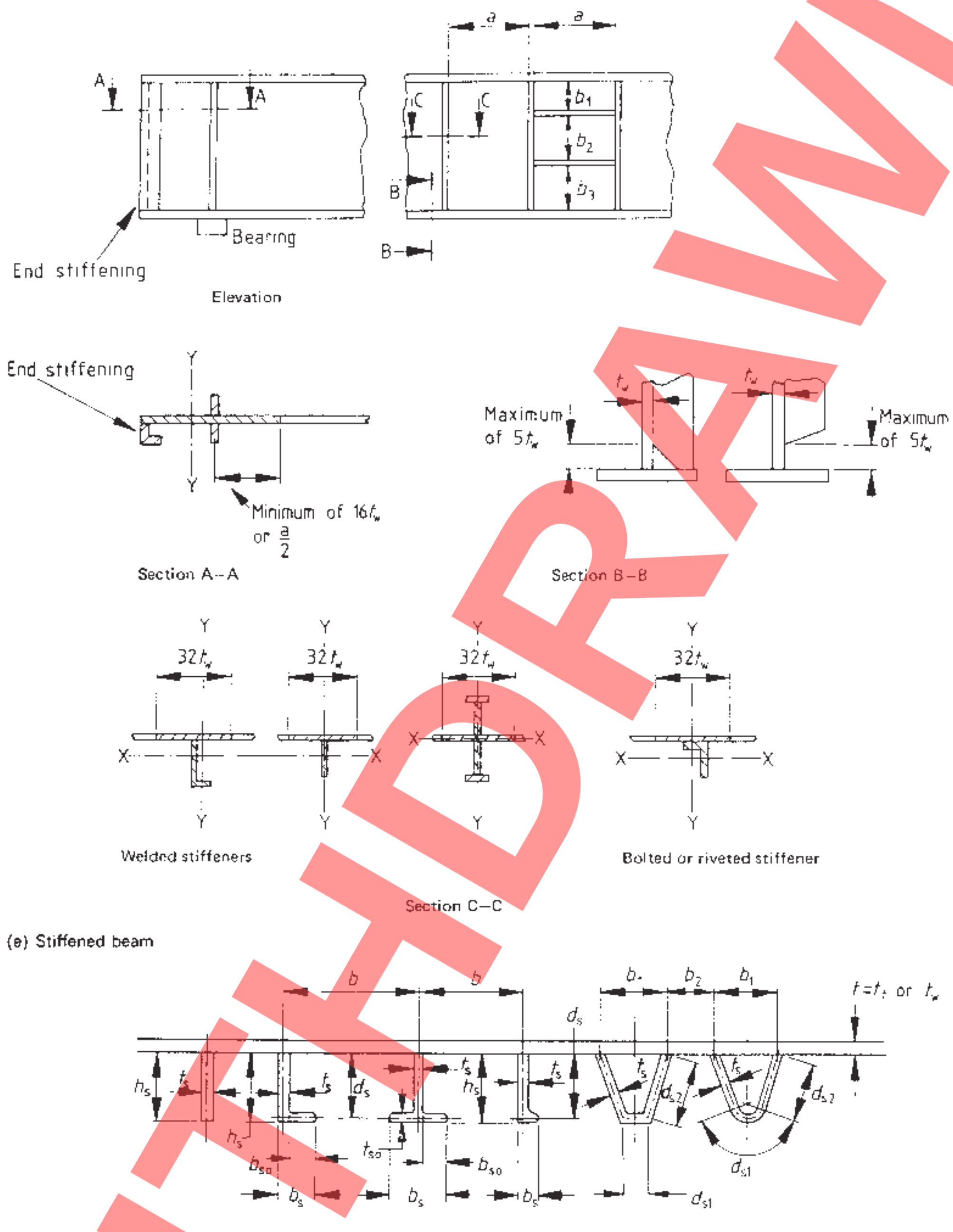


Figure 1A Geometric notation for sections that may be encountered in assessment



NOTE. d_{s1}, d_{s2} measured on centreline.

(f) Flange and web stiffeners

Figure 1 *(concluded)*

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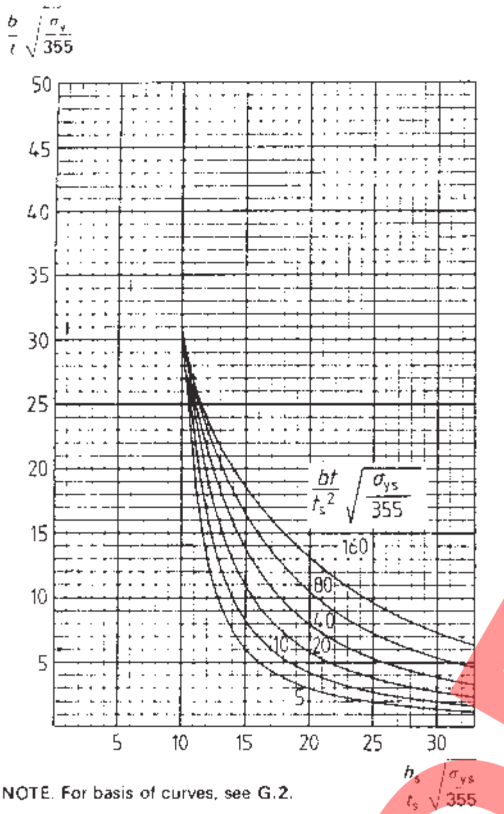
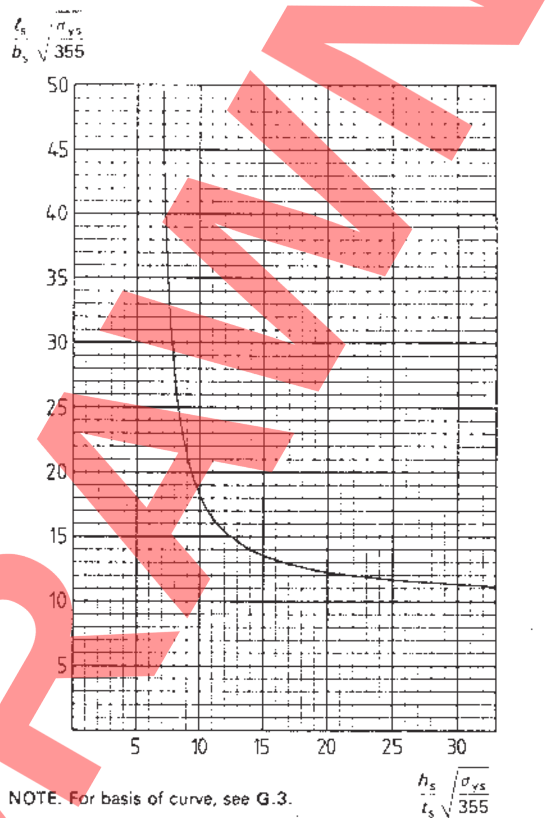


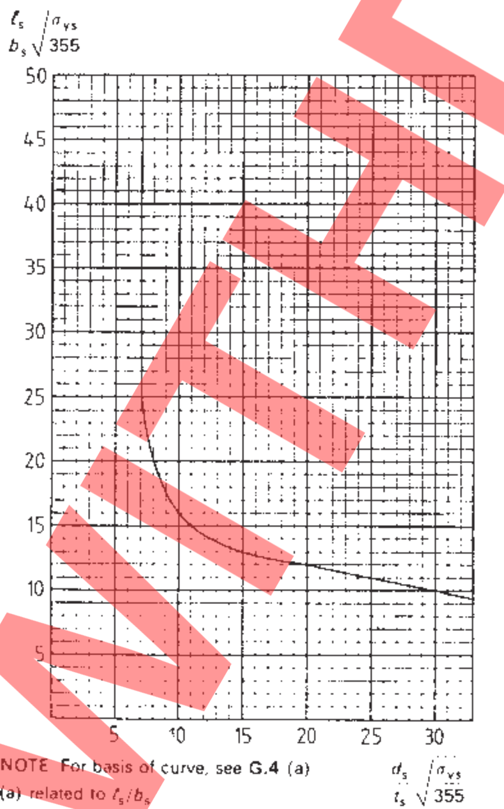
NOTE. For basis of curves, see G.2.

Figure 2. Limiting slenderness for flat stiffeners



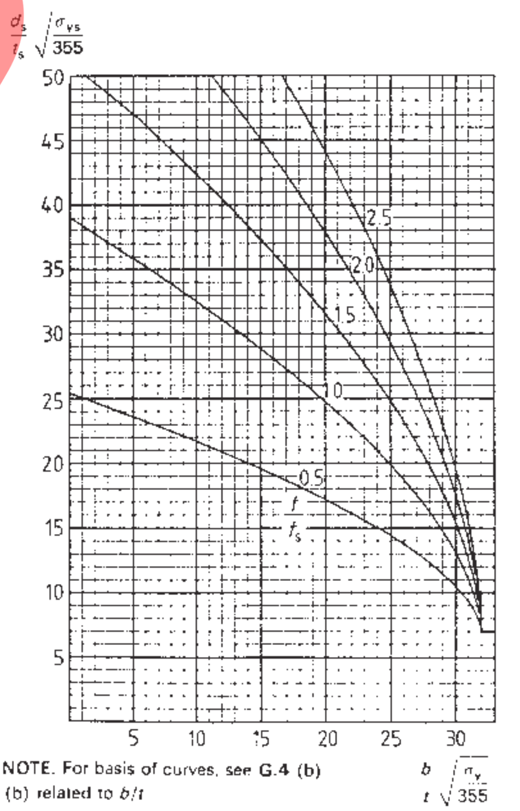
NOTE. For basis of curve, see G.3.

Figure 3. Limiting slenderness for angle stiffeners



NOTE. For basis of curve, see G.4 (a)

(a) related to ℓ_s/b_s



NOTE. For basis of curves, see G.4 (b)

(b) related to b/t

Figure 4. Limiting slenderness for tee stiffeners

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9.3.5 *Flanges curved in elevation.* Flanges curved in elevation should be such that:

(a) $\frac{b_{fo}}{t_{fo}} \leqslant \frac{R_f}{6b_{fo}}$

(b) $\frac{b}{t_f} \leqslant \frac{R_f}{2b}$

where

R_f is the radius of curvature of the flange
b is the distance between adjacent longitudinal stiffeners and/or webs
t_f is the thickness of the flange in the panel between such longitudinal stiffeners and/or webs
b_{fo} and t_{fo} are as defined in **9.3.2.**

9.3.6 *Circular hollow sections.* The ratio of outside diameter to wall thickness of a circular hollow section should not exceed:

$$100\sqrt{\frac{355}{\sigma_y}}$$

where

σ_y is as defined in **9.3.1.**

9.3.7 *Compact sections*

9.3.7.1 *General.* Compact sections are those in which the full plastic moment can be developed before, and maintained after, the onset of local buckling. Rolled or fabricated I-beams, channels or hollow sections are defined as compact provided that they meet the provisions of **9.3.7.2** and **9.3.7.3**, or **9.3.7.4**, as appropriate.

Longitudinal stiffeners, if any, should be ignored in calculating the section properties and in deriving the strength of a compact beam.

NOTE. For beams built in stages, the requirements should be applied to the cross section of the beam appropriate to the stage considered (see **9.9.5**).

9.3.7.2 *Webs*

9.3.7.2.1 *Webs partly in compression and partly in tension.* The depth between the elastic neutral axis of the beam and the compressive edge of the web should not exceed:

$$28t_w\sqrt{\frac{355}{\sigma_{yw}}}$$

where

t_w is the thickness of the web plate
σ_{yw} is the nominal yield stress of the web material.

9.3.7.2.2 *Webs wholly in compression.* The depth of the web should not exceed:

$$\left\{24+4\left[1-\left(\frac{\sigma_{e1}}{\sigma_{e2}}\right)^3\right]\right\}t_w\sqrt{\frac{355}{\sigma_{yw}}}$$

where

t_w and σ_{yw} are as defined in **9.3.7.2.1**
σ_{e1}, σ_{e2} are the longitudinal stresses at the two edges of the web, as defined in figure 1, such that
$\sigma_{e1} < \sigma_{e2}$

NOTE. The depth of the web referred to in **9.3.7.2.1** and **9.3.7.2.2** should be measured in its plane and taken clear of root fillets for rolled sections and welds or flange angles for fabricated sections.

9.3.7.3 *Compression flanges*

9.3.7.3.1 *Compression flange outstands.* The projection of a compression flange outstand, b_{fo}, should not exceed:

$$7t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$$

where

σ_{yf} is the nominal yield stress of the flange material
t_{fo} and b_{fo} are as defined in **9.3.2.1.**

9.3.7.3.2 *Compression flanges in box sections.* The clear width of the compression flange should not exceed:

$$24t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

where

t_f is the thickness of the compression flange plate
σ_{yf} is the nominal yield of the flange material.

The clear width is taken as between:

(a) root fillets for rolled sections;
(b) webs for welded construction;
(c) lines of rivets or bolts connecting the flange to the web for other fabricated sections.

9.3.7.3.3 *Composite compression flanges.* In composite compression flanges, the spacing of shear connectors perpendicular to the direction of compression should not exceed:

$$24t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

and the spacing in the direction of compression should not exceed:

$$12t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

but may be increased to:

$$18t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

in any line of shear connectors where adjacent lines are staggered.

where

t_f and σ_{yf} are as defined in **9.3.7.3.2.**

9.3.7.4 *Circular hollow sections.* The ratio of the outside diameter to the wall thickness of a circular hollow section should not exceed:

$$50\sqrt{\frac{355}{\sigma_y}}$$

where

σ_y is the nominal yield stress of the material of the circular hollow section.

9.4 Effective section

9.4.1 *Effective section for global analysis.* Gross section properties may be used for global analysis except for transverse members for which reference should be made to **9.15.2.1.** For composite construction the procedure set out in 5.1 of Part 5 of BS 5400:1979 should be adopted for all limit states.

9.3.5 Flanges curved in elevation

Add at end:

Assessment of flanges curved in elevation but not complying with the above shall be analysed in detail allowing for the effects of curvature on the stability of the elements.

9.3.6 Circular hollow sections

Delete the expression and substitute $60(355/\sigma_y)$

Add at end:

See **9.3.1** and Appendix S for the assessment of non-complying sections.

9.3.7.1 General

Add at end:

Where any part of a cross section fails to comply with the appropriate requirements the complete section shall be assessed as non-compact.

Add new clause 9.3.7.2.3

9.3.7.2.3 Alternative method

As an alternative to **9.3.7.2.1** & **9.3.7.2.2** the depth of the web shall not exceed:

$$\frac{34t_w}{m}\sqrt{\frac{355}{\sigma_{yw}}}$$

when m does not exceed 0.5

or

$$\frac{374t_w}{(13m-1)}\sqrt{\frac{355}{\sigma_{yw}}}$$

when m exceeds 0.5

where

- m is the ratio of the depth of the web plate which is on the compressive side of the plastic neutral axis of the beam to the depth of the web plate.
- t_w is the thickness of the web plate
- σ_{yw} is the nominal yield stress of the web material or any other lower stress assumed in assessment.

NOTE 1. The depth of the web referred to in this clause shall be measured in its plane and taken clear of root fillets for rolled sections and welds or flange angles for fabricated sections.

NOTE 2. In calculating the plastic neutral axis of the beam, only the longitudinal stress due to bending moment and/or axial force need to be considered.

9.3.7.3.3 Composite compression flanges.

Delete the existing expressions and substitute the following expressions respectively:

$$30\,t_f\sqrt{355/\sigma_{yf}}$$

$$15\,t_f\sqrt{355/\sigma_{yf}}$$

$$22\,t_f\sqrt{355/\sigma_{yf}}$$

9.3.7.4 Circular hollow sections.

Delete the existing expressions and substitute $46(355/\sigma_y)$.

Add new Clause 9.3.8:

9.3.8 Plastic sections

9.3.8.1 General

The use of plastic sections and analysis shall be in accordance with **7.4**. Plastic sections are those which possess adequate ductility to enable them to carry the full plastic moment whilst allowing rotation at a plastic hinge to occur. Rolled or fabricated I-beams, channels and hollow sections can be taken to have plastic sections provided that:

(a) They meet the limitations of shape defined in **9.3.8.2** to **9.3.8.4**.

(b) The steel materials satisfy the requirements of **6.3** and **6.4**.

Longitudinal stiffeners, if any, shall be ignored in calculating the section properties and in deriving the strength of a beam.

All parts of the cross section including stiffeners shall comply with the appropriate requirements.

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9.3.5 *Flanges curved in elevation.* Flanges curved in elevation should be such that:

(a) $\frac{b_{fo}}{t_{fo}} \leqslant \frac{R_f}{6 b_{fo}}$

(b) $\frac{b}{t_f} \leqslant \frac{R_f}{2b}$

where

R_f is the radius of curvature of the flange
b is the distance between adjacent longitudinal stiffeners and/or webs
t_f is the thickness of the flange in the panel between such longitudinal stiffeners and/or webs
b_{fo} and t_{fo} are as defined in **9.3.2**.

9.3.6 *Circular hollow sections.* The ratio of outside diameter to wall thickness of a circular hollow section should not exceed:

$$100\sqrt{\frac{355}{\sigma_y}}$$

where

σ_y is as defined in **9.3.1**.

9.3.7 *Compact sections*

9.3.7.1 *General.* Compact sections are those in which the full plastic moment can be developed before, and maintained after, the onset of local buckling. Rolled or fabricated I-beams, channels or hollow sections are defined as compact provided that they meet the provisions of **9.3.7.2** and **9.3.7.3**, or **9.3.7.4**, as appropriate.

Longitudinal stiffeners, if any, should be ignored in calculating the section properties and in deriving the strength of a compact beam.

NOTE. For beams built in stages, the requirements should be applied to the cross section of the beam appropriate to the stage considered (see **9.9.5**).

9.3.7.2 *Webs*

9.3.7.2.1 *Webs partly in compression and partly in tension.* The depth between the elastic neutral axis of the beam and the compressive edge of the web should not exceed:

$$28 t_w \sqrt{\frac{355}{\sigma_{yw}}}$$

where

t_w is the thickness of the web plate
σ_{yw} is the nominal yield stress of the web material.

9.3.7.2.2 *Webs wholly in compression.* The depth of the web should not exceed:

$$\left\{24 + 4\left[1 - \left(\frac{\sigma_{e1}}{\sigma_{e2}}\right)^3\right]\right\} t_w \sqrt{\frac{355}{\sigma_{yw}}}$$

where

t_w and σ_{yw} are as defined in **9.3.7.2.1**
σ_{e1}, σ_{e2} are the longitudinal stresses at the two edges of the web, as defined in figure 1, such that
$\sigma_{e1} < \sigma_{e2}$

NOTE. The depth of the web referred to in 9.3.7.2.1 and **9.3.7.2.2** should be measured in its plane and taken clear of root fillets for rolled sections and welds or flange angles for fabricated sections.

9.3.7.3 *Compression flanges*

9.3.7.3.1 *Compression flange outstands.* The projection of a compression flange outstand, b_{fo} should not exceed:

$$7 t_{fo}\sqrt{\frac{355}{\sigma_{yf}}}$$

where

σ_{yf} is the nominal yield stress of the flange material
t_{fo} and b_{fo} are as defined in **9.3.2.1**.

9.3.7.3.2 *Compression flanges in box sections.* The clear width of the compression flange should not exceed:

$$24 t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

where

t_f is the thickness of the compression flange plate
σ_{yf} is the nominal yield of the flange material.

The clear width is taken as between:

(a) root fillets for rolled sections;
(b) webs for welded construction;
(c) lines of rivets or bolts connecting the flange to the web for other fabricated sections.

9.3.7.3.3 *Composite compression flanges.* In composite compression flanges, the spacing of shear connectors perpendicular to the direction of compression should not exceed:

$$24 t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

and the spacing in the direction of compression should not exceed:

$$12 t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

but may be increased to:

$$18 t_f\sqrt{\frac{355}{\sigma_{yf}}}$$

in any line of shear connectors where adjacent lines are staggered.

where

t_f and σ_{yf} are as defined in **9.3.7.3.2**.

9.3.7.4 *Circular hollow sections.* The ratio of the outside diameter to the wall thickness of a circular hollow section should not exceed:

$$50\sqrt{\frac{355}{\sigma_y}}$$

where

σ_y is the nominal yield stress of the material of the circular hollow section.

9.4 Effective section

9.4.1 *Effective section for global analysis.* Gross section properties may be used for global analysis except for transverse members for which reference should be made to **9.15.2.1**. For composite construction the procedure set out in **5.1** of Part 5 of BS 5400:1979 should be adopted for all limit states.

9.3.8.2 Webs

The depth d_1 between the plastic neutral axis of the beam and the compressive edge of the web shall not exceed:

(a) $28t_w\sqrt{\frac{355}{\sigma_{yw}}}$, if $d_1 \leq 0.5\ d_w$

(b) $\left(32 - \frac{8d_1}{d_w}\right)t_w\sqrt{\frac{355}{\sigma_{yw}}}$, if $d_1 > 0.5\ d_w$

but not less than $24t_w\sqrt{\frac{355}{\sigma_{yw}}}$

where

t_w is the thickness of the web plate

d_w is the depth of the web as defined in Figure 1

σ_{yw} is the yield stress of the web material as defined in **6.2.1**.

9.3.8.3 Compression flanges

Compression flanges shall comply with the provisions for compact flanges given in **9.3.7.3**.

9.3.8.4 Circular hollow sections

The ratio of the outside diameter to the wall thickness of a circular hollow section shall not exceed:

$$33\left(\frac{355}{\sigma_y}\right)$$

where

σ_y is the yield stress of the material of the circular hollow section as defined in **6.2.1**.

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9.4.2 *Effective section for bending stress analysis*

9.4.2.1 *General.* The elastic modulus of a section, or the plastic modulus of a section, should be determined taking account of the provisions of **9.4.2.2** to **9.4.2.7**.

NOTE. Additional or alternative provisions are given elsewhere for specific elements such as stiffeners.

9.4.2.2 *Deduction for holes.* Holes should be deducted in accordance with **11.3.3**, where the longitudinal stress is tensile, and in accordance with **10.5.2** where the longitudinal stress is compressive.

9.4.2.3 *Shear lag effects.* When the effects of shear lag are to be taken into account in accordance with **9.2.3** an effective breadth of a flange should be used, determined in accordance with **8.2**.

9.4.2.4 *Effective compression flange.* The effective area of each panel of a compression flange should be taken as equal to $K_c bt$;

where

b and t are the unsupported width and the thickness, respectively, of the flange panel (see figure 1 and note 1 below)

K_c is taken as 1.0 for all outstands which are in accordance with 9.3.2, and for all stiffeners which are in accordance with 9.3.4, respectively, or as a coefficient, to be determined from figure 5, for plates with edges supported by adjacent components as follows:

(a) when the number of open stiffeners is three or more, or the number of closed stiffeners is two or more, from the appropriate curve for restrained panels;

(b) when there is only one longitudinal stiffener, or the flange is unstiffened from the appropriate curve for unrestrained panels;

(c) when the number of open stiffeners is two, by taking the mean of the values obtained from the two curves 1 and 2, or from curves 3, as appropriate.

NOTE 1. In using figure 5 the value of the slenderness parameter, λ should be taken as:

$$\lambda = \frac{b}{t_f}\sqrt{\frac{\sigma_{yf}}{355}}$$

for panels in a stiffened compression flange, or for an unstiffened box.

In the case of closed stiffeners, when b_1 and b_2 are different, an average value may be taken provided:

$$0.67 \leqslant \frac{b_1}{b_2} \leqslant 1.5$$

where

σ_{yf} is the nominal yield stress of the flange plate

$t, t_{fo}, b_{fo}, b, b_1, b_2$ are as shown in figure 1.

NOTE 2. For rolled sections or unstiffened flanges gross properties may be used, providing the reduction due to K_c is incorporated in the calculation of the limiting compressive stress (see **9.8**).

9.4.2.5 *Effective web*

9.4.2.5.1 *Beams without longitudinal stiffeners.* An effective web thickness t_{we} should be used as follows:

(a) $t_{we} = t_w$, if $\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} \leqslant 68$

(b) $t_{we} = \left(1.425 - 0.00625\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}\right)t_w$,

if $68 < \frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} < 228$

(c) $t_{we} = 0$, if $\frac{y_c}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} \geqslant 228$

where

y_c is the depth of web measured in its plane from the elastic neutral axis of the gross section of the beam to the compressive edge of the web. For beams built and loaded in stages, y_c should be calculated for the cross section of the beam appropriate to the stage considered (see **9.9.5**)

σ_{yw} is the nominal yield stress of the web material

t_w is the thickness of the web.

9.4.2.5.2 *Beams with effective longitudinal stiffeners.* An effective web thickness t_{we} should be used as follows:

$t_{we} = t_w$ for beams with longitudinal stiffeners designed in accordance with **9.11.5**

where

t_w is as defined in **9.4.2.5.1**.

9.4.2.6 *Stiffener continuity.* The area of longitudinal stiffeners should be included for stress analysis only if they are continuous on either side of the section under consideration over a distance equal to the depth of the beam.

9.4.2.7 *Composite construction.* For composite construction the area of concrete in a tensile zone should be ignored.

9.5 Evaluation of stresses

9.5.1 *General.* Longitudinal stresses due to bending, and to axial force if any, should be calculated on the basis of an effective section in accordance with **9.4**, **10.5**, or **11.3**, as appropriate.

The shear flow in a web due to applied shear force may be taken as the average value throughout the net depth of the web, equal to $(d_w - h_h)$.

where

d_w is the full depth of a rolled section and the depth of a web plate between flanges in a fabricated section, both as shown in figure 1

h_h is the height of any hole in the section.

9.5.2 *Stresses in longitudinally stiffened webs.* The longitudinal stress in a longitudinally stiffened web should be calculated by the elastic theory without any assumption of redistribution of stresses. If the stress varies within the length a between transverse stiffeners, the value appropriate for stiffener design should be taken as that on the line of the stiffener at a distance $0.4a$ from the more severely stressed end.

9.4.2.4 Effective compression flange

In line 3, delete '$K_c bt$' *and substitute* '$K_c k_h A_c$'

Delete definitions for b and t

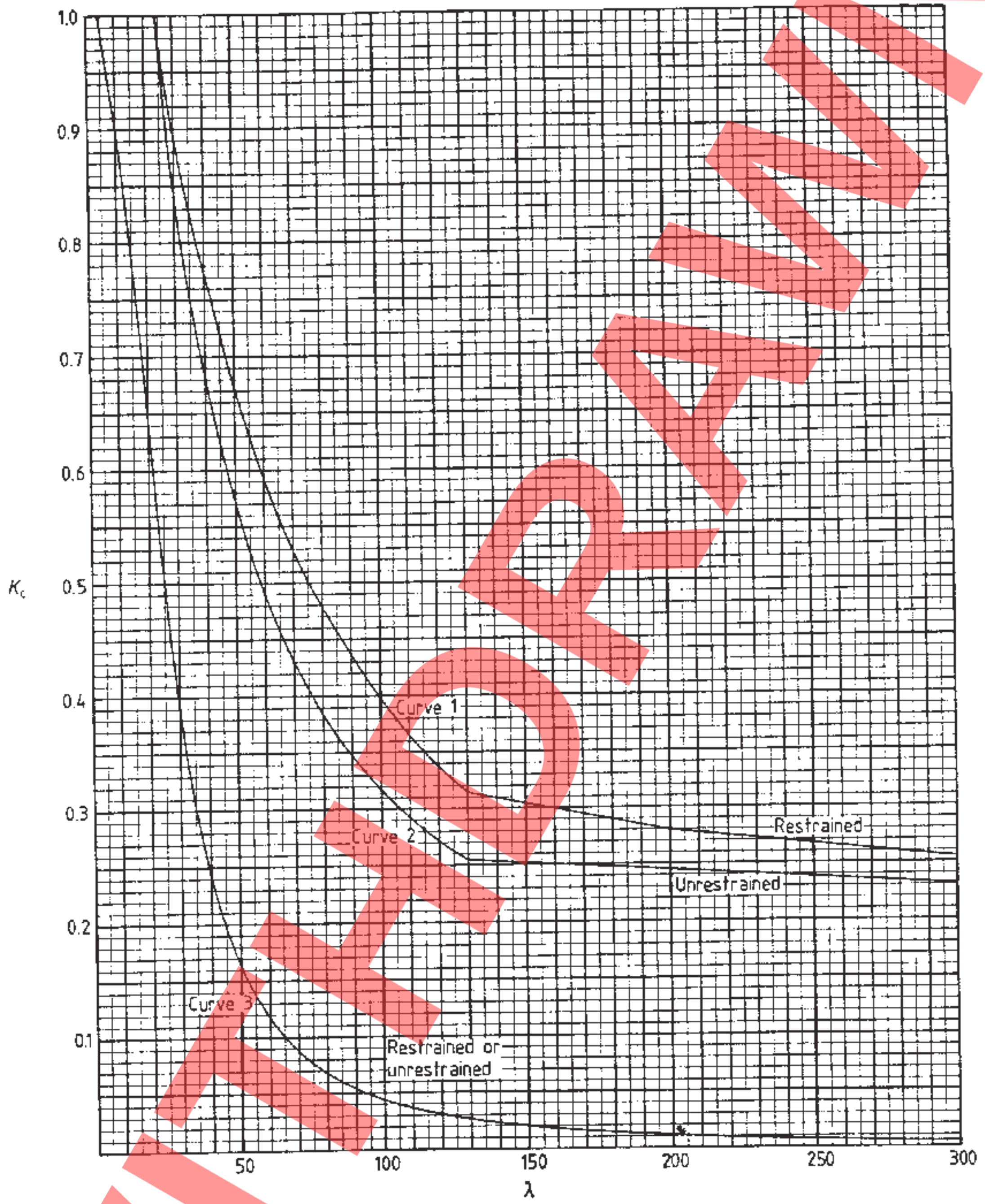
Add the following definitions:

k_h = 1.0 for a part free from holes or for a part with one or more holes greater than 40 mm in diameter, or 1.2 for a part in which holes do not exceed 40 mm in diameter, provided that $k_h A_c$ in no case exceeds the gross area of the part.

A_c is the net area of each part, derived from the gross area, less a deduction across a section perpendicular to the centre line of the part for open holes or clearance holes for pins, black bolts or countersunk bolts. Holes carrying rivets, HSFG, close tolerance or turned barrel bolts, or fully filled plug holes, need not be deducted.

Delete the entire 'NOTE 2' and substitute the following 'NOTE 2':

NOTE 2: The values of λ for plating given in **9.10.2.2** may be adopted when assessing the adequacy of longitudinal flange stiffeners complying with (a) above in accordance with **9.10.2.3** and **9.10.3.3.2**.



NOTE 1. The value of K_c to be used is the higher of the values obtained using either:

(a) curve 1 or 2 as relevant, with

$$\lambda = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} \text{ or}$$

(b) curve 3 with $\lambda = \frac{a}{t}\sqrt{\frac{\sigma_y}{355}}$

where

a is the panel dimension in the direction of stress considered
b is the panel dimension normal to the direction of stress.

Item (a) will always give the higher value for K_c when $a/b \geqslant 0.5$.

For $a/b < 0.5$, items (a) or (b) may give a higher value.

NOTE 2. For basis of curves, see G.5.

Figure 5. Coefficient K_c for plate panels under direct compression

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9.5.3 *Stresses in longitudinally stiffened compression flanges.* Longitudinal stresses in longitudinally stiffened compression flanges should be determined both at the mid-plane of the flange plate (when checking for yield) and at the centroid of the effective section of the stiffener (when checking for buckling).

If the stress varies substantially within the length a between transverse stiffeners, the stresses should be calculated at all sections (when checking for yield) and at a point $0.4a$ from the higher stressed end (when checking for buckling).

9.5.4 *Redistribution of web stresses in a longitudinally stiffened beam.* The longitudinal stresses in any web panel or panels, of a beam with longitudinal stiffeners either in the compression flange or the web or both, may be assumed to be reduced by not more than 60% by shedding any appropriate part of the moment and/or axial force to the flanges, provided that the assumed stress distribution, after such shedding, is such that the whole of the applied bending moment and axial force is transmitted and equilibrium is maintained. The percentage reduction in stress so assumed should be uniform within any one panel of web plate bounded on each longitudinal edge by a longitudinal stiffener, or by a flange, but may vary from panel to panel.

No shedding should be made from any panel containing a hole or opening having a diameter in any direction greater than six times the thickness of the web, or one-fifth of the smaller dimension of the panel, whichever is less; nor from any panel any part of which is within a distance from the nearest edge of such hole or opening equal to the largest diameter of the hole or opening.

9.5.5 *Redistribution of tension flange stresses in a longitudinally stiffened beam.* When elastic distribution of stresses in a cross section in a longitudinally stiffened beam for the ultimate limit state causes yielding of the tension flange but not buckling or yielding of the compression flange, redistribution of the stresses may be assumed subject to the following.

(a) Redistribution may only be made at the ultimate limit state, and the whole cross section should satisfy the serviceability limit state, without redistribution.
(b) A linearly varying pattern of longitudinal strains should be assumed over the whole cross section, such that the stresses produced are in equilibrium with the load effects at the ultimate limit state. Longitudinal stresses at any point should be taken as the lesser of:

(1) the assumed strain times the modulus of elasticity E

(2) $\sqrt{\left(\dfrac{\sigma_y}{\gamma_m \gamma_{f3}}\right)^2 - 3\tau^2}$

where

τ is the elastic shear stress at point under consideration

σ_y is the nominal yield stress of the section under consideration.

(c) The strain assumed in the compression flange should not exceed $1/E$ times the stress capacity obtained from **9.10**, and the strain assumed in the tension flange should not exceed $2\sigma_y/E$.
(d) The web stresses obtained in (b) should be in accordance with **9.11**. For the buckling check of a web plate panel, in accordance with **9.11.4**, an equivalent system of stress patterns, shown in figure 19, may be assumed such that the net axial force along, and the bending moment about, the longitudinal centreline of the panel remain unchanged.

9.5.6 *Transverse stresses in webs.* The transverse stress in the plane of a web due to load applied to a flange may be calculated on the assumption that the load is dispersed uniformly:

(a) at an angle of 60° from the line of application of the load through the thickness of any plate against which the load is bearing; and
(b) at an angle of 45° from the line of application of the load through the web plate itself (see figure 6).

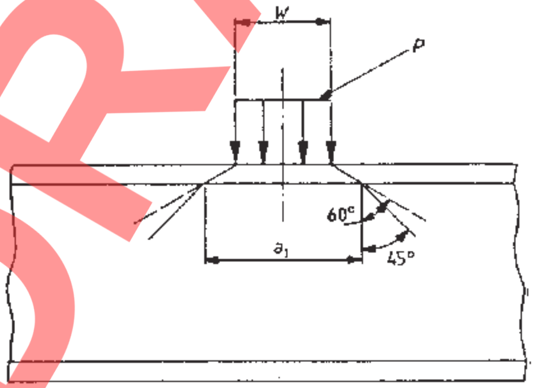

Figure 6. Dispersal of load through an unstiffened web

9.5.7 *Flanges curved in elevation*

9.5.7.1 *Stresses in flanges.* For flanges curved in elevation, a transverse bending stress due to the radial component of the longitudinal force in the flange should be taken as:

$\dfrac{3\sigma_f b_{fo}^2}{R_1 t_{fo}}$ in a flange outstand

$\dfrac{3\sigma_f b^2}{4R_1 t_f}$ in a plate panel of a flange between longitudinal stiffeners and/or webs,

where

σ_f is the longitudinal stress in the flange
b is the distance between successive longitudinal stiffeners and/or webs
t_f is the thickness of the flange in the panel being considered
b_{fo} and t_{fo} are as defined in **9.3.2** (see also figure 1)
R_1 is the radius of curvature at the flange.

9.5.4 Redistribution of web stresses in a longitudinally stiffened beam.

Add at end:

The effective longitudinal stiffness of the web panels, from which stress is assumed to be redistributed, shall be reduced by a fraction ρ_w (equivalent to using a modular ratio for the panels of $(1- \rho_w)$). The modified properties of the cross section shall be used to calculate the revised longitudinal stress distribution either from the load effects calculated by use of the gross section properties in the global analysis or by use of those calculated using the modified properties for those portions of the beam in which redistribution is assumed. Due account shall be taken of any change in bending moment due to longitudinal loads resulting from change in effective centroid position.

9.5.6 Transverse stresses in webs.

Delete the entire Clause and substitute the following:

The transverse stress in the plane of a web due to load applied to a flange shall be calculated on the assumption that the load is dispersed (as shown in figure 6) uniformly:

(a) At an angle of 60° from the line of application of the load through the combined thickness against which the load is bearing, where present, of bearing plate, flange plates, horizontal leg and root radius of flange angles or of rolled section beams.

(b) At an angle of 45° from the line of application of the load through the remainder of the web plate itself.

Transfer of load by direct bearing between flange plate and web shall not be assumed in the case of riveted construction unless reasonable evidence of direct contact is available, ie by sight of the end of the beam, for example.

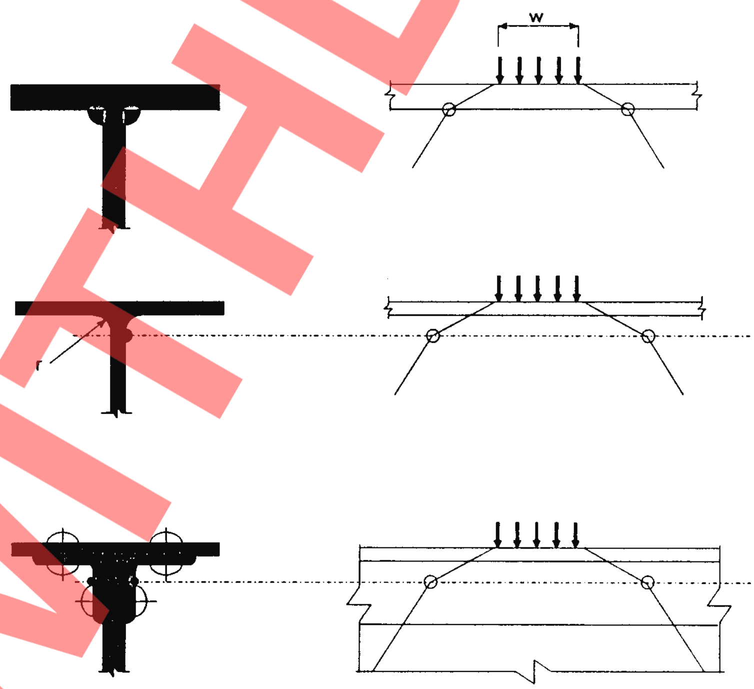

Figure 6. Dispersal of load through webs
(Load P shown shall be compressive or tensile)

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NOTE. The stresses are not applicable when the section is unsymmetrical about a vertical axis or curved in plan or any plane other than the vertical, such a section being outside the scope of this clause.

9.5.7.2 *Stresses in webs*

(a) *Shear force*. The vertical components of the forces in flanges should be taken into account in computing the shear force carried by the web.

(b) *Edge force*. The edge of a web attached to a curved portion of a flange should be considered to be subjected to a force, acting in the plane of the web, equal to:

$$\frac{\sigma_f B_f t_f}{R_f \cos \beta} \text{ per unit length,}$$

where, as shown in figure 1,

B_f is the width of an unstiffened flange, in a beam having only one web, or half the distance between successive longitudinal stiffeners or webs, together with any adjacent outstand

β is the slope of the web to the vertical

σ_f, t_f, R_f are as defined in **9.5.7.1**.

9.5.8 ***Flanges sloping in elevation.*** In computing the shear force carried by the web at any section of a beam, the vertical components of the longitudinal forces in sloping flanges should be taken into account.

9.6 Effective length for lateral torsional buckling

9.6.1 ***General.*** All beams should be restrained at their supports against rotation about their longitudinal axis in accordance with **9.12.4**.

In all cases the effective length for lateral torsional buckling ℓ_e should be determined in accordance with **9.6.2** to **9.6.6**, as appropriate. However, if the second moment of area of a cross section about the axis of bending is smaller than that about an axis perpendicular to it, the cross section as a whole is stable against lateral torsional buckling and its effective length ℓ_e may be taken as zero. Consideration should, however, be given to possible instability of constituent parts of the cross section.

9.6.2 ***Beams with intermediate lateral restraints*** When a compression flange of a beam is provided with effective discrete lateral restraints in accordance with **9.12.1**, ℓ_e should be taken as the greatest distance between such points of lateral restraint or between a restraint and a support. Where such restraint is provided by interconnecting bracing between two or more beams, consideration should be given to the possibility of lateral instability of the combined cross section.

9.6.3 ***Beams (other than cantilevers) without intermediate lateral restraints.*** When there is no intermediate lateral restraint to a compression flange, ℓ_e should be taken as:

$$\ell_e = k_1 k_2 L$$

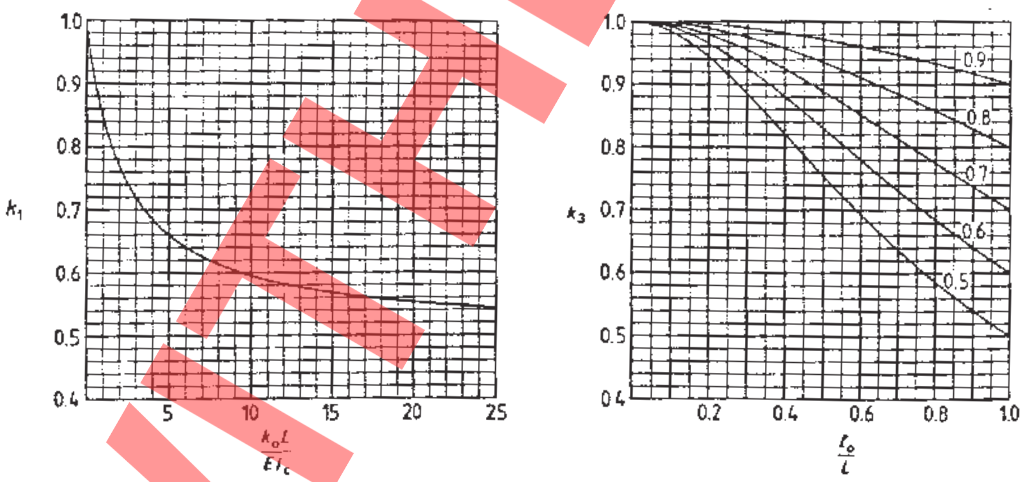
where

L is the span of the beam (i.e. between lateral restraints at supports)

k_1 may conservatively be taken as:
1.0 if the compression flange is free to rotate in plan at the points of support; or
0.85 if the compression flange is partially restrained against rotation in plan at the points of support, or where it is fully restrained against rotation in plan at one support, and free to rotate in plan at the other; or
0.7 if the compression flange is fully restrained against rotation in plan at the points of support.

NOTE. A more accurate value of k_1, allowing for the degree of restraint in plan, may be obtained from figure 7(a).

k_2 = 1.0, or
= 1.2 if the load is applied to the top flange and both the flange and the load are free to move laterally.



(a) Effect of rotational end restraint

(b) Effect of bending restraint

NOTE 1. k_o is the smaller value, at either end, of the rotational stiffness to lateral bending of the compression flange, chord or strut

I_c is as defined in 9.6.5, 10.4.1 or 12.5.1, as appropriate

L is the span of the beam or truss or length between the ends of a compression member effectively held in position

ℓ_o is the value of ℓ_e obtained from 9.6.5, 9.6.6 or 12.5.1 but calculated with k_3 = 1.0.

NOTE 2. For basis of curves, see G.6.

Figure 7. Influence on effective length of compression flange restraint

9.6.1 General

Add at end:

Where the resistance of the restraining systems is less than required to resist force F_R under **9.12.4.1** then the slenderness parameter λ_{LT} appropriate to the length l_e at the support under consideration, shall be modified as follows:

$$\lambda_{LT'} = \frac{\lambda_{LT}}{\left[\frac{1}{8}\left(\frac{5F_{RD}}{F_R} + 3\right)\right]^{1/2}}$$

where

$\lambda_{LT'}$	is a modified value of λ_{LT}
λ_{LT}	is defined in **9.7.1**
l_e	is defined in **9.6.2**
F_R	is as defined in **9.12.4.1**
F_{RD}	is the available resistance which is less than F_R excluding the effects of wind, frictional and other applied forces.

9.6.4 *Cantilevers without intermediate lateral restraints.* When a cantilever beam is not provided with lateral restraint between its support and tip, ℓ_e may be taken from table 8, in which L is the length of the cantilever.

9.6.5 *Beam restrained by U-frames.* When intermediate restraint to the compression flange is provided only by U-frames in accordance with **9.12.2**, ℓ_e should be taken as:

$$\ell_e = 2.5k_3\,(EI_c\ell_u\delta)^{0.25} \text{ but not less than } \ell_u$$

where

k_3 may be taken as 1.0, but where the compression flange is restrained against rotation in plan at the supports, a lower value of k_3 may be obtained from figure 7(b)

I_c is the second moment of area of the compression flange about its centroidal axis parallel to the web of the beam at the point of maximum bending moment

ℓ_u is the distance between U-frames

δ is the lateral deflection which would occur in the U-frame at the level of the centroid of the flange being considered, when a unit force acts laterally to the U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-frame should be taken as fixed in position at each point of intersection between the cross member and a vertical, and as free and unconnected at all other points.

In cases of symmetrical U-frames, where cross members and verticals are each of constant moment of inertia throughout their own length, it may be assumed that:

$$\delta = \frac{d_1^{\,3}}{3EI_1} + \frac{uBd_2^{\,2}}{EI_2} + fd_2^{\,2}$$

where

d_1 is the distance from the centroid of the compression flange to the nearer face of the cross member of the U-frame, as shown in figure 8

Table 8. Effective length ℓ_e for a cantilever beam without intermediate lateral restraint

Restraint conditions		Position of load	
At support	At tip	On tension flange where there is no lateral restraint to load or flange	All other positions
1. Built in	(a) Free	1.4L	0.8L
	(b) Tension flange held against displacement	1.4L	0.7L
	(c) Both flanges held against lateral displacement	0.6L	0.6L
2. Continuous, and both flanges held against lateral displacement	(a) Free	2.5L	1.0L
	(b) Tension flange held against lateral displacement	2.5L	0.9L
	(c) Both flanges held against lateral displacement	1.5L	0.8L
3. Continuous, with tension flange only held against lateral displacement	(a) Free	7.5L	3.0L
	(b) Tension flange held against lateral displacement	7.5L	2.7L
	(c) Both flanges held against lateral displacement	4.5L	2.4L

NOTE. L is the length of the cantilever.

9.6.5 Beams restrained by U-frames

Delete the equation for l_e and substitute:

$$l_e = k_3 k_5 (E I_c I_u \delta)^{0.25} \quad \text{but not less than } l_u$$

Add the definition for k_5 after the definition for k_3

k_5 = 2.5 where the beams are restrained at their supports against torsion about their longitudinal axis in accordance with **9.12.4.**

= π where the beams are unrestrained at their supports against torsion about their longitudinal axis.

NOTE: Where the restraint against torsion at supports is less than required to resist forces F_R under **9.12.4.1** or the stiffness of the bearing stiffeners is less than required to meet the criteria of **9.12.4.2** then linear interpolation shall be used to determine a value of k_5 between 2.5 and π.

Add the following paragraph at the end of the definition for f.

Values of f shall be determined experimentally or taken from test results available which shall cover the required ultimate capacity of the joint and which shall be representative of the particular type of joint.

Add at end:

Where in assessment the stiffness of the restraint against torsion at supports is less than that required under **9.12.4.2** the effective length, shall be taken as l_e^1:

$$l_e^1 = l_e \sqrt{\frac{1 + \dfrac{1}{k_6}}{1 + \dfrac{k_6}{1 + \dfrac{\delta_e}{2}\Sigma\left(\dfrac{1}{\delta}\right)}}}$$

where

l_e is as defined above
$\Sigma(1/\delta)$ is the sum of the values of $1/\delta$ for each of the intermediate U-frames in the effective length
δ_e is the value of δ for the support restraint.

$$k_6 = \frac{2 l_e^3 \Sigma\left(\dfrac{1}{\delta}\right)}{\pi^4 E I_c}$$

d_2 is the distance from the centroid of the compression flange to the centroidal axis of the cross member of the U-frame, as shown in figure 8

I_1 is the second moment of area of the effective section of the vertical about its axis of bending. A width of web plate of up to 16 times the web thickness may be included on each side of the centreline of its connection when determining the effective section of the vertical

I_2 is the second moment of area of the cross member of the U-frame about an axis perpendicular to the plane of the U-frame. A width of deck on either side of the U-frame, equal to $B/8$ or $\ell_u/2$, whichever is less, may be taken as the effective cross member when no other discrete member is present, or may be taken together with a cross member if structurally connected to it. In calculating the transformed area of a concrete deck, the gross area of concrete within this effective width may be considered

u =0.5 for an outer beam, or
=0.33 for an inner beam, if there are three or more beams interconnected by U-frames.

B is the distance between centres of consecutive beams, or the maximum distance when the beams are not exactly parallel

f is the flexibility of the joint between the cross member and the verticals of the U-frame, expressed in radians per unit moment; f may be taken as:
(a) 0.5×10^{-10} rad/N mm, when the cross member is bolted or riveted through unstiffened end-plates or cleats (see figure 42 type (a)), or
(b) 0.2×10^{-10} rad/N mm, when the cross member is bolted or riveted through stiffened end plates (see figure 42 type (b)), or
(c) 0.1×10^{-10} rad/N mm, when the cross member is welded right round its cross section or the connection is by bolting or riveting between stiffened end-plates on the cross member and a stiffened part of the vertical (see figure 42 type (c)).

9.6.6 *Beams continuously restrained by deck*

9.6.6.1 *Deck at compression flange level.* When restraint to the compression flange is provided by a deck connected to the flange over the length of the beam, in accordance with **9.12.3.1**, ℓ_e may be taken as zero.

9.6.6.2 *Deck not at compression flange level.* When restraint to the compression flange is provided by a deck connected to an unstiffened web, either directly or via the tension flange, over the length of the beam in accordance with **9.12.3.2**, the effect of this restraint should be allowed for by assuming that the deck and webs of the main beams comprise a continuous series of effective U-frames of unit length longitudinally. In this case, ℓ_e should be taken as:

$$\ell_e = 2.5 k_3 (E I_c \delta)^{0.25}$$

where

k_3 and I_c are as defined in **9.6.5**

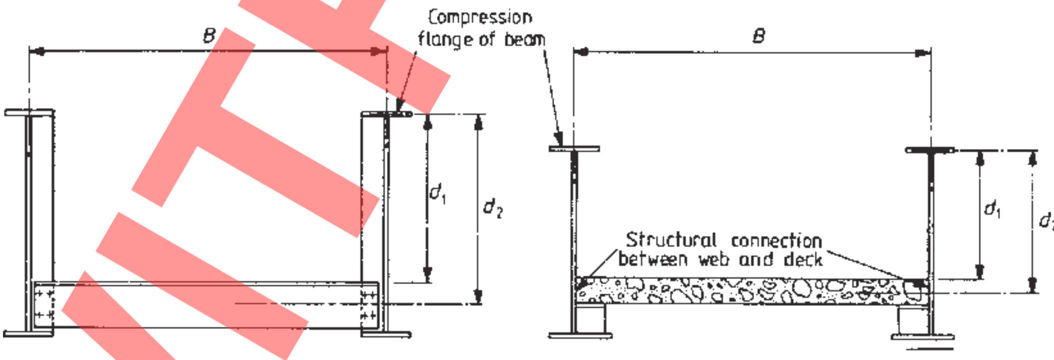
δ is the lateral deflection which would occur, in an effective U-frame, at the level of the centroid of the flange being considered, when a unit force acts laterally to the effective U-frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same effective U-frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The effective U-frame should be taken as fixed in position at each point of intersection of deck and web, and as free and unconnected at all other points.

In cases where the deck and webs of the beam are of constant thickness throughout the span, and the beam is of constant depth, it may be assumed that:

$$\delta = \frac{d_1^3}{3EI_1} + \frac{uBd_2^2}{EI_2}$$

where

d_1 is the distance from the centroid of the compression flange to the nearest surface of the structural deck (see figure 8)



(a) Main beams restrained by U-frames (see **9.6.5**)

(b) Main beams continuously restrained by deck (see **9.6.6**)

Figure 8. Restraint of compression flange by U- frames or deck

9.6.6.2 Deck not at compression flange level

Add at end:

Where in assessment the stiffness of the restraint against torsion at supports is less than that required under **9.12.4.2** the effective length shall be taken as l_e^{1} as defined in **9.6.5**.

d_2 is the distance from the centroid of the compression flange to the centroidal axis of the deck (see figure 8)

$$I_1 = \frac{t_w^3}{12}$$

t_w is the thickness of the web of the beam

I_2 is the second moment of area of the deck per unit length, about its axis of bending, with the gross concrete area being transformed in terms of steel

u and B are as defined in **9.6.5**.

9.7 Slenderness

9.7.1 ***General.*** The slenderness parameter λ_{LT} required for the calculation of the limiting compressive stress (see **9.8**) should be determined for all beams in accordance with **9.7.2** to **9.7.5** appropriate to the type of beam, using the effective length for lateral torsional buckling obtained from **9.6**.

9.7.2 ***Uniform I, channel, tee or angle sections.*** The value of λ_{LT} for a beam of I, channel, tee or angle section, uniform between points of effective lateral restraint to the compression flange, and bending about its X-X axis, as defined in figure 1, should be taken as:

$$\lambda_{LT} = \frac{\ell_e}{r_y} k_4 \eta v$$

where

ℓ_e is the effective length determined in accordance with **9.6**

r_y is the radius of gyration of the gross cross section of the beam about its Y-Y axis (see figure 1)

k_4 = 0.9 for rolled I or channel section beams in accordance with BS 4 or BS 4848 or any I section symmetrical about both axes with t_f not greater than twice the web thickness, or
= 1.0 for all other beams

η = 1.0 for beams restrained by U-frames or continuously restrained by a deck (see **9.6.5** or **9.6.6**), or may be conservatively taken as 1.0 for other beams, but where the bending moment varies substantially between points of lateral restraint, advantage may be obtained by using η, from figure 9(a), if the loading is substantially concentrated within the middle-fifth of the length between full restraints or from figure 9(b), for other loading patterns

v is dependent on the shape of the beam, and may be obtained from table 9, using the parameters:

$$\lambda_F = \frac{\ell_e}{r_y}\left(\frac{t_f}{D}\right) \text{ and } i = \frac{I_c}{I_c + I_t}$$

t_f is the mean thickness of the two flanges of an I or channel section, or the mean thickness of the table of a tee or leg of an angle section. For beams designed in accordance with **9.6.6** λ_F should be taken as zero

D is the overall depth of the cross section (see figure 1)

I_c and I_t are the second moments of area of the compression and tension flange, respectively, about their Y-Y axes, as defined in figure 1, at the section being checked. For beams with $I_c \geqslant I_t$ or with $\lambda_F \geqslant 8$, λ_{LT} may conservatively be taken as ℓ_e/r_y.

NOTE 1. Where a flange is common to two or more main beams (for example in a girder bridge with a composite deck) a safe estimate of the properties r_y, I_c or I_t may be made by assuming that each beam acts with an appropriate width of flange, taken as half the sum of the distances to the adjacent beams on each side in the case of an internal beam, or half the distance to the adjacent beam on one side, plus any outstand on the other, in the case of an external beam.

NOTE 2. In calculating t_f, I_c and I_t for composite beams, the equivalent thickness of the composite flange in compression should be based on the long term elastic modulus for concrete. Concrete in tension should be ignored and the equivalent thickness of tension reinforcement should be taken as the area of reinforcement divided by the flange width over which it is placed.

9.7.3 ***Other uniform sections***

9.7.3.1 *Uniform rectangular or trapezoidal box sections.* The value of λ_{LT} for a beam of rectangular or trapezoidal box section, uniform between points of effective lateral restraint to the compression flange, should be taken as:

$$\lambda_{LT} = 2.25\eta\xi\sqrt{\frac{SZ_{xc}\ell_e}{r_y\sqrt{AJ}}}$$

where

η and r_y are as defined in **9.7.2**

ℓ_e is as determined in accordance with **9.6**

Z_{xc} is the elastic modulus of the section with respect to the extreme compression fibre

A is the area of the gross cross section

J is the torsional constant $4A_o^2/\Sigma(B/t)$

A_o is the area enclosed by the median line of the perimeter material of the section

B and t are the width and thickness, respectively, of each wall of the section forming the closed perimeter.

NOTE 1. In the case of a wall made from material other than steel, t should be taken as the actual thickness multiplied by the ratio of the shear modulus of the material used to the shear modulus of steel. Where the shear modulus varies with the load history, the long term value should be used.

$$\xi = \left(\frac{I_x - I_y}{I_x - 0.385J}\right)^{0.25}$$

I_x and I_y are the second moments of area of the gross cross section about axis through the centroid normal to the plane of bending and in the plane of bending respectively

S = Z_{pe}/Z_{xc} for compact sections (see **9.3.7**), or
= $D/2y_t$ for non-compact sections

Z_{pe} is the plastic modulus of the section (see **9.9.1.2**)

D is the overall depth of the section

y_t is the distance from the axis of zero stress to the extreme tension fibre of the section.

NOTE 2. In composite construction with the concrete in tension, the extreme fibre is the outer surface of the tensile reinforcing steel.

NOTE 3. In stage construction y_t should be calculated from the total stresses on the section at the stage under consideration.

9.7.2 Uniform I, channel, tee or angle sections

Delete the existing definition for k_4 *and substitute the new defnitions as follows:*

$$k_4 = \left[\frac{4Z^2_{pe}\left(1 - \frac{I_y}{I_x}\right)}{A^2h^2}\right]^{1/4}$$ for flanged beams symmetrical about the minor axis

$$k_4 = \left[I_y Z^2_{pe}\frac{\left(1 - \frac{I_y}{I_x}\right)}{A^2C_w}\right]^{1/4}$$ for flanged beams symmetrical about the major axis

=1.0 for all other beams

C_w is the warping constant and can be taken equal to

$$\frac{d_f^2\, t_{ft}\, t_{fb}\, B_{ft}^3\, B_{fb}^3}{12\left(t_{ft}\, B_{ft}^3 + t_{fb}\, B_{fb}^3\right)}$$

Z_{pe} is defined in **9.9.1.2**

A, I_x, I_y are defined in **9.7.3.1**

d_f is defined in **9.9.3.1**

t_{ft} B_{ft} are the thickness and width respectively of the top flange

t_{fb} B_{fb} are the thickness and width respectively of the bottom flange

h is the distance between the centroids of the flanges.

For composite beams in which the area of longitudinal reinforcement in the slab is at least 25% of the area of the steel top flange the value of k_4 may be assumed to be:

$$\left[0.64 - \frac{0.213}{d_f^2}B_f^2\right]^{1/4}$$, but not less than 0.6

where B_f is the average width of the two flanges, the top flange width being taken as the effective width of the slab.

*Add the following **NOTE 3** at end of the Clause*

NOTE 3: Angle sections used alone as beams are strictly not covered by the above. The behaviour of angle sections is affected by the non-coincidence of the principal U-U and V-V axes.

9.7.3.1 Uniform rectangular or trapezoidal box sections

Delete the definition for 'S' and substitute the following:

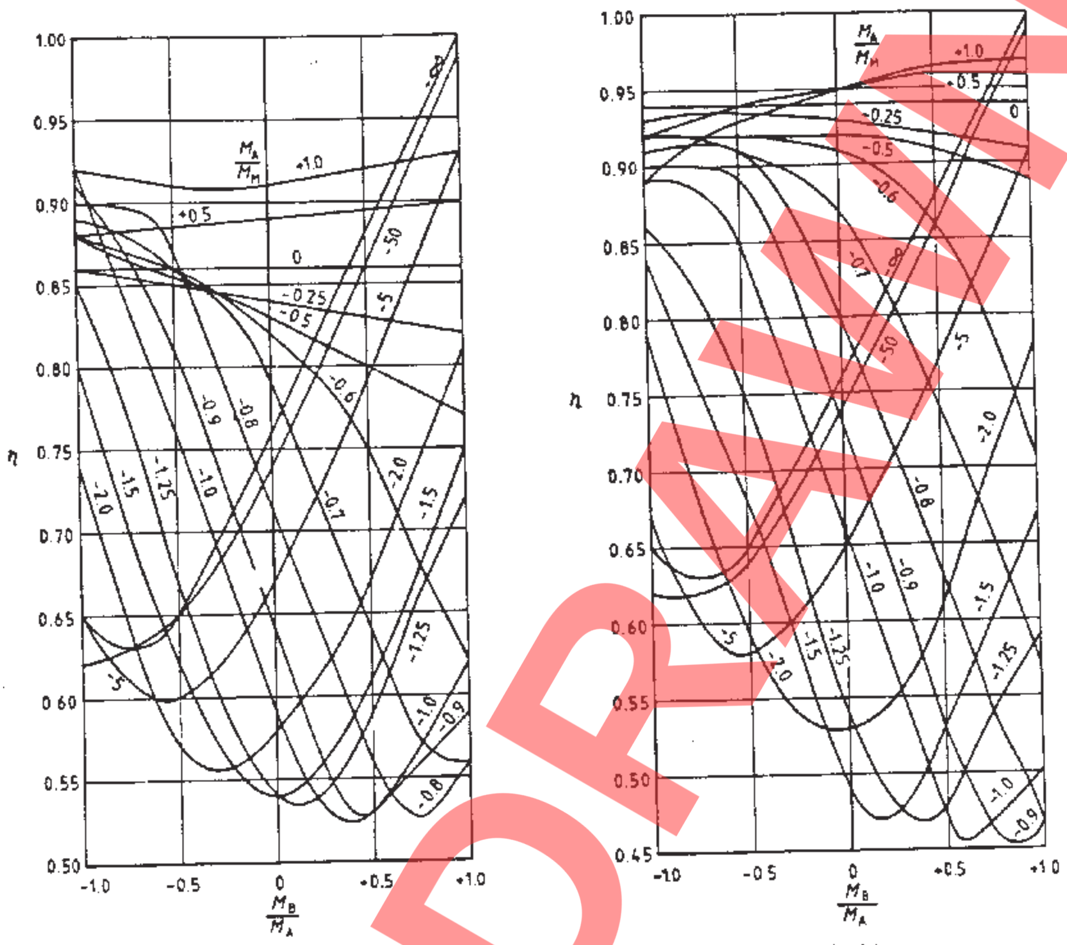
$$S = Z_{pe} / Z_{xc}$$

Delete the existing definition for "ξ" and substitute the new definition as follows:

$$\xi = \left[\frac{(I_x - I_y)(I_x - 0.385J)}{I^2_x}\right]^{0.25}$$

Delete the existing definition for 'y_t'.

Delete the entire 'NOTE 2' and 'NOTE 3'.



(a) Applied loading substantially concentrated within the middle-fifth of the length between points of full restraint

(b) Applied loading other than for (a)

NOTE 1. The procedure for using figure 9 is as follows:

(a) all hogging moments should be considered positive

(b) ends A and B should be chosen such that $M_A \geq M_B$ regardless of sign

(c) M_M is the mid-span moment on a simply supported span equal to the length between full restraints.

NOTE 2. Examples on the use of figure 9 are as follows:

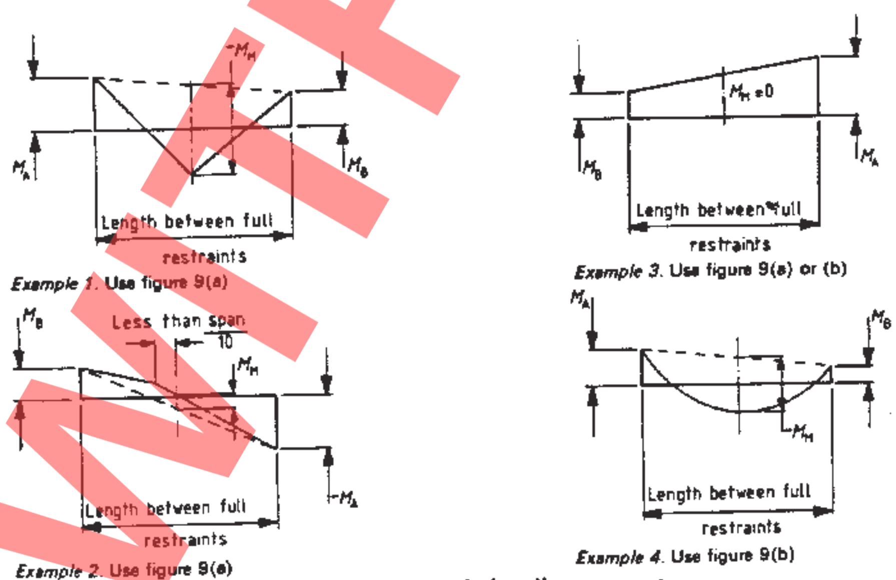


Figure 9. Slenderness factor η for variation in bending moment

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Table 9. Slenderness factor *v* for beams of uniform section

i	1.0	0.8	0.6	0.5	0.4	0.3	0.2	0.1	0
λ_F									
0.0	0.791	0.842	0.932	1.000	1.119	1.291	1.582	2.237	∞
1.0	0.784	0.834	0.922	0.988	1.102	1.266	1.535	2.110	6.364
2.0	0.764	0.813	0.895	0.956	1.057	1.200	1.421	1.840	3.237
3.0	0.737	0.784	0.859	0.912	0.998	1.116	1.287	1.573	2.214
4.0	0.708	0.752	0.818	0.864	0.936	1.031	1.162	1.359	1.711
5.0	0.679	0.719	0.778	0.817	0.878	0.954	1.055	1.196	1.415
6.0	0.651	0.688	0.740	0.774	0.824	0.887	0.966	1.071	1.219
7.0	0.626	0.660	0.705	0.734	0.777	0.829	0.892	0.973	1.080
8.0	0.602	0.633	0.674	0.699	0.736	0.779	0.831	0.895	0.977
9.0	0.581	0.609	0.645	0.668	0.699	0.736	0.780	0.832	0.896
10.0	0.562	0.587	0.620	0.639	0.667	0.699	0.736	0.779	0.831
11.0	0.544	0.567	0.597	0.614	0.639	0.666	0.698	0.735	0.778
12.0	0.528	0.549	0.576	0.591	0.613	0.638	0.665	0.697	0.733
13.0	0.512	0.533	0.557	0.571	0.590	0.612	0.636	0.664	0.695
14.0	0.499	0.517	0.539	0.552	0.570	0.589	0.611	0.635	0.662
15.0	0.486	0.503	0.523	0.535	0.551	0.568	0.588	0.609	0.633
16.0	0.474	0.490	0.509	0.519	0.534	0.550	0.567	0.586	0.607
17.0	0.463	0.478	0.495	0.505	0.518	0.533	0.548	0.566	0.585
18.0	0.452	0.466	0.482	0.492	0.504	0.517	0.531	0.547	0.564
19.0	0.442	0.456	0.471	0.479	0.491	0.503	0.516	0.530	0.546
20.0	0.433	0.446	0.460	0.468	0.478	0.489	0.502	0.515	0.529

NOTE 1. $\lambda_F = \dfrac{\ell_e t_1}{r_y D}$, $i = \dfrac{I_c}{I_c + I_t}$, $i = 0.5$ when flanges are equal.

NOTE 2. Intermediate values to the right of the stepped line should be determined from the formula given in note 3 rather than by interpolation between the tabulated values.

NOTE 3. $v = [\{4i(1-i) + 0.05\lambda_F{}^2 + \psi_i{}^2\}^{0.5} + \psi_i]^{-0.5}$

where

$\psi_i = 0.8\ (2i - 1)$, when $I_c \geq I_t$

$\psi_i = 2i - 1$, when $I_c < I_t$

9.7.3.2 *Uniform solid rectangular sections.* The value of λ_{LT} for a beam of homogeneous solid rectangular section, which is uniform between points of full lateral restraint to the compression flange, should be taken as:

$$\lambda_{LT} = 2.8\eta \frac{\sqrt{\ell_e D}}{B}$$

where

- η is as defined in **9.7.2**
- ℓ_e is determined in accordance with **9.6**
- D is the depth of the section in the plane of bending
- B is the width of the section.

9.7.4 ***Varying sections.*** The value of λ_{LT} for a beam of varying section should be taken as:

$(1.5 - 0.5\rho_I)$ times the value obtained from **9.7.2** or **9.7.3** using the values of r_y and v appropriate to the point of maximum bending moment

where

$$\rho_I = \frac{\text{minimum total area of two flanges at any section in } L}{\text{maximum total area of two flanges at any section in } L}$$

L is the distance between points of lateral restraint to the compression flange.

9.7.5 *Other cases and alternative methods.* For cases not covered by 9.7.2, 9.7.3 or 9.7.4, or as an alternative, λ_{LT} may be taken as:

$$\lambda_{LT} = \sqrt{\frac{\pi^2 ES}{\sigma_{ci}}}$$

where

- S is as defined in **9.7.3.1**
- σ_{ci} is the maximum compressive bending stress in the beam when, under the given pattern of loading, the beam reaches its theoretical elastic critical buckling condition as determined by an elastic analysis.

9.8 Limiting compressive stress

9.8.1 ***General.*** The limiting compressive stress σ_{lc} should be determined from **9.8.2** or **9.8.3**, as appropriate to the type of section and the value of the basic limiting stress, σ_{li}.

The value of σ_{li}/σ_{yc} should be obtained from figure 10 according to the value of:

$$\lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$$

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9.7.4 Varying sections

Add at end:

For the purpose of determining the limiting stress at the minimum section the value of λ_{LT} shall be obtained from **9.7.2** or **9.7.3** assuming that the minimum section is uniform throughout L.

Add new Clause 9.7.6

9.7.6 Slenderness limitations for plastic analysis

The slenderness parameter $\lambda_{LT}\sqrt{\sigma_{yc}/355}$ for sections which are assessed using plastic methods of analysis (see **7.4**) shall not exceed 30.

9.8.1 General

Add at end:

Where in assessment of the adequacy of a beam allowance is to be made for initial departures from straightness of the flanges Δ_F, measured in accordance with Table 5 of Part 6, σ_{li} shall be calculated from the equation in Appendix G7 with η taken as:

$$\eta = 0.005(\beta - 45) + \left(\frac{\beta - 45}{\beta}\right)[1.2\Delta_F - 0.0012l]\frac{y}{r_y^2}$$

but not less than zero

where

Δ_F is the greater of the values measured in accordance with 4(a) and 4(b) respectively of Table 5 of B.S. 5400: Part 6 over a gauge length equal to the length of the beams between points of effective lateral support.

y is the distance in the x-direction from the y-y centroidal axis to the extreme fibre of the compression flange (see Figure 1).

r_y is the radius of gyration of the gross cross section about its y-y axis.

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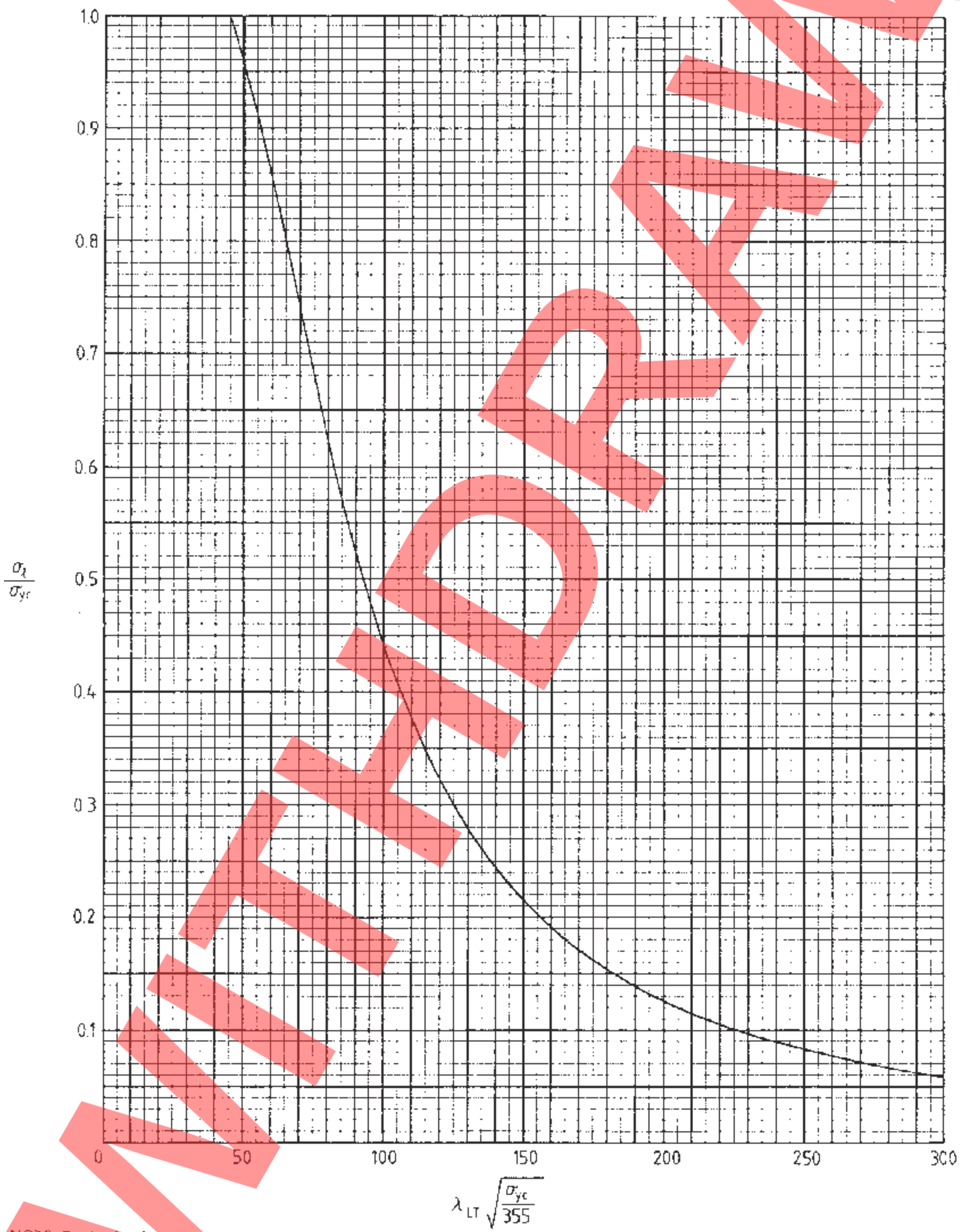
where

λ_{LT} is the slenderness parameter derived in accordance with 9.7

σ_{yc} is the nominal yield stress of the compressive flange material except that, where gross section properties have been used for the effective section for unstiffened flanges in accordance with note 2 of **9.4.2.4**, σ_{yc} should be taken as K_c times the nominal yield stress, where K_c is obtained from the unrestrained curve of figure 5 corresponding to the slenderness ratio λ of the compression flange.

If the bending moment changes sign between adjacent points of lateral restraint, σ_{li} should be calculated separately for each sagging and hogging zone, with the appropriate values of λ_{LT} in accordance with 9.7.



NOTE. For basis of curve, see **G.7**.

Figure 10. Basic limiting stress σ_{li}

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9.8.2 *Compact sections.* The limiting compressive stress, σ_{lc} for compact sections (see 9.3.7) should be taken as σ_{li} as derived in 9.8.1.

9.8.3 *Non-compact sections.* The limiting compressive stress, σ_{lc} for non-compact sections, should be taken as the lesser of:

$$\frac{D\sigma_{li}}{2y_t} \text{ or } \sigma_{yc}$$

where

σ_{li} is as derived in 9.8.1
σ_{yc} is as defined in 9.8.1
D is the overall depth of the section, including the concrete in composite construction
y_t is as defined in 9.7.3.1.

9.9 Beams without longitudinal stiffeners

9.9.1 *Bending resistance*

9.9.1.1 *General.* Beams should be designed in accordance with 9.9.1.2 or 9.9.1.3, as appropriate.

Beams with flanges curved in elevation should be designed in accordance with 9.10 and 9.11. Beams constructed in stages in which the loading and section properties change should be in accordance with 9.9.5. Effects due to differential temperature and concrete shrinkage should be taken into account in accordance with 9.9.7. Unsymmetric beams should be additionally checked for the serviceability limit state in accordance with 9.9.8.

9.9.1.2 *Compact sections.* The bending resistance M_D of a beam which is of compact section, as defined in 9.3.7, should be taken as:

$$M_D = \frac{Z_{pe}\sigma_{lc}}{\gamma_m\gamma_{f3}}$$

where

σ_{lc} is the limiting compressive stress derived in accordance with 9.8.2
Z_{pe} is the plastic modulus of the effective section derived in accordance with 9.4.2.

NOTE. If the section is composed of steels having different values of nominal yield stress, Z_{pe} should be based on the transformed section having a nominal yield stress equal to that of the compression flange.

For composite sections the transformed area of the concrete compression flange should be obtained from:

$$\text{the concrete flange area} \times \frac{0.4 f_{cu}}{\sigma_{yc}/\gamma_m}$$

where

f_{cu} is the concrete cube strength in accordance with Part 4
σ_{yc} is the nominal yield stress of the steel compression flange.

Concrete in tension should be ignored but the transformed area of the reinforcement in concrete subject to tension should be included and obtained from:

$$\text{the gross area of reinforcement} \times \frac{0.87 f_y}{\sigma_{yc}/\gamma_m}$$

where f_y is the characteristic strength of the reinforcement in accordance with Part 4.

9.9.1.3 *Non-compact sections.* The bending resistance M_D of a beam which is not of compact section (as defined in 9.3.7) should be taken as the lesser of:

(a) $M_D = \dfrac{Z_{xc}\sigma_{lc}}{\gamma_m\gamma_{f3}}$, or

(b) $M_D = \dfrac{Z_{xt}\sigma_{yt}}{\gamma_m\gamma_{f3}}$

where

Z_{xc} and Z_{xt} are the elastic moduli of the section with respect to the extreme compression and to the extreme tension fibre, respectively, based on the effective section derived in accordance with 9.4.2
σ_{lc} is the limiting compressive stress derived from 9.8.3
σ_{yt} is the nominal yield stress of the tension flange material.

9.9.2 *Shear resistance*

9.9.2.1 *General.* The shear resistance of a web of a beam with transverse stiffeners at supports and with or without intermediate transverse stiffeners should be determined in accordance with 9.9.2.2 provided that:

(a) there are no longitudinal stiffeners on the web or the compression flange;
(b) the web panel considered has no openings other than those within the limits set out in 9.3.3.2 (a), (b) or (c);
(c) the provisions of 9.9.4 and 10.6 are met if the beam is subjected to axial load;
(d) the flanges are parallel and straight in elevation.

Web panels which do not meet these conditions should be designed in accordance with 9.11.

9.9.2.2 *Shear resistance under pure shear.* The shear resistance V_D of a web panel under pure shear should be taken as:

$$V_D = \left[\frac{t_w(d_w - h_h)}{\gamma_m\gamma_{f3}}\right]\tau_l$$

where

t_w is the thickness of the web
d_w $= D$, the overall depth of a rolled section, or is the depth of the web measured clear between flanges of a fabricated section
h_h is the height of the largest hole or cut-out if any, within the panel being considered, but in the case of beams without intermediate transverse stiffeners the hole or cut-out may be ignored at sections further than $1.5h_h$ longitudinally from the edge of the hole
τ_l is the limiting shear strength of the web panel determined from figures 11 to 17 corresponding to the values of τ_y, ϕ, m_{fw} and the slenderness ratio λ given by:

$$\lambda = \frac{d_{we}}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

If the value of τ_l/τ_y from figures 11 to 17 is less than the value of the shear co-efficient K_q for an unrestrained panel from figure 22, the value of this ratio may be taken as K_q.

d_{we} is the depth of web clear between flange plates for welded sections, or the depth of web between toes of angles connecting the web to the flanges for riveted/bolted construction, or is the depth of web clear of root fillets for rolled sections

$$\tau_y = \frac{\sigma_{yw}}{\sqrt{3}}$$

σ_{yw} is the nominal yield stress of the web material

ϕ $= \dfrac{a}{d_{we}}$, the aspect ratio of the panel

a is the clear length of panel between transverse stiffeners

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9.8.3 Non-compact sections

Add at end:

Alternatively, the limiting compression stress, σ_{lc}, for non-compact sections may be calculated as follows:

(a) For composite construction where the compression flange of the non-compact section under consideration is attached to a concrete or composite slab by shear connection and the overall width of the slab is not less than the depth of the steel section, σ_{lc} shall be taken as σ_{yc}

For non-compact sections with $\lambda_{LT}\sqrt{\sigma_{yc}/355} <$ 45, σ_{lc} shall be taken as the lesser of

$$S\sigma_{li}\left[1.01 - 0.006\left(\lambda_{LT}\sqrt{\sigma_{yc}\ /\ 355 - 45}\right)\right] \ or\ \sigma_{yc}$$

(c) For other non-compact sections, σ_{lc}, shall be taken as the lesser of:

$$S\sigma_{li} \text{ or } \sigma_{yc}$$

where

σ_{li} is as derived in **9.8.1**
σ_{yc} is as defined in **9.8.1**
$S = Z_{pe}/Z_{xc}$
Z_{pe} is the plastic modulus of the effective section derived in accordance with **9.4.2.**
Z_{xc} is the elastic modulus of the section with respect to the extreme compression fibre, based on the effective section derived in accordance with **9.4.2.**

9.9.1.3 Non-compact sections

Add the following NOTE at the end:

NOTE: For composite sections, Z_{xc} and Z_{xt} shall be based on the transformed section. The transformed area of the concrete compression flange shall be obtained using either the short-term or the long-term modulus of elasticity of the concrete as appropriate to the type of loading. Concrete in tension shall be ignored but the area of the longitudinal reinforcement shall be included.

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$m_{fw} = \dfrac{\sigma_{yf} b_{fe} t_f^2}{2 \sigma_{yw} d_{we}^2 t_w}$ taking the smaller value at the top or the bottom flanges and ignoring any concrete,

b_{fe} is the smallest of:

(a) $10 t_f \sqrt{\dfrac{355}{\sigma_{yf}}}$ or

(b) the distance from the mid-plane of the web to the nearer edge of the flange, i.e. taken as zero if there is no flange outstand; or

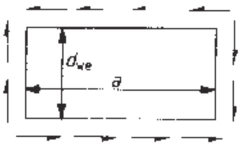
(c) if there are two more webs, half the clear distance between webs.

t_f is the flange plate thickness

σ_{yf} is the nominal yield stress of the flange material.

Notes to figures 11 to 17

NOTE 1. $\phi = \dfrac{a}{d_{we}}$ shown as follows:



NOTE 2. For definitions of a, d_{we}, t_w, σ_{yw}, τ_y and m_{fw}, see **9.9.2.2.**

NOTE 3. Values of τ_l/τ_y above 1.0, plotted as dotted lines, are only to be used for interpolation purposes.

NOTE 4. For basis of curves, see **G.8.**

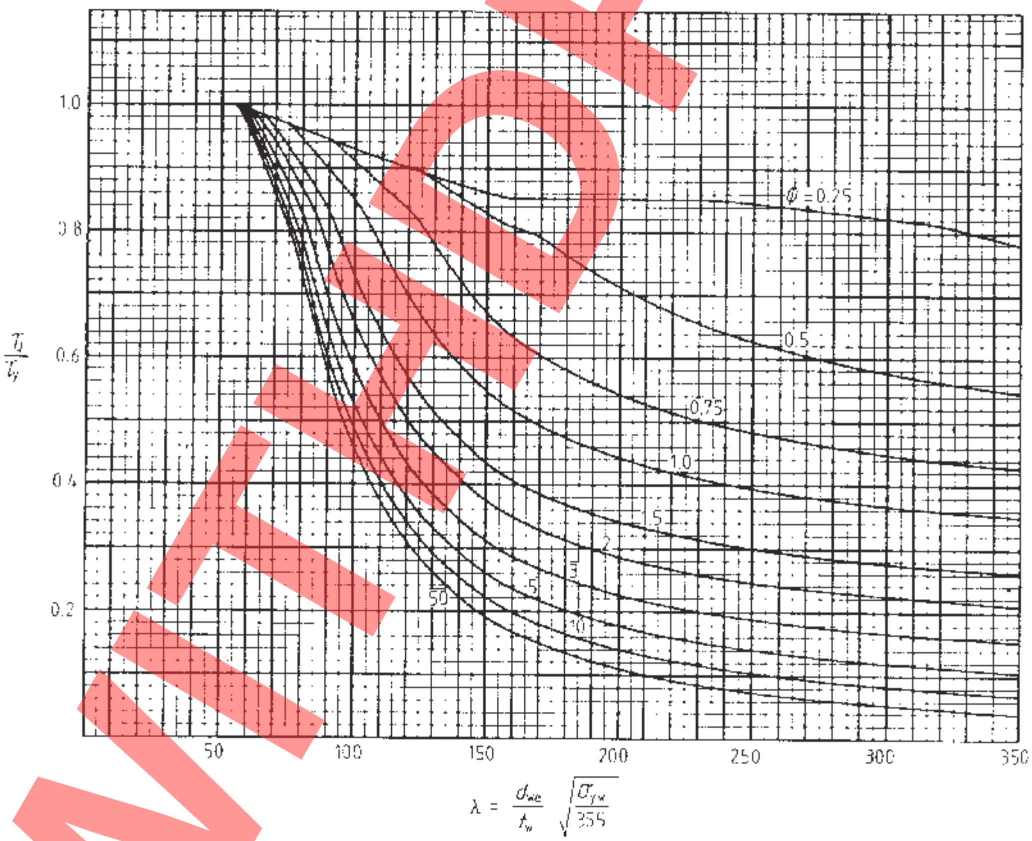


Figure 11. Limiting shear strength τ_l for $m_{fw} = 0$

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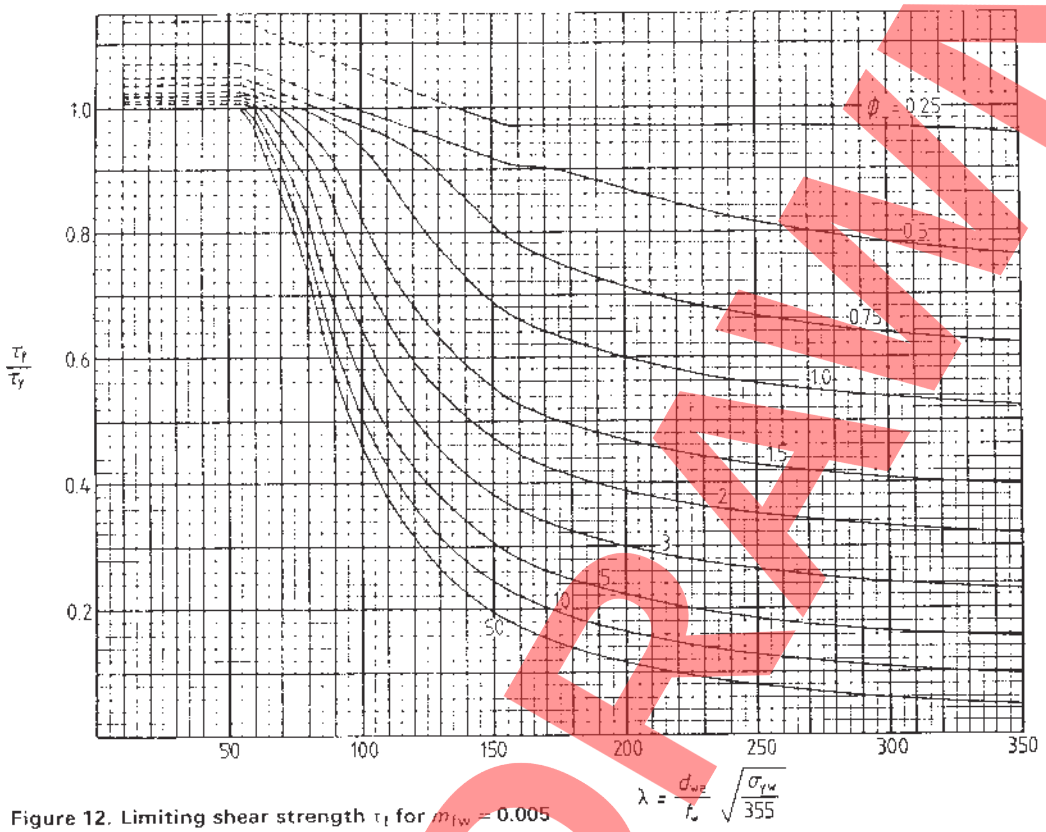

Figure 12. Limiting shear strength τ_l for $m_{fw} = 0.005$

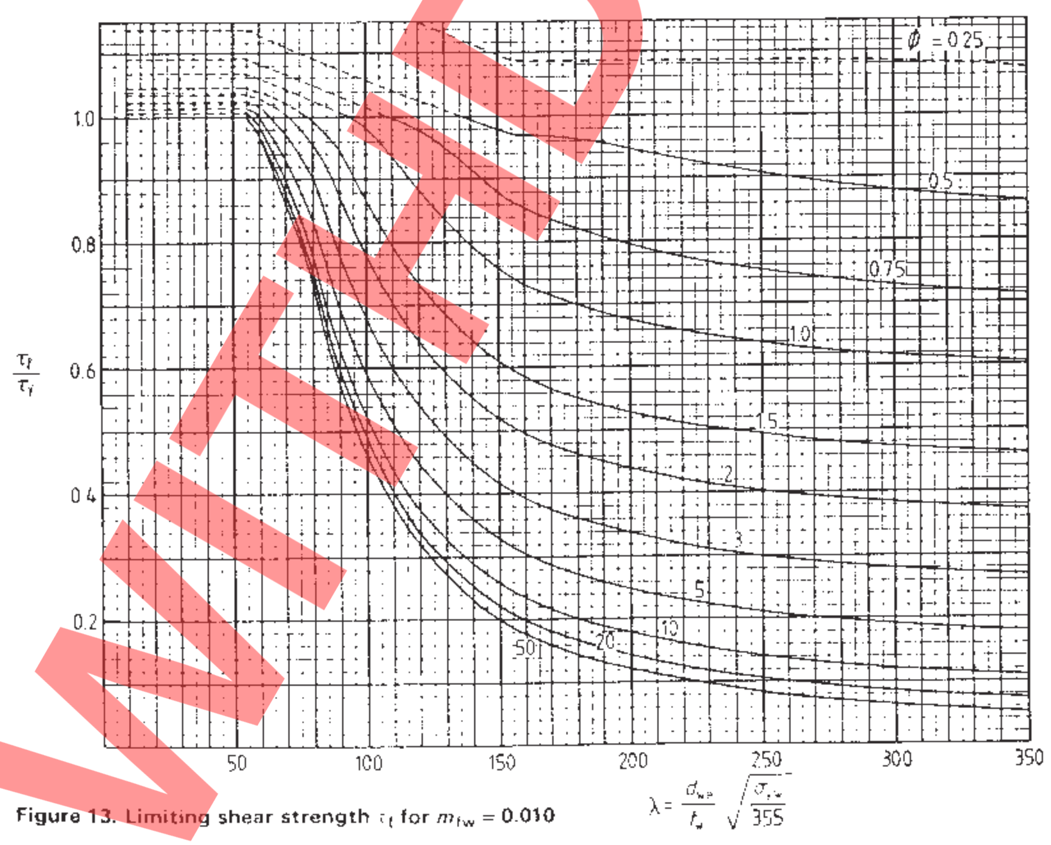

Figure 13. Limiting shear strength τ_l for $m_{fw} = 0.010$

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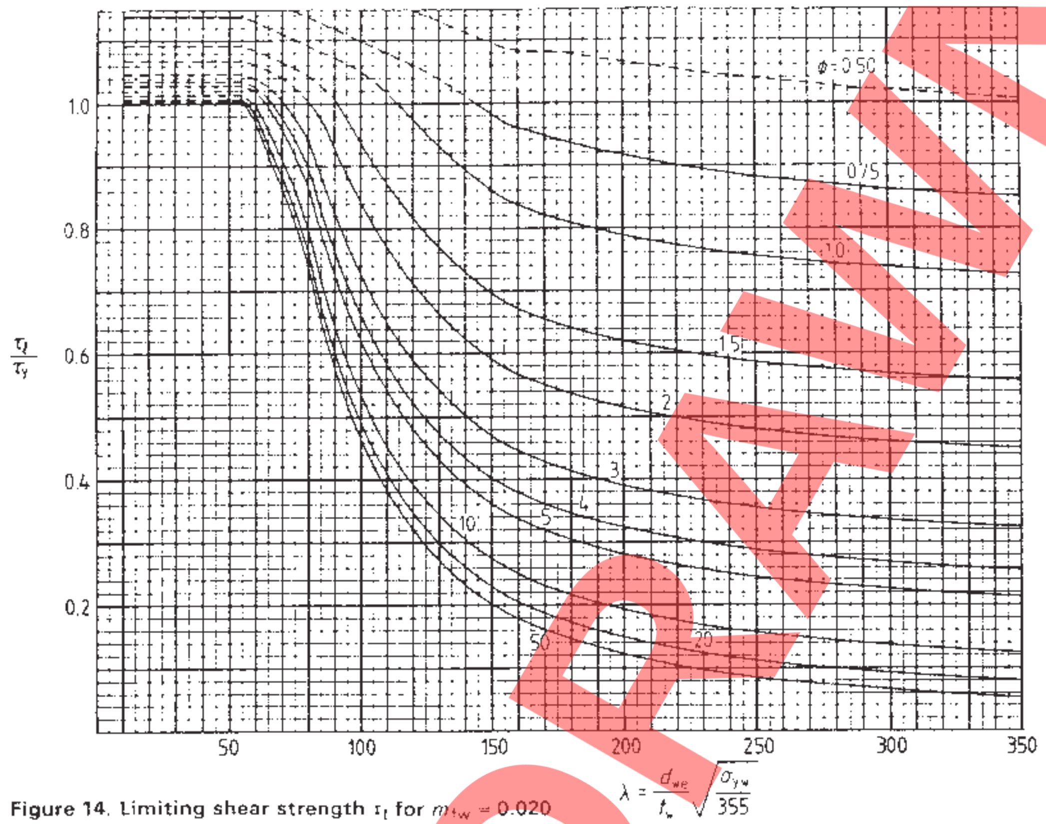

Figure 14. Limiting shear strength τ_l for m_{fw} = 0.020

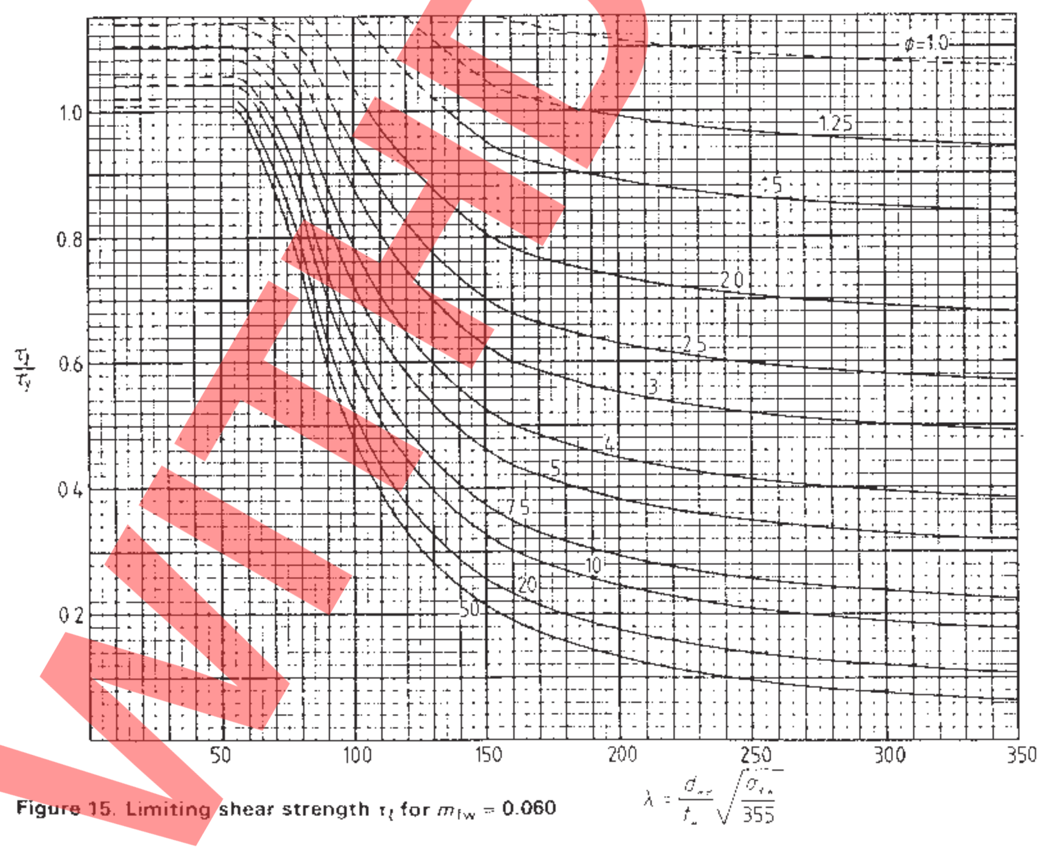

Figure 15. Limiting shear strength τ_l for m_{fw} = 0.060

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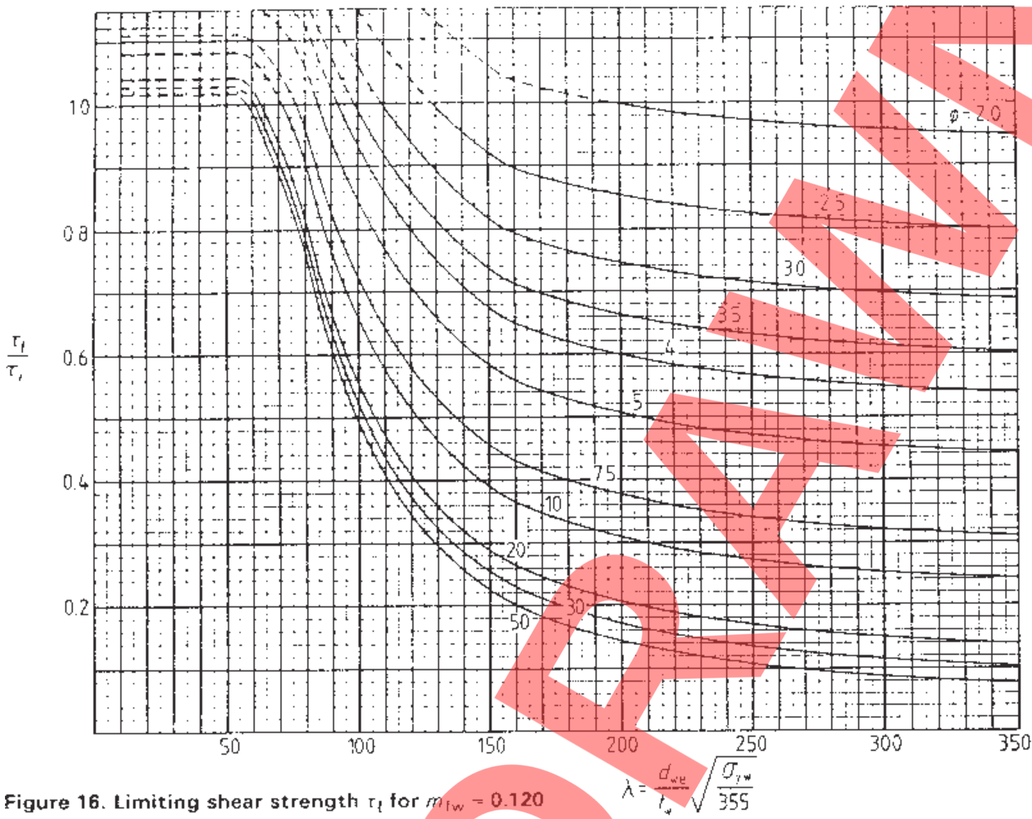

Figure 16. Limiting shear strength τ_ℓ for m_{fw} = 0.120

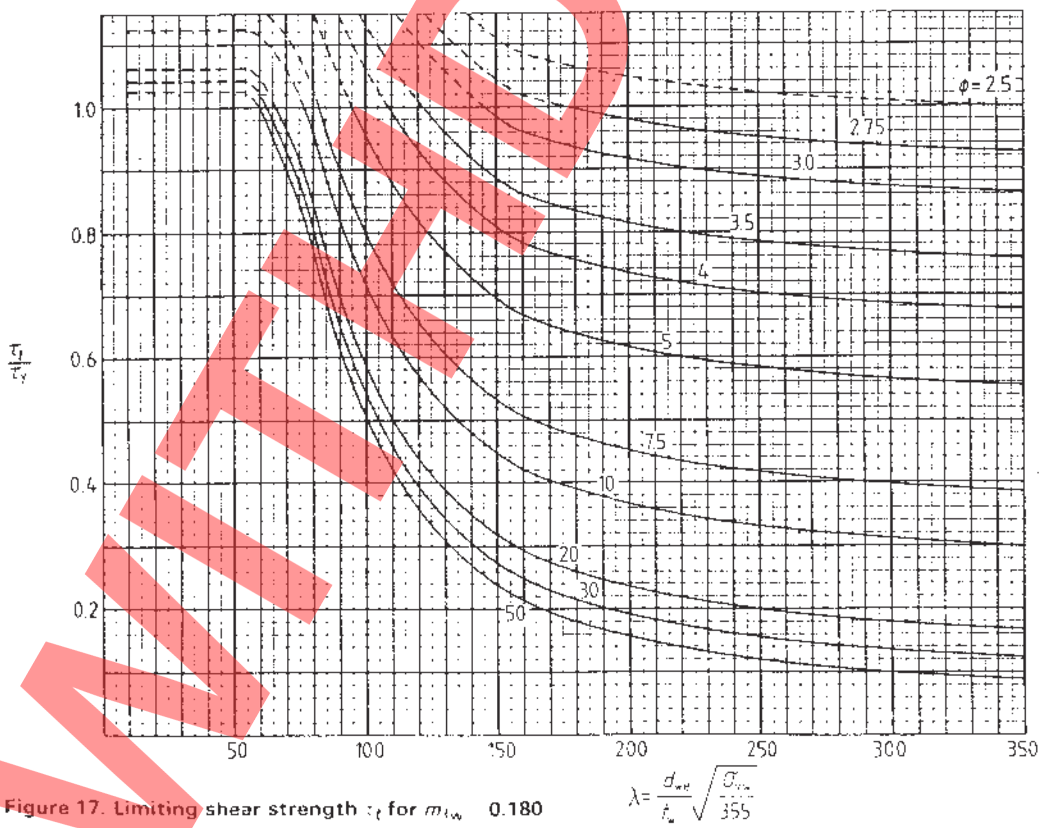

Figure 17. Limiting shear strength τ_ℓ for m_{fw} 0.180

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9.9.3 *Combined bending and shear*

9.9.3.1 *Webs with intermediate transverse stiffeners.* Beams should be in accordance with the following:

(a) $V \leqslant V_D$

(b) $M \leqslant M_D$

(c) if $M > M_R$, then $\frac{M}{M_D} + \left(1 - \frac{M_R}{M_D}\right)\left(\frac{2V}{V_R} - 1\right) \leqslant 1$

(d) if $V > V_R$, then $\frac{V}{V_D} + \left(1 - \frac{V_R}{V_D}\right)\left(\frac{2M}{M_R} - 1\right) \leqslant 1$

For all rolled I and channel sections of grade 43 and grade 50 steel in accordance with BS 4 or BS 4848, and for other beams for which $V_D = V_R$, the equations given in (c) and (d) are reduced to:

$$\text{if } M > M_R, \ \frac{M}{M_D} + \left(1 - \frac{M_R}{M_D}\right)\left(\frac{2V}{V_D} - 1\right) \leqslant 1$$

where

V is the maximum shear force in the panel

V_D is the shear capacity of the panel under pure shear determined in accordance with **9.9.2.2**

V_R is the value of V_D obtained by taking $m_{fw} = 0$ when applying **9.9.2.2**

M is the maximum bending moment within the length of the panel

M_D is the bending resistance of the beam determined in accordance with **9.9.1**

$M_R = \frac{F_f d_f}{\gamma_m \gamma_{f3}}$ but not greater than M_D

d_f is the distance between the centroids of the two flanges; for a composite flange the distance should be measured from the centroid of the transformed flange section

F_f $= \sigma_f A_{fe}$, the limiting force in the flange, to be taken as the lower value for the two flanges

σ_f $= \sigma_{yt}$ the nominal yield stress for the tension flange material, or

$= \sigma_{lc}$, the limiting compressive stress derived in accordance with **9.8** for the compression flange

A_{fe} is the area of the effective flange section derived in accordance with **9.4.2**.

NOTE 1. Bending moments up to a value M_D can be resisted by a beam if the shear force V is less than 0.5 V_R.

NOTE 2. Shear forces up to a value V_D can be resisted by a beam if the bending moment M is less than 0.5 M_R.

9.9.3.2 *Webs with transverse stiffeners at the supports only.* For a web having transverse stiffeners at support positions only, the provisions of **9.9.3.1** should be applied at all sections of the beam with V and M defined as follows:

V is the shear force at any section of the beam

M is the coexistent bending moment at the same section of the beam.

9.9.4 *Combined bending and axial load*

9.9.4.1 *Yielding of beam.* All points at all sections of a beam subjected to combined bending and axial load should be such that:

$$\frac{P}{A_e} \pm \frac{M_x}{Z_x} \pm \frac{M_y}{Z_y} \leqslant \frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

P is the axial load in the beam at the section under consideration

M_x, M_y are the co-incident bending moments about the X-X and Y-Y axes respectively

A_e is the effective area of the beam, calculated at the section under consideration in accordance with **10.5** or **11.3**, as appropriate

Z_x, Z_y are the elastic moduli of the effective beam section about the X-X and Y-Y axes, respectively, at the section under consideration, derived from **9.4.2**

σ_y is the nominal yield stress of the part of the section under consideration.

A beam subjected to combined bending and axial tension should also be in accordance with the provisions of **11.5.2**.

9.9.4.2 *Buckling of beam.* A beam subjected to combined bending and axial compression should be such that:

$$\frac{P_{max}}{P_D} + \frac{M_{x\,max}}{M_{Dx}} + \frac{M_{y\,max}}{M_{Dy}} \leqslant 1$$

where

P_{max}, $M_{x\,max}$, $M_{y\,max}$ are the maximum axial load, and bending moments about the X-X and Y-Y axes, respectively (see figure 1), within the middle-third of the length of the beam between points of restraint

P_D is the axial resistance derived in accordance with **10.6.1**.

M_{Dx}, M_{Dy} are the corresponding bending resistances of the beam, with respect to extreme compression fibres, determined in accordance with **9.9.1**.

A beam subjected to combined bending and axial tension should also be in accordance with **11.5.2**.

9.9.4.3 *Compact and stocky members.* As an alternative to **9.9.4.1** and **9.9.4.2**, compact sections subjected to combined bending and axial compression may be designed in accordance with **10.6.3**, provided that they also meet the provisions for slenderness for stocky members contained therein.

9.9.5 *Beams built in several stages*

9.9.5.1 *General.* When the cross-section of a beam and the applied loading increase by stages, e.g. a steel section initially carrying self-weight and weight of concrete deck but acting compositely for subsequently applied loads, a check for adequacy should be made for each stage of construction.

9.9.5.2 *Compact sections.* For beams that are of compact section, as defined in **9.3.7**, the entire load at any stage may be assumed to act on the cross-section of the beam appropriate to that stage.

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9.9.4.2 Buckling of beam

Add at end:

Alternatively, a uniform beam of I section subject to combined bending and axial compression is deemed to pass assessment if the following criteria are satisfied:

$$\frac{\gamma_m \gamma_{f3} P_{max}}{A_e} + \frac{\gamma_m \gamma_{f3} M_{xmax}}{Z_{xc}} + \frac{\gamma_m \gamma_{f3} M_{ymax} N_y}{Z_{yc}} + \pi^2 E \eta \left(\frac{r_y}{l_e}\right)^2 \left(\frac{K_y}{1 - K_y}\right) \leq \sigma_{yc}$$

where

A_e is the effective cross sectional area of the beam as defined in 10.5.2

Z_{xc} is the section modulus with reference to the x-x axis and the extreme fibres of the compression flange

Z_{yc} is the section modulus with reference to the y-y axis and the extreme fibres in bending compression

$$N_y = 1 + \frac{4}{\pi}\left(\frac{K_y}{1 - K_y}\right)\left(1 + \left(\frac{M_{xmax}}{M_{cr}}\right) v^2\right)$$

$$K_y = \frac{P_{max}\, l^2{}_e}{\pi^2 E I_y} + \left(\frac{M_{xmax}}{M_{cr}}\right)^2$$

$$K_p = P_{max}\, l_e^2 \;/\, \pi^2 E I_y$$

$$M_{cr} = \frac{\pi^2 E\, Z_{xc}}{\lambda^2{}_{LT}}$$

σ_{yc} is as defined in **9.8.1**
λ_{LT} is as defined in **9.7.2**
v is as defined in **9.7.2**
l_e is as defined in **9.7.2**

$$\eta = 0.0062\left(\frac{l_e}{r_y} - 15\right)\frac{K_p}{K_y} + 0.005\,(\beta - 45)\,\frac{M^2{}_{xmax}}{M^2{}_{cr} K_y}\left(1 + \frac{v^2 M_{cr}}{M_{xmax}}\right)$$

β is as defined in Appendix G.7

NOTE: The value of γ_m separately associated with P_{max}, M_{xmax} and M_{ymax} shall be in accordance with Table 2, with due account of the γ_m value associated with M_{DX} and M_{DY} as derived in **9.9.1** and replaced above by the relevant $Z_c \sigma_{yc}/\gamma_m \gamma_{f3}$ term in the criterion above. 37

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9.9.5.3 *Non-compact sections.* For the beam that is not of compact section, as defined in 9.3.7, the stresses appropriate to the cross-section and the loading at each stage of construction should be calculated. The sum of the stresses at each stage of construction should be calculated separately for bending about each axis and for axial load.

The stress at an extreme fibre due to bending about one axis should not exceed

(a) $\dfrac{\sigma_{lc}}{\gamma_m \gamma_{f3}}$ if compressive, or

(b) $\dfrac{\sigma_{yt}}{\gamma_m \gamma_{f3}}$ if tensile.

In the interaction formulae in 9.9.3 and 9.9.4.2, V_D, V_R, M_D, M_R, P_D, M_{DX} and M_{DY} should be taken appropriate to the cross-section at the stage under consideration. The applied moments should be taken as follows:

$\sigma_{xx} Z_x$ for M and M_{xmax}

$\sigma_{yy} Z_y$ for M_{ymax}

The total stresses at all points at all sections should not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_{lc} and σ_{yt} are as defined in 9.9.1.3, appropriate to the cross-section at the stage under consideration.
σ_{xx} and σ_{yy} are the sums to the stage considered of the stresses of the extreme fibres of the section due to bending about the X-X and Y-Y axes respectively.
Z_x and Z_y are the elastic moduli of the effective section for the stage considered about the X-X and Y-Y axes, respectively, for the corresponding extreme fibres.

9.9.6 *Webs subjected to in-plane patch loading.* The effects of in-plane patch loading on a longitudinal edge of web should be taken into account if the transverse stress σ_2 in the web plate caused by this loading is greater than:

$$3\sigma_{yw} \frac{t_w}{\sqrt{w d_{we}}} \sqrt{\frac{355}{\sigma_{yw}}}$$

where

- w is the width of the patch loading along the span of the beam (see figure 6)
- t_w is the web plate thickness
- d_{we} is the depth of web as defined in 9.9.2.2
- σ_{yw} is the nominal yield stress of the web material.

A method for checking the adequacy of a web under patch loading is given in appendix D.

9.9.7 *Differential temperature and concrete shrinkage.* When, as required by 9.2.1 or 9.2.3, differential temperature and shrinkage effects are to be taken into account, the effects should be separated into the following parts:

(a) Stresses forming the internal stress distribution through the section, ignoring any continuity over supports.

(b) Bending moments and shears due to requirements for continuity over supports in a continuous beam.

For the strength checks contained in 9.9.1 to 9.9.4, the values of bending moments and shears from (b) should be combined with other load effects as appropriate.

For serviceability limit state the stresses calculated from (a) should be added to the stresses due to load effects (including the moments from (b) above at appropriate points on the section). The resultant total stresses should not exceed:

$$\frac{\sigma_{lc}}{\gamma_m \gamma_{f3}} \quad \text{or} \quad \frac{\sigma_{yt}}{\gamma_m \gamma_{f3}} \quad \text{as appropriate.}$$

9.9.8 *Serviceability check for unsymmetric cross sections.* The smaller flange of unsymmetric beams designed as compact should be checked for the serviceability limit state, treating the beams as non-compact.

9.10 Flanges in beams with longitudinal stiffeners in the cross-section

9.10.1 *Strength of unstiffened flanges*

Clause 9.10.1.1 *Flanges straight in elevation.* The stresses in the extreme fibres of a beam with longitudinal stiffeners on the web, including any re-distribution of stresses from the web, should not exceed:

(a) $\dfrac{\sigma_{lc}}{\gamma_m \gamma_{f3}}$ in compression;

(b) $\dfrac{\sigma_{yf}}{\gamma_m \gamma_{f3}}$ in tension;

where

σ_{lc} is as defined in 9.8
σ_{yf} is the nominal yield stress of the flange material.

9.10.1.2 *Flanges curved in elevation.* Flanges curved in elevation should be in accordance with 9.10.1.1.

Additionally the stresses in the flange plate including those due to flange curvature (as calculated in accordance with 9.5.7.1) should be in accordance with 9.10.2.1.

9.10.2 *Strength of stiffened flanges*

9.10.2.1 *Yielding of flange plate.* The design of the flange plate should satisfy the following yield criterion:

$$\sigma_f^2 + \sigma_2^2 - \sigma_f\sigma_2 + 3\tau^2 \leqslant \left(\frac{\sigma_{yf}}{\gamma_m \gamma_{f3}}\right)^2$$

where

- σ_f is the longitudinal stress at the mid-plane of the flange plate, including any re-distribution of str from the web, treated as positive when compressive
- σ_2 is the co-existent in-plane transverse stress at the mid-plane of the flange plate, treated as positive when compressive, due to bending of cross beams or diaphragms, or due to curvature (see 9.5.7.1)
- τ $= \tau_1 + 0.5\tau_2$
- τ_1 is the in-plane shear stress in the flange plate due to torsion on a box beam
- τ_2 is the shear stress in the flange plate at the junction with the web of the beam due to shear force on the beam
- σ_{yf} is the nominal yield stress of the flange plate material.

9.9.7 Differential temperature and concrete shrinkage.

In the last paragraph, insert the following after 'from (b)':

taking into account the effect of shear lag.

9.9.8 Serviceability check for unsymmetric cross-sections

Add at end:

However in assessment where

$$\rho_{ft} \geq 0.016\, k^2 + \frac{0.16}{k}(\rho_D)^k$$

the section need not be checked for the serviceability limit state when the partial load factors as given in BD 37/88 have been used.

In this expression

k = 2 for steel beams; or 1 for composite beams

ρ_{ft} is the proportion of the sectional area of the tension flange of the beam to the total area of the beam

ρ_D is the proportion of unfactored dead load (ie not including superimposed) moments to unfactored total moments.

Where the compression flange area is less than the tension flange area, ea. in a non-composite stage of construction, ρ_{ft} shall be based on the ratio of the compression flange area to the total area of the beam.

NOTE: For composite beams the transformed section appropriate to live load shall be used for the relevant sectional area, ie short term modulus.

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9.10.2.2 *Effective section for longitudinal flange stiffeners.* The effective section of a longitudinal stiffener should be taken as the stiffener combined with a width of flange plate equal to $0.5K_c b$ on each side of the stiffener,

where

K_c is obtained from figure 5 (in accordance with **9.4.2.4**) for a compression flange, or
$= 1.0$ for a tension flange

b is the spacing of longitudinal stiffeners.

9.10.2.3 *Strength of longitudinal flange stiffeners.* The design of a longitudinal stiffener should be such that:

(a) $\sigma_a + 2.5\tau_1 k_{s1} < \dfrac{k_{\ell 1}\sigma_{ys}}{\gamma_m \gamma_{f3}}$, and

(b) $\sigma_a + 2.5\tau_1 k_{s2} < \dfrac{k_{\ell 2}\sigma_{ye}}{\gamma_m \gamma_{f3}}$

where

σ_a is the longitudinal stress including any re-distribution of stresses from the web, positive when compressive, at the centroid of the effective section of the stiffener

τ_1 is the in-plane shear stress in the flange plate due to torsion on the beam, always to be taken as positive

σ_{ys} is the nominal yield stress of the stiffener material

$\sigma_{ye} = \sqrt{\sigma_{yf}^2 - 3\tau^2\gamma_m^2\gamma_{f3}^2}$

τ is as defined in **9.10.2.1**

σ_{yf} is the nominal yield stress of the flange plate material

$k_{\ell 1}$ and $k_{\ell 2}$ are values of the reduction factor k_ℓ obtained from figure 18

k_{s1} and k_{s2} are coefficients obtained from figure 18

In using figure 18:

$\lambda = \dfrac{\ell}{r_{se}}\sqrt{\dfrac{\sigma_{ys}}{355}}$ when obtaining $k_{\ell 1}$ and k_{s1}

$\lambda = \dfrac{\ell}{r_{se}}\sqrt{\dfrac{\sigma_{ye}}{355}}$ when obtaining $k_{\ell 2}$ and k_{s2}

$\eta = \dfrac{y_o \Delta}{r_{se}^2}$ when obtaining $k_{\ell 1}$

$\eta = \dfrac{y_z \Delta}{r_{se}^2}$ when obtaining $k_{\ell 2}$

where

ℓ is the spacing of cross beams and/or diaphragms which restrain longitudinal stiffeners (for flanges not stiffened transversely see **9.10.4**)

r_{se} is the radius of gyration of the effective section of a longitudinal stiffener about the centroidal axis parallel to the flange plate

y_o is the distance from the centroid of the effective stiffener section to the point on the stiffener furthest from the plate

y_z is the distance from the centroid of the effective stiffener section to the mid-plane of the flange plate

$\Delta = \dfrac{\ell}{625} + \dfrac{r_{se}^2}{2y_{Bs}} + \dfrac{e_t}{2}$

y_{Bs} is the distance from the centroid of the effective stiffener section to the neutral axis of the cross section of the beam (determined in accordance with **9.4.2**)

NOTE. The neutral axis occurs where the total longitudinal stress is zero.

e_t is the greatest offset of the flange plate from a straight line of length ℓ, due to specified camber or curvature.

9.10.2.4 *Longitudinally varying moment.* If the longitudinal stress in the flange varies within the length ℓ, the provisions of **9.10.2.1** should be satisfied at all sections within the length ℓ, and the provisions of **9.10.2.3** should be satisfied with σ_a taken at a point 0.4ℓ from the end where the stress is greater.

9.10.3 ***Stiffened flanges subjected to local bending***

9.10.3.1 *Strength.* Stiffened flanges subjected to bending due to wheel or other local loads in addition to in-plane stresses should satisfy the provisions of **9.10.3.2** and **9.10.3.3**.

9.10.3.2 *Ultimate limit state.* Provided that the provisions of **9.10.3.3** are satisfied, no account need be taken of local bending stresses when checking a stiffened flange at the ultimate limit state. Under in-plane forces the provisions of **9.10.2.1** to **9.10.2.4** should be satisfied.

9.10.3.3 *Serviceability limit state*

9.10.3.3.1 *Flange plate.* The design of the flange plate should satisfy the following yield criterion at all sections:

$$(\sigma_{fz}+\sigma_f)^2+(\sigma_2+\sigma_{2b})^2-(\sigma_{fz}+\sigma_f)(\sigma_2+\sigma_{2b})+3\tau^2 \le \left(\frac{\sigma_{yf}}{\gamma_m\gamma_{f3}}\right)^2$$

where

σ_f, σ_2 and τ are as defined in 9.10.2.1 due to global effects

σ_{fz} is the stress at the mid-plane of the flange plate due to local bending of the effective stiffener section spanning between transverse members

σ_{2b} is the stress due to local bending at the extreme fibre of the flange plate spanning between longitudinal stiffeners and transverse membrane action.

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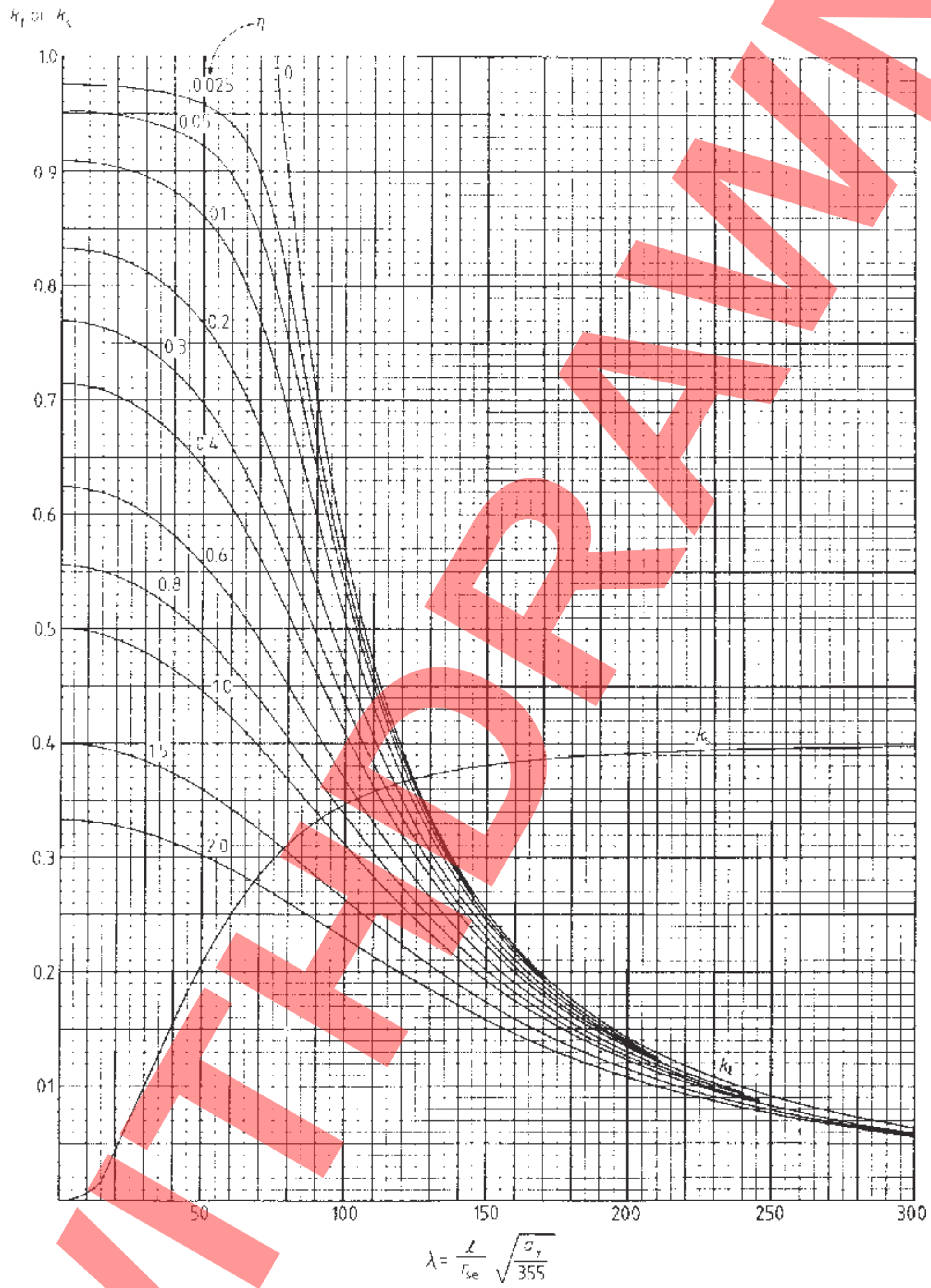

NOTE 1. For appropriate values of η, σ_y and ℓ to be used in obtaining k_f and k_s, see 9.10.2.3

NOTE 2. For basis of curves, see G.9.

Figure 18. Parameters for the design of longitudinal flange stiffeners

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9.10.3.3.2 *Longitudinal stiffeners*. The design of the longitudinal stiffeners should be such that, at all points in the region subjected to local moments, the following provisions are satisfied for the serviceability limit state, under the combined effects of in-plane forces and local bending:

(a) stresses in the stiffener due to local bending and in-plane forces should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

(b) $$\frac{\sigma_a - 2.5\tau_1 k_{s1}}{k_{\ell 1}\sigma_{ys}} + \frac{\sigma_{fo}}{\sigma_{ys}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}, \text{ and}$$

(c) $$\frac{\sigma_a + 2.5\tau_1 k_{s2}}{k_{\ell 2}\sigma_{ye}} + \frac{\sigma_{fz}}{\sigma_{ye}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{fo} is the stress due to local bending at the point on the stiffener furthest from the flange plate

σ_{fz} is as defined in 9.10.3.3.1

σ_a, τ_1, σ_{ys}, and σ_{ye} are as defined in 9.10.2.3

k_{s1} and k_{s2} are coefficients obtained from figure 18

$k_{\ell 1}$ and $k_{\ell 2}$ are values of the reduction factor determined from figure 18.

For zones of local sagging moment, i.e. causing local compressive stresses in the plate, the values of λ to be used in figure 18 are given in 9.10.2.3.

For zones of local hogging moment, i.e. causing local tensile stresses in the plate, the values of λ to be used in figure 18 are given by:

$$\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ys} - \gamma_m \gamma_{f3}\sigma_{fo}}{355}} \text{ for } k_{s1} \text{ and } k_{\ell 1}$$

$$\lambda = \frac{\ell}{r_{se}}\sqrt{\frac{\sigma_{ye} - \gamma_m \gamma_{f3}\sigma_{fz}}{355}} \text{ for } k_{s2} \text{ and } k_{\ell 2}$$

where

ℓ and r_{se} are defined in 9.10.2.3.

9.10.4 *Longitudinally stiffened flange not stiffened transversely.* The longitudinal stiffeners should satisfy the provisions of 9.10.2 and 9.10.3, but with ℓ taken as ℓ_e, given by:

$$\ell_e = 1.5\left(\frac{B}{t_f}\right)^{0.75} [I_{se}(n+1)]^{0.25}$$

where

B is the total width of the stiffened flange between main beam webs

I_{se} is the second moment of area of the effective section of each longitudinal stiffener

t_f is the flange plate thickness

n is the number of longitudinal stiffeners in width B.

9.10.5 *Curtailment of longitudinal flange stiffeners* Where longitudinal stiffeners are curtailed, the stiffener section should be extended beyond the theoretical cut-off point. The attachment of this extension is required to develop the load in the stiffener calculated at its theoretical cut-off point. This extension should be ignored for all other strength checks.

9.11 Webs in beams with longitudinal stiffeners in the cross-section

9.11.1 *General.* Webs should either be solid or have openings within the limits set out in 9.3.3.

A 'web panel' is defined as an area of web plate bounded on each transverse edge by a transverse stiffener or a diaphragm, and on each longitudinal edge either by a longitudinal stiffener or a flange of a beam.

An 'outer panel' is defined as a web panel adjacent to a flange of a beam.

9.11.2 *Strength.* The design of web panels should be such that, at all points on the panel, the yield criterion of 9.11.3 and the buckling criterion of 9.11.4 are both satisfied.

Longitudinal web stiffeners, if any, should satisfy the provisions of 9.11.5 and 9.11.6.

Intermediate transverse stiffeners, if any, should satisfy the provisions of 9.13.

9.11.3 *Yielding of web panels.* The following condition should be satisfied at all points on the panel:

$$\sigma_{1e}^2 + \sigma_2^2 - \sigma_{1e}\sigma_2 + 3\tau^2 \leqslant \left(\frac{\sigma_{yw}}{\gamma_m \gamma_{f3}}\right)^2$$

where

σ_1, σ_2, σ_b and τ are the coexistent components of stress shown in figure 19

(a) In the absence of transverse stresses in the panel ($\sigma_2 = 0$):

$$\sigma_{1e} = \sigma_1 + 0.77\sigma_b$$

where

σ_1 is the mean longitudinal stress on a cross section of the panel after any assumed redistribution in accordance with 9.5.4, considered positive if compressive

σ_b is the maximum longitudinal stress due to in-plane bending of the individual panel after any assumed redistribution in accordance with 9.5.4, considered positive if compressive

τ is the average shear stress due to the applied shear force and, in a closed section, due to the applied torsional moment.

NOTE. In this case it is only necessary to satisfy the criterion at all points along the longitudinal edges of the panel.

(b) In the presence of transverse stresses σ_2 in the panel:

$$\sigma_{1e} = \sigma_1 + k\sigma_b$$

where

σ_2 is the transverse stress, considered positive if compressive

σ_1, τ and σ_b are as defined in (a)

k $= 2y/b$ or 0.77, whichever is smaller

y is the perpendicular distance from the point being considered to the longitudinal centreline of the panel, to be taken always as positive

b is the width of panel (see figure 19).

9.10.5 Curtailment of longitudinal flange stiffeners

Add at end:

In assessment, where longitudinal flange stiffeners are curtailed prematurely beyond the theoretical cut off point, due account of this shall be taken in application of the foregoing clauses. The arrangement shall always be checked to ensure the extension beyond any assumed cut off point is sufficient to develop the assessment loads in the stiffener. The assessment procedure shall take due account of the actual end of the stiffener in deriving the capacity of the arrangement, by working back to the point where the stiffener can be assumed to be effective. The resulting extension shall be ignored for calculating stresses and other strength checks.

9.11.1 General

Add at end:

Webs not complying with the requirements of **9.3.3** (with respect to openings) or of **9.11.6** (with respect to partly extended/curtailed stiffeners) shall be assessed for the effects of stress concentrations and detailed local analyses shall be used in all such cases.

9.11.4 *Buckling of web panels*

9.11.4.1 *General.* In the case of webs subject to transverse stress as well as other in-plane stresses, the maximum intensity of transverse stress σ_2, acting over part of the length of a longitudinal edge, may conservatively be assumed to act over the whole length of the panel. Alternatively, the method set out in appendix D may be used.

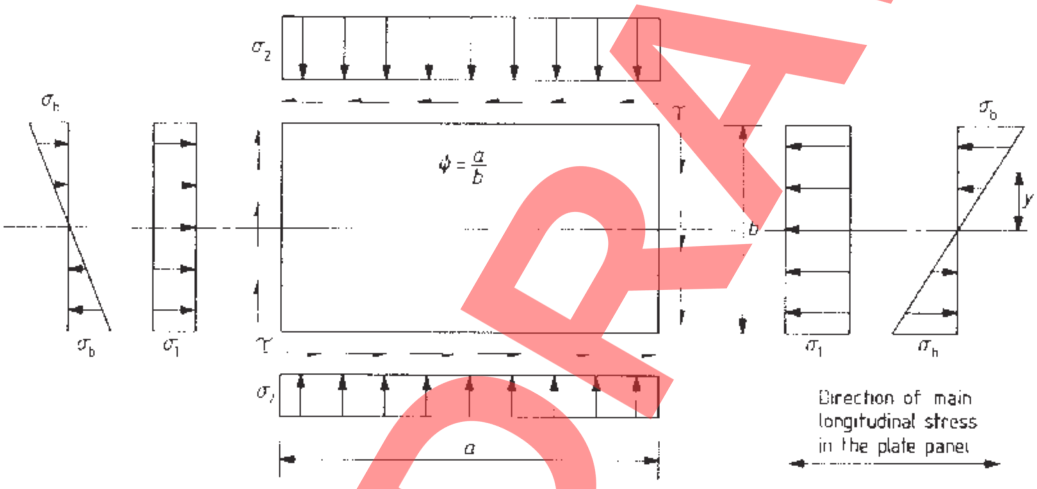
The transverse stress σ_2 in each panel of the web should be taken as that at the edge of the panel nearest to the load, calculated using the dispersion shown in figure 20.

9.11.4.2 *Restraint of web panels*

9.11.4.2.1 *General.* In order to calculate the buckling coefficients K_1, K_q, K_b and K_2 required in **9.11.4.3**, the effective in-plane boundary restraint of the panel should be determined in accordance with **9.11.4.2.2** or **9.11.4.2.3**, as appropriate.

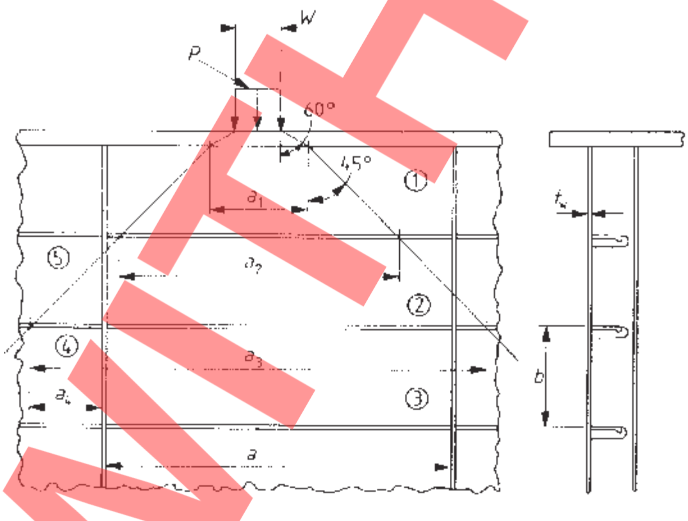
Any panel not meeting the provisions given in **9.11.4.2.2** and **9.11.4.2.3** should be treated as unrestrained.

9.11.4.2.2 *Restraint for derivation of K_1, K_q and K_b.* All interior web panels (i.e. not adjacent to a flange) may be treated as restrained.



NOTE. For transverse web stiffeners see **9.13.3.2**, for diaphragm plate panels see **9.17.6.5**.

Figure 19. Stresses on web panels



(a) Typical panels

Panel	Transverse stress σ_2 to be used for each panel over length a	w_2 for use in D.2
1	$\frac{P}{a_1 t_w}$	a_1
2	$\frac{P}{a_2 t_w}$	a_2
3	$\frac{P}{a_3 t_w}$	a
4	$\frac{P}{a_3 t_w}$	a_4
5	0	0

(b) Treatment of panels

NOTE. P is the component of the applied load in the plane of the web.

Figure 20. Dispersal of load through a longitudinally stiffened web

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Figure 20. Dispersal of load through a longitudinally stiffened web.

At the end of the 'NOTE', *add* 'and shall be compressive or tensile.'

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Any web panel adjacent to a flange may be treated as restrained provided that either:

(a) its slenderness ratio λ is less than 24; or

(b) (1) $m_{fw} > \left[\dfrac{\sigma_{yf}^2}{\sigma_{yf}^2 - \gamma_m^2\gamma_{f3}^2\sigma_f^2}\right]$

$\times [0.00025(\lambda - 24)]$

for $24 \leqslant \lambda \leqslant 84$, but λ taken as 84 for $\lambda > 84$ for this purpose only, and

(2) if $\lambda > 66 + 28/\phi^2$ then m_{fw} should be greater than the limiting value obtained from figure 21.

Where

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

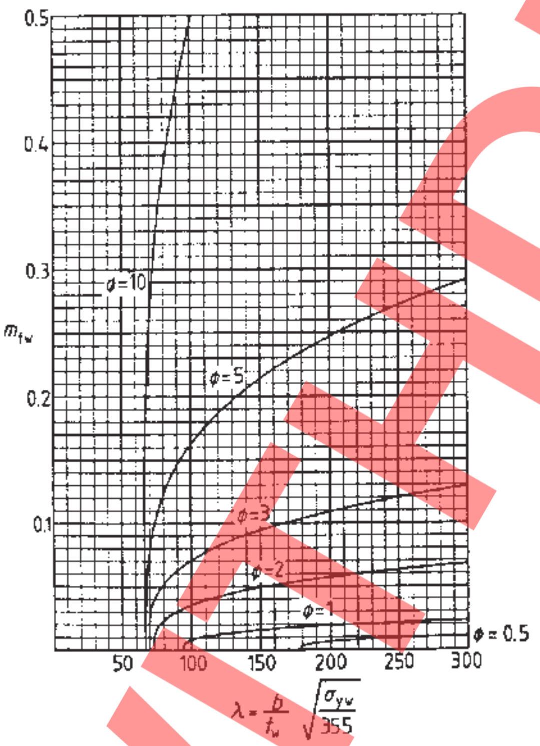
$$m_{fw} = \frac{\sigma_{yf} b_{fe} t_f^2}{2\sigma_{yw} b^2 t_w}$$

t_w, b_{fe}, t_f, σ_{yf} and σ_{yw} are as defined in **9.9.2.2**

ϕ is the aspect ratio a/b as shown on figure 19

a and b are the length and width of the panel respectively (see figure 19)

σ_f is the longitudinal stress in the flange plate, including any re-distribution of stresses.



NOTE 1. $\phi = a/b$ (see figure 19)

where

a is the dimension of the panel in the direction of main longitudinal stress

b is the panel dimension normal to a.

NOTE 2. For basis of curves, see G.10.

Figure 21. Minimum value of m_{fw} for outer panel restraint

For a web without longitudinal stiffeners both flanges should satisfy the criteria given in this clause for the web to be taken as restrained.

9.11.4.2.3 *Restraint for derivation of K_2.* When the plate extends beyond transverse stiffeners bounding a panel by at least a distance $a/2$, the panel may be assumed restrained.

9.11.4.3 *Buckling coefficients*

9.11.4.3.1 *General.* The coefficients K_1, K_q, K_b, and K_2 should be obtained from **9.11.4.3.2**, **9.11.4.3.3**, **9.11.4.3.4** and **9.11.4.3.5** respectively, with the panel assumed to be restrained or unrestrained in-plane as determined from **9.11.4.2**.

9.11.4.3.2 *Axial coefficient K_1.* K_1 should be taken as the greater of the values obtained as follows, either:

(a) from figure 22(a) using curve 1 or 2, as appropriate, with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 24, when:

$$K_1 = \left(\frac{t_w}{b}\right)^2 \frac{204\,500}{\sigma_{yw}}, \text{ or}$$

(b) from figure 22(a) curve 3 with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 4.33, when:

$$K_1 = \left(\frac{t_w}{a}\right)^2 \frac{6660}{\sigma_{yw}}$$

where

t_w is the web thickness

σ_{yw} is the nominal yield stress of the web material

a and b are as defined in figure 22(a).

9.11.4.3.3 *Shear coefficient K_q.* K_q should be taken from figure 22(b), unless:

$$\frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}} \text{ is less than } 35\sqrt{1 + (b/a)^2}$$

$$\text{when } K_q = \left(\frac{t_w}{b}\right)^2 \frac{435\,000[1 + (b/a)^2]}{\sigma_{yw}}$$

where

t_w and σ_{yw} are as defined in **9.11.4.3.2**

a and b are as defined in figure 22(b).

9.11.4.3.4 *Bending coefficient K_b.* K_b should be obtained from figure 22(c).

9.11.4.3.5 *Transverse coefficient K_2.* K_2 should be taken as the greater of the values obtained as follows:

(a) from figure 22(a) using curve 1 or 2 as appropriate with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 24, when:

$$K_2 = \left(\frac{t_w}{a}\right)^2 \frac{204\,500}{\sigma_{yw}},$$

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9.11.4.3.1 General.

Add at end:

Where the out-of-flatness of the plate panels exceed the tolerance in Part 6, allowance shall be made for this in deriving the buckling coefficients and their interaction. A method for this is included in the accompanying Advice Note.

Where the out-of-flatness of the plate panels is less than the tolerance in Part 6, allowance may be made for this in deriving the buckling coefficients and their interactions.

9.11.4.3.2 Axial coefficient K_1

Add at end:

If stress is tensile then $K_1 = 1.0$.

9.11.4.3.5 Transverse coefficient K_2

Add at end:

If stress is tensile then $K_2 = 1.0$.

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(b) from figure 22(a) curve 3 with:

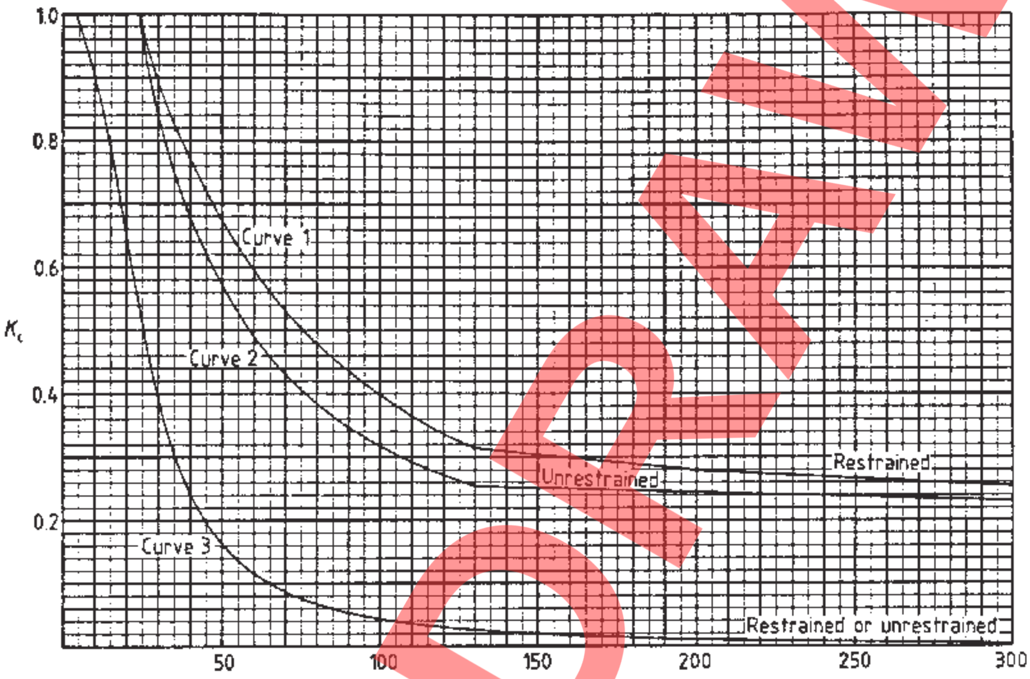
$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

unless λ is less than 4.33, when:

$$K_2 = \left(\frac{t_w}{b}\right)^2 \frac{6660}{\sigma_{yw}}$$

where

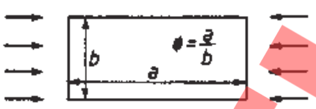
t_w and σ_{yw} are as defined in 9.11.4.3.2
a and b are as defined in figure 22(a).



λ (See notes 1 and 2 and 9.11.4.3.2 and 9.11.4.3.5)

(a) K_1 and K_2

NOTE 1. For stresses shown as follows:



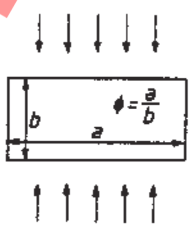
K_1 is the greater value of K_c obtained from curve 1 or 2, as appropriate, with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

or curve 3 with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

NOTE 2. For stresses shown as follows:



K_2 is the greater value of K_c obtained from curve 1 or 2, as appropriate, with:

$$\lambda = \frac{a}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

or curve 3 with:

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

NOTE 3. For basis of curves, see G.11(a).

Figure 22. Buckling coefficients K_1, K_2, K_q and K_b

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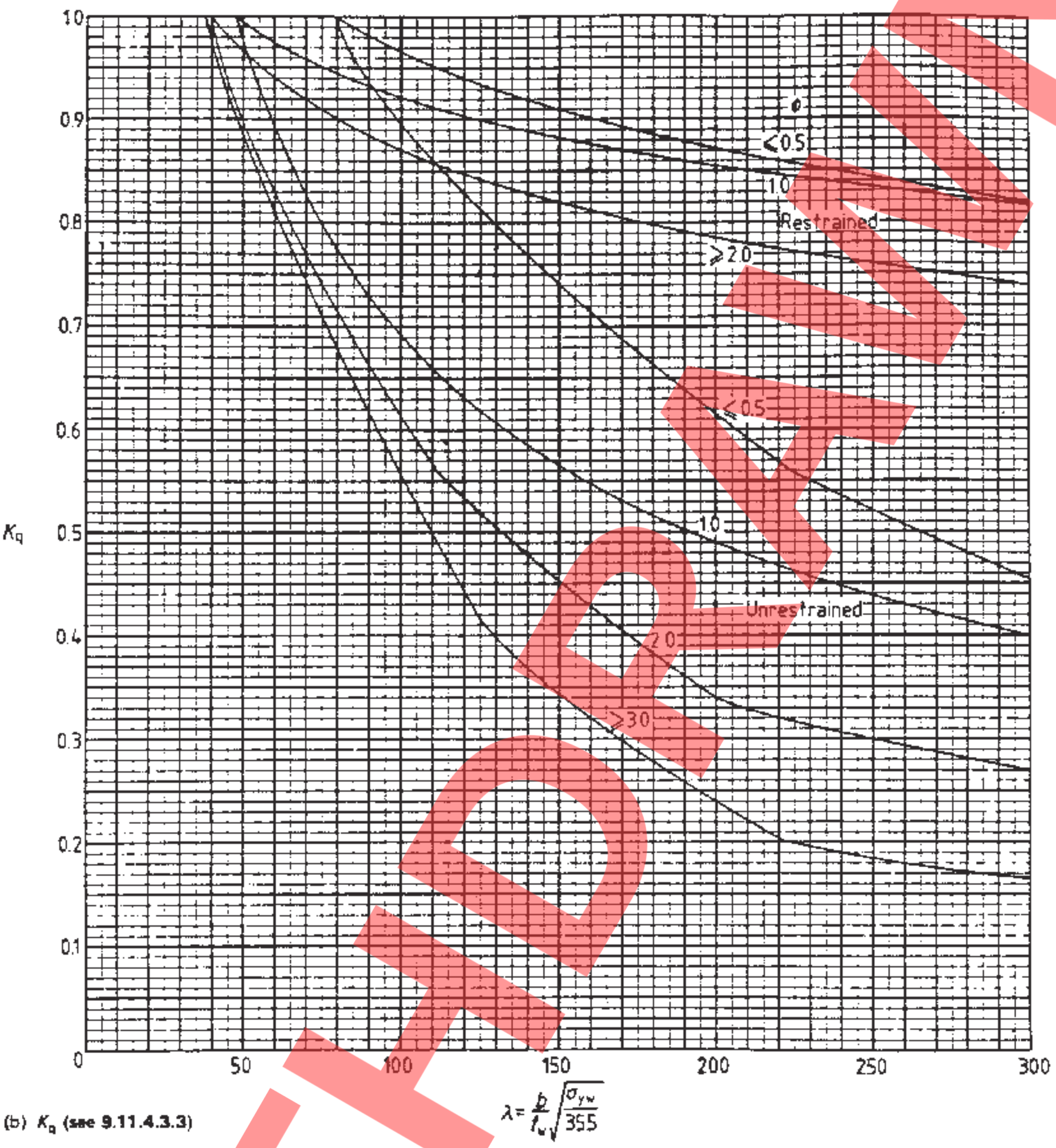
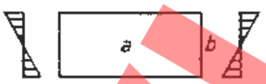


(b) K_q (see 9.11.4.3.3)

NOTE 1. For intermediate values of ϕ, K_q may be obtained by linear interpolation between two adjacent values of ϕ; for this purpose a curve may be extended beyond K_q = 1 by the expression in 9.11.4.3.3.

NOTE 2. $\phi = a/b$ shown as follows:



NOTE 3. For basis of curves, see G.11(b).

Figure 22 *(continued)*

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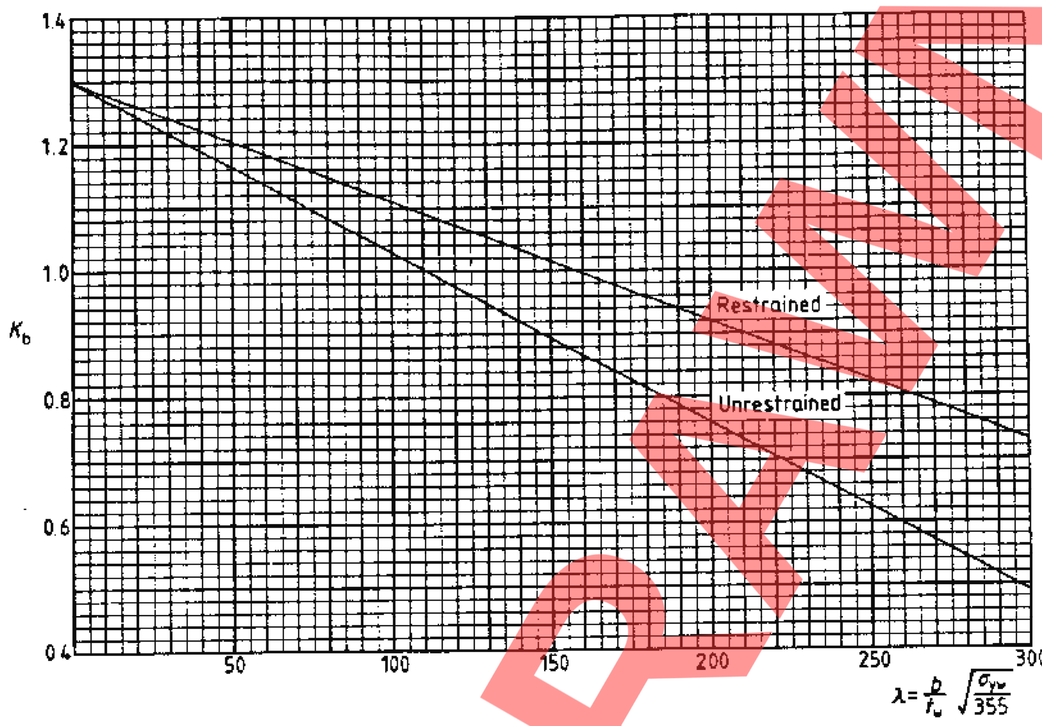

(c) K_b (see **9.11.4.3.4**)

NOTE. For basis of curves, see G.11 (c).

Figure 22. (*concluded*)

9.11.4.4 *Interaction buckling criterion.* The following condition should be satisfied for each panel:

$$m_c + m_b + 3m_q \leqslant 1$$

(a) In the absence of transverse stresses in the panel:

$$m_c = \frac{\sigma_1 \gamma_m \gamma_{f3}}{\sigma_{yw} K_1 (1 - \rho)}$$

$$m_b = \left(\frac{\sigma_b \gamma_m \gamma_{f3}}{\sigma_{yw} K_b (1 - \rho)}\right)^2$$

$$m_q = \left(\frac{\tau \gamma_m \gamma_{f3}}{\sigma_{yw} K_q}\right)^2$$

where

σ_1 is as defined in **9.11.3** but the algebraically higher of the two values on the opposite edges should be used

K_1, K_b and K_q are coefficients derived in accordance with **9.11.4.3.2** to **9.11.4.3.4**

σ_{yw} is the nominal yield stress of the web material

ρ = 0 for a restrained panel, irrespective of any redistribution of bending moment or axial force assumed under **9.5.4**, or
the proportion of stress assumed to be redistributed from the panel in accordance with **9.5.4** for an unrestrained panel

σ_b and τ are as defined in **9.11.3**, but the average values over the whole panel should be used.

(b) In the presence of transverse stresses in the panel:

m_c $= \sqrt{(m_1 + m_2)}$ for positive values of $(m_1 + m_2)$
$= -[\sqrt{-(m_1 + m_2)}]$ for negative values of $(m_1 + m_2)$

m_1 $= \left[\frac{\sigma_1 \gamma_m \gamma_{f3}}{\sigma_{yw} K_1 (1 - \rho)}\right]^2$ to be taken as negative if σ_1 is tensile

m_2 $= \left(\frac{\sigma_2 \gamma_m \gamma_{f3}}{\sigma_{yw} K_2}\right)^2$ to be taken as negative if σ_2 is tensile

where

K_1, σ_1, σ_{yw} and ρ are as defined above

σ_2 is as defined in **9.11.3**, but the algebraically higher of the two values on the opposite edges should be used

K_2 is a coefficient derived in accordance with **9.11.4.3.5**

m_b and m_q are as defined above.

9.11.5 ***Longitudinal web stiffeners***

9.11.5.1 *Effective section for longitudinal web stiffeners.* The effective stiffener section should comprise the stiffener with a width of web plate on each side of the stiffener connection centreline not exceeding:

$$16 t_w (1 - \rho) \text{ or } \frac{b}{2} (1 - \rho)$$

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where

- t_w is the thickness of the web plate
- ρ is the proportion of the longitudinal stress assumed to be redistributed from the relevant panel in accordance with **9.5.4**
- b is the width of the relevant plate panel adjacent to the stiffener.

9.11.5.2 *Strength of longitudinal web stiffeners.* The design of a longitudinal stiffener should be such that

$$\sigma_{se} \leqslant \frac{\sigma_{ls}}{\gamma_m \gamma_{f3}}$$

where

σ_{ls} is the limiting stiffener stress obtained from figure 23 using the value of:

$$\lambda = \frac{a}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

$$\sigma_{se} = k_s'\sigma_1 + \left(2.5\tau + \frac{a^2}{b^2}\sigma_2\right)\frac{bt_w k_s}{A_{se}}$$

- k_s' = 1.0 for continuous longitudinal stiffeners, or = $2.5k_s$ or discontinuous longitudinal stiffeners
- σ_1 is the longitudinal stress along the stiffener connection centreline (derived in accordance with **9.5.2**), taken as positive if compressive
- σ_2 is the coexistent transverse stress, if any, taken as positive if compressive
- τ is the average shear stress
- a is the clear distance between transverse web stiffeners
- b is the mean of the clear widths of the web plate panels above and below the line of attachment of the stiffener under consideration
- t_w is the web plate thickness
- A_{se} is the area of effective stiffener section
- k_s is obtained from figure 23 using the value of:

$$\lambda = \frac{a}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

- r_{se} is the radius of gyration of the effective stiffener section about an X-X axis parallel to the web (see figure 1).

9.11.6 *Curtailment of longitudinal web stiffeners*
Where longitudinal stiffeners are curtailed, the stiffener section should be extended beyond the theoretical cut-off point. The attachment of this extension is required to develop the load in the stiffener which is calculated at its theoretical cut-off point. This extension should be ignored for all other strength checks.

9.12 Restraints to compression flanges

9.12.1 *Elements providing effective intermediate discrete lateral restraints.* Where the effective length is determined in accordance with **9.6.2**, the compression flange should be provided with effective lateral restraints by means of bracing members. The bracing members should be so arranged and proportioned that at all transverse sections of the beam a restraining lateral shear force F can be resisted, having a value equal to:

$F = \dfrac{\Sigma P_f}{80}$ plus the direct shear arising from wind and other laterally applied forces, or

$F = \dfrac{\Sigma P_f}{40}$ when the effects of wind and other laterally applied forces are not included

where

ΣP_f is the sum of the greatest forces in two of the compression flanges of the beams connected by the bracing at the section under consideration.

9.12.2 *Intermediate U-frame restraints*

9.12.2.1 *General.* Intermediate U-frames may be used to provide lateral restraint as required by **9.6.5**; an effective lateral bracing or decking system should be provided at the level of the cross member of the U-frame along the entire span. When required, intermediate U-frames should be designed in accordance with **9.12.2.2** and **9.12.2.3**.

9.12.2.2 *Strength.* Where the effective length is determined in accordance with **9.6.5**, each intermediate U-frame and its connections should be designed to resist, in addition to the effects of wind and other applied forces, the effect of horizontal forces F_u, acting normal to the compression flange at the level of its centroid given by

$$F_u = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{\ell_e}{667\delta}\text{, but not greater than}$$

$$\left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{EI_c}{16.7\ell_u^2}$$

where

- ℓ_e, δ, I_c and ℓ_u are as derived in **9.6.5**
- σ_{fc} is the maximum compressive stress in the flange
- $\sigma_{ci} = \dfrac{\pi^2 ES}{\lambda_{LT}^2}$
- S = Z_{pe}/Z_{xc} for compact sections (see **9.3.7**), or = $D/2y_t$ for non-compact sections
- Z_{pe} is the plastic modulus of the section (see **9.9.1.2**)
- Z_{xc} is the elastic modulus of the section with respect to the extreme compression fibre
- D is the overall depth of the section
- y_t is the distance from the axis of zero stress to the extreme tension fibre of the section

NOTE 1. In composite construction with the concrete in tension the extreme fibre is the outer surface of the tensile reinforcing steel.

- λ_{LT} is as derived in **9.7**.

When there are several interconnecting beams, two such forces F_u should be applied, in the same or opposite directions, in such a way as to produce the most severe effect in the part being considered.

NOTE 2. When a concrete deck constitutes the whole or part of the cross member of the U-frame, in accordance with **9.6.5**, only those shear connectors on the main beam flange, which are within half the effective width of the concrete deck acting with or as the cross member, should be assumed to transmit the load effects at the corner of the U-frame.

9.11.5.2 Strength of longitudinal web stiffeners

Add at end:

Where in assessment of the adequacy of a longitudinal web stiffener allowance is to be made for initial departures from straightness, Δ_{sx}, measured in accordance with Part 6 over a gauge length taken as a, σ_{ls} and k_s shall be calculated from the equations in Appendix G12 with η taken as:-

$$\eta = 0.0083\,(\lambda - 15) + \left(\frac{\lambda - 15}{\lambda}\right)\left[\frac{(1.2\Delta_{sx} - 0.0016a)y}{r^2_{se}}\right]$$

but not less than zero.

in this expression

y is the distance from the neutral axis of the effective stiffener to the extreme fibre under consideration

Δ_{sx} is taken as positive when the bowing is in a direction away from the extreme fibre under consideration.

The strength shall be checked for both the extreme fibres in the outstand and in the associated web plate

9.11.6 Curtailment of longitudinal web stiffeners

Add at end:

In assessment, where longitudinal web stiffeners are curtailed prematurely beyond the theoretical cut off point, due account of this shall be taken in application of the foregoing clauses. The arrangement shall always be checked to ensure the extension beyond any assumed cut off point is sufficient to develop the assessment loads in the stiffener. The assessment procedure shall take due account of the actual end of the stiffener in deriving the capacity of the arrangement, by working back to the point where the stiffener can be assumed to be effective. The resulting extension shall be ignored for calculating stresses and other strength checks, but may be used in assessing stability in accordance with **9.11.7.**

Add New Clause 9.11.7:

9.11.7 Discontinuous longitudinal stiffeners not connected to transverse stiffeners

Where longitudinal stiffeners are discontinuous, i.e. they are fitted between transverse stiffeners and are not adequately connected to them, their area shall be ignored in calculating the stresses in the cross section. They may, however, be used in assessing the stability of the web under shear and/or compression provided they are terminated not more than four times the web thickness from the transverse stiffeners. In carrying out such stability checks the longitudinal stiffeners shall be assumed to carry a compressive stress equal to that in the web plate calculated in accordance with the above.

9.12.1 Elements providing effective intermediate discrete lateral restraints

Delete the whole clause and substitute the following-

Where the effective length is determined in accordance with **9.6.2**, the beam shall be provided with an effective restraint system which has sufficient strength and stiffness to inhibit lateral movement of the compression flange relative to the supports. This shall be provided by (a) lateral restraints or (b) torsional restraints.

where

- t_w is the thickness of the web plate
- ρ is the proportion of the longitudinal stress assumed to be redistributed from the relevant panel in accordance with **9.5.4**
- b is the width of the relevant plate panel adjacent to the stiffener.

9.11.5.2 *Strength of longitudinal web stiffeners*. The design of a longitudinal stiffener should be such that

$$\sigma_{se} \leqslant \frac{\sigma_{ls}}{\gamma_m \gamma_{f3}}$$

where

σ_{ls} is the limiting stiffener stress obtained from figure 23 using the value of:

$$\lambda = \frac{a}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$$

$$\sigma_{se} = k_s' \sigma_1 + \left(2.5\tau + \frac{a^2}{b^2} \sigma_2\right) \frac{b t_w k_s}{A_{se}}$$

- k_s' = 1.0 for continuous longitudinal stiffeners, or = $2.5 k_s$ or discontinuous longitudinal stiffeners
- σ_1 is the longitudinal stress along the stiffener connection centreline (derived in accordance with **9.5.2**), taken as positive if compressive
- σ_2 is the coexistent transverse stress, if any, taken as positive if compressive
- τ is the average shear stress
- a is the clear distance between transverse web stiffeners
- b is the mean of the clear widths of the web plate panels above and below the line of attachment of the stiffener under consideration
- t_w is the web plate thickness
- A_{se} is the area of effective stiffener section
- k_s is obtained from figure 23 using the value of:

$$\lambda = \frac{a}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$$

- r_{se} is the radius of gyration of the effective stiffener section about an X-X axis parallel to the web (see figure 1).

9.11.6 ***Curtailment of longitudinal web stiffeners*** Where longitudinal stiffeners are curtailed, the stiffener section should be extended beyond the theoretical cut-off point. The attachment of this extension is required to develop the load in the stiffener which is calculated at its theoretical cut-off point. This extension should be ignored for all other strength checks.

9.12 Restraints to compression flanges

9.12.1 ***Elements providing effective intermediate discrete lateral restraints.*** Where the effective length is determined in accordance with **9.6.2**, the compression flange should be provided with effective lateral restraints by means of bracing members. The bracing members should be so arranged and proportioned that at all transverse sections of the beam a restraining lateral shear force F can be resisted, having a value equal to:

$F = \dfrac{\Sigma P_f}{80}$ plus the direct shear arising from wind and other laterally applied forces, or

$F = \dfrac{\Sigma P_f}{40}$ when the effects of wind and other laterally applied forces are not included

where

ΣP_f is the sum of the greatest forces in two of the compression flanges of the beams connected by the bracing at the section under consideration.

9.12.2 ***Intermediate U-frame restraints***

9.12.2.1 *General.* Intermediate U-frames may be used to provide lateral restraint as required by **9.6.5**: an effective lateral bracing or decking system should be provided at the level of the cross member of the U-frame along the entire span. When required, intermediate U-frames should be designed in accordance with **9.12.2.2** and **9.12.2.3**.

9.12.2.2 *Strength.* Where the effective length is determined in accordance with **9.6.5**, each intermediate U-frame and its connections should be designed to resist, in addition to the effects of wind and other applied forces, the effect of horizontal forces F_u, acting normal to the compression flange at the level of its centroid given by

$$F_u = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right) \frac{\ell_e}{667\delta}, \text{ but not greater than}$$

$$\left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right) \frac{E I_c}{16.7 \ell_u^2}$$

where

- ℓ_e, δ, I_c and ℓ_u are as derived in **9.6.5**
- σ_{fc} is the maximum compressive stress in the flange
- $\sigma_{ci} = \dfrac{\pi^2 E S}{\lambda_{LT}^2}$
- S = Z_{pe}/Z_{xc} for compact sections (see **9.3.7**), or = $D/2y_t$ for non-compact sections
- Z_{pe} is the plastic modulus of the section (see **9.9.1.2**)
- Z_{xc} is the elastic modulus of the section with respect to the extreme compression fibre
- D is the overall depth of the section
- y_t is the distance from the axis of zero stress to the extreme tension fibre of the section

NOTE 1. In composite construction with the concrete in tension the extreme fibre is the outer surface of the tensile reinforcing steel.

- λ_{LT} is as derived in **9.7**.

When there are several interconnecting beams, two such forces F_u should be applied, in the same or opposite directions, in such a way as to produce the most severe effect in the part being considered.

NOTE 2. When a concrete deck constitutes the whole or part of the cross member of the U-frame, in accordance with **9.6.5**, only those shear connectors on the main beam flange, which are within half the effective width of the concrete deck acting with or as the cross member, should be assumed to transmit the load effects at the corner of the U-frame.

(a) Lateral restraints

Lateral restraints should be capable of resisting force F and should either be connected to an appropriate system of plan bracing capable of transferring the restraint forces to the supports or else connected to other beams or part of the structure capable of fulfilling this function.

It should be noted that where two or more parallel beams require lateral restraint at intervals, it is not adequate merely to connect the compression flanges together.

(b) Torsional restraints

Torsional restraints should be capable of resisting two equal and opposite forces F applied normal to the beam and in the planes of its two flanges.

Torsional restraint shall be provided by means of a suitable diaphragm between two beams or equivalent triangulated lateral bracing such that the beams transfer the restraint effects by equal and opposite vertical shears to the supports. Restraint shall alternatively be provided by external means.

The force F should be taken as:

$F = \Sigma P_f/80$ when the effects of wind and other laterally applied forces are included, or

$F = \Sigma P_f/40$ when the effects of wind and other laterally applied forces are not included.

where

ΣP_f is the sum of the greatest forces in two of the compression flanges of the beams connected by the bracing at the restraint under consideration.

Each intermediate restraint should be capable of resisting forces F appropriate to the restraint under consideration for which the assumed compression flange forces should be the maximum which occur at the restraint, or if greater, at a position midway between the restraints and the adjacent restraints (including any support restraint present which should be assumed to be an intermediate restraint for this purpose). The restraint system should be designed to resist the most severe effects arising from forces F at one restraint only within any length of flange in compression. Vertical component forces arising from application of forces F should be taken into consideration in design of the beams and restraints. Any eccentricity of the bracing members with respect to the line of action of forces F should be taken in account.'

9.12.2.2 Strength

Delete the definition for 'S' and substitute the following:

'$S=Z_{pe}/Z_{xc}$'

Delete the definition for 'y_t'
Delete the entire 'NOTE 1.'
Change 'NOTE 2' to 'NOTE 1'.

Add at end:

When a bridge with compression flanges restrained by U-frames is to be assessed using measured deviations of the flanges from straightness, the horizontal forces, F_u shall either be calculated by non-linear elastic analysis with the measured deviations from straightness allowed for in the initial geometry, or from the following:

$$F_u = \frac{1.2}{\delta} \Delta F \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}} \right)$$

but not greater than

$$\frac{48\, EI_c}{l^3_u} = D_F \left(\frac{s_{fc}}{s_{ci} - s_{fc}} \right)$$

where

Δf is the initial departure from straightness of the compression flange at the position of the U-frame measured in accordance with Table 5 in Part 6 at the mid-point of a gauge length G equal to the greater of 1.2

where

t_w is the thickness of the web plate

ρ is the proportion of the longitudinal stress assumed to be redistributed from the relevant panel in accordance with **9.5.4**

b is the width of the relevant plate panel adjacent to the stiffener.

9.11.5.2 *Strength of longitudinal web stiffeners*. The design of a longitudinal stiffener should be such that

$$\sigma_{se} \leqslant \frac{\sigma_{ls}}{\gamma_m \gamma_{f3}}$$

where

σ_{ls} is the limiting stiffener stress obtained from figure 23 using the value of:

$$\lambda = \frac{a}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$$

$$\sigma_{se} = k_s' \sigma_1 + \left(2.5\tau + \frac{a^2}{b^2}\sigma_2\right)\frac{b t_w k_s}{A_{se}}$$

k_s' $= 1.0$ for continuous longitudinal stiffeners, or $= 2.5 k_s$ or discontinuous longitudinal stiffeners

σ_1 is the longitudinal stress along the stiffener connection centreline (derived in accordance with **9.5.2**), taken as positive if compressive

σ_2 is the coexistent transverse stress, if any, taken as positive if compressive

τ is the average shear stress

a is the clear distance between transverse web stiffeners

b is the mean of the clear widths of the web plate panels above and below the line of attachment of the stiffener under consideration

t_w is the web plate thickness

A_{se} is the area of effective stiffener section

k_s is obtained from figure 23 using the value of:

$$\lambda = \frac{a}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$$

r_{se} is the radius of gyration of the effective stiffener section about an X-X axis parallel to the web (see figure 1).

9.11.6 *Curtailment of longitudinal web stiffeners*

Where longitudinal stiffeners are curtailed, the stiffener section should be extended beyond the theoretical cut-off point. The attachment of this extension is required to develop the load in the stiffener which is calculated at its theoretical cut-off point. This extension should be ignored for all other strength checks.

9.12 Restraints to compression flanges

9.12.1 *Elements providing effective intermediate discrete lateral restraints.* Where the effective length is determined in accordance with **9.6.2**, the compression flange should be provided with effective lateral restraints by means of bracing members. The bracing members should be so arranged and proportioned that at all transverse sections of the beam a restraining lateral shear force F can be resisted, having a value equal to:

$F = \dfrac{\Sigma P_f}{80}$ plus the direct shear arising from wind and other laterally applied forces, or

$F = \dfrac{\Sigma P_f}{40}$ when the effects of wind and other laterally applied forces are not included

where

ΣP_f is the sum of the greatest forces in two of the compression flanges of the beams connected by the bracing at the section under consideration.

9.12.2 *Intermediate U-frame restraints*

9.12.2.1 *General.* Intermediate U-frames may be used to provide lateral restraint as required by **9.6.5**: an effective lateral bracing or decking system should be provided at the level of the cross member of the U-frame along the entire span. When required, intermediate U-frames should be designed in accordance with **9.12.2.2** and **9.12.2.3**.

9.12.2.2 *Strength.* Where the effective length is determined in accordance with **9.6.5**, each intermediate U-frame and its connections should be designed to resist, in addition to the effects of wind and other applied forces, the effect of horizontal forces F_u, acting normal to the compression flange at the level of its centroid given by

$$F_u = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{\ell_e}{667\delta}, \text{ but not greater than}$$

$$\left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{EI_c}{16.7\ell_u^2}$$

where

ℓ_e, δ, I_c and ℓ_u are as derived in **9.6.5**

σ_{fc} is the maximum compressive stress in the flange

$$\sigma_{ci} = \frac{\pi^2 ES}{\lambda_{LT}^2}$$

S $= Z_{pe}/Z_{xc}$ for compact sections (see **9.3.7**), or $= D/2y_t$ for non-compact sections

Z_{pe} is the plastic modulus of the section (see **9.9.1.2**)

Z_{xc} is the elastic modulus of the section with respect to the extreme compression fibre

D is the overall depth of the section

y_t is the distance from the axis of zero stress to the extreme tension fibre of the section

NOTE 1. In composite construction with the concrete in tension the extreme fibre is the outer surface of the tensile reinforcing steel.

λ_{LT} is as derived in **9.7**.

When there are several interconnecting beams, two such forces F_u should be applied, in the same or opposite directions, in such a way as to produce the most severe effect in the part being considered.

NOTE 2. When a concrete deck constitutes the whole or part of the cross member of the U-frame, in accordance with **9.6.5**, only those shear connectors on the main beam flange, which are within half the effective width of the concrete deck acting with or as the cross member, should be assumed to transmit the load effects at the corner of the U-frame.

times the effective length l_e determined in accordance with **9.6.5** or **9.6.6** as appropriate or twice the spacing of the U-frames along the beam.

Where a beam with intermediate U-frames is not rigidly restrained at its supports (see **9.12.4.2**) F_u shall be factored by:

$$\left(\frac{1}{1 + \frac{n\delta_e}{6} \Sigma \frac{1}{\delta}} \right)$$

with l_e taken as l_e^{1} in accordance with **9.6.5**

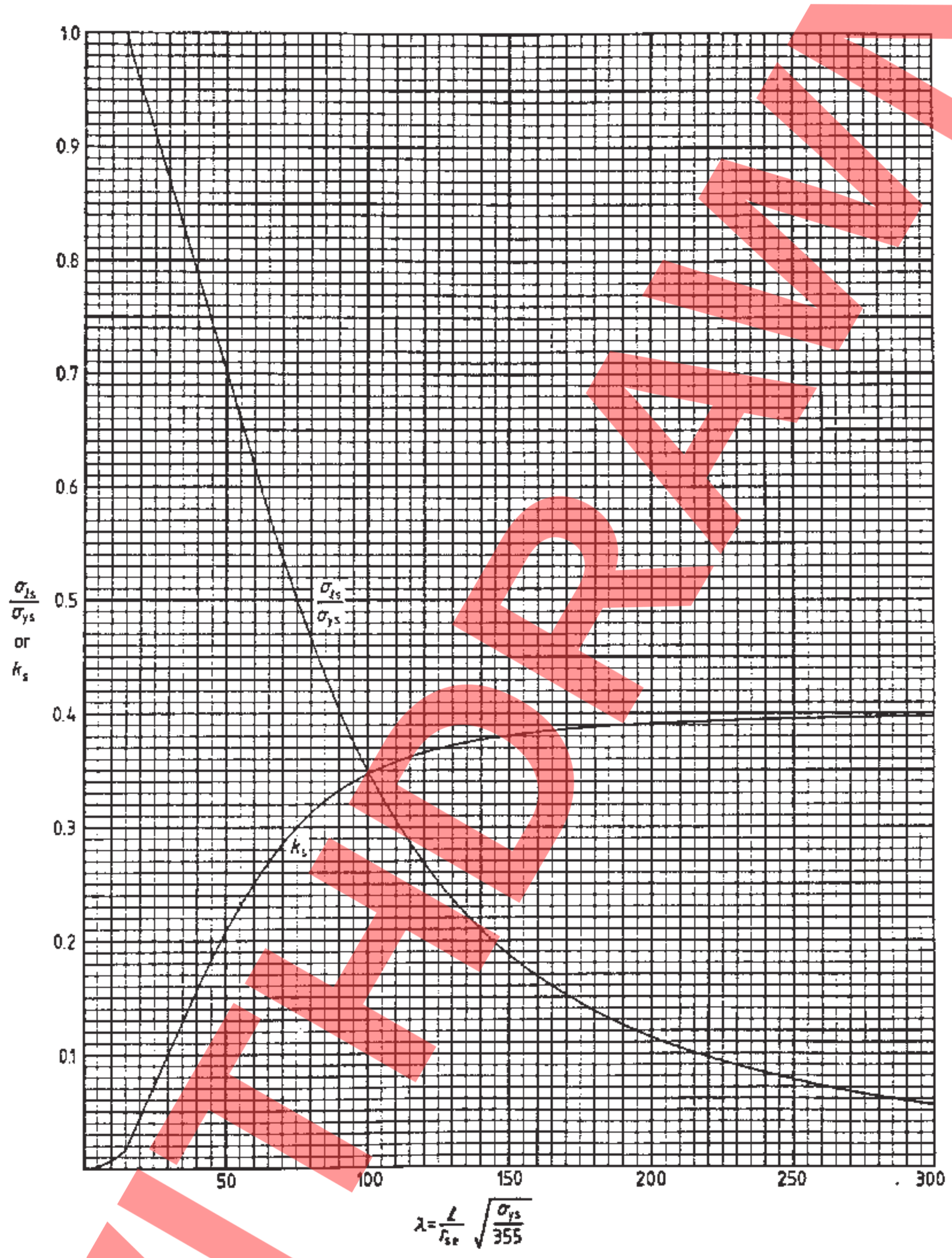
where

δ, l_u, I_c	are as defined in **9.6.5**
δ_e	is the value of δ for an end support
$\Sigma\, 1/\delta$	is the sum of the value of $1/\delta$ for each of the intermediate U-frames within a length l_e adjacent to the support
n	is the number of supports adjacent to the half-wavelength.

NOTE: In assessment of a beam for which l_e is greater than l_u, σ_{ci} shall be taken as $\pi^2E(r_{yc}/l_e)^2$

where

l_e	is determined in accordance with **9.6.5** or **9.6.6.2** as appropriate
r_{yc}	is the radius of gyration of the compression flange about its centroidal axis parallel to the web at the point of maximum bending moment.



NOTE 1. For the design of longitudinal web stiffeners (see 9.11.5.2) use $\ell = a$.

NOTE 2. For the design of transverse web stiffeners (see 9.13.3), bearing stiffeners (see 9.14.3 and 9.14.4), and diaphragm stiffeners (see 9.17.6.3 and 9.17.6.7) use $\ell = \ell_s$.

NOTE 3. For basis of curves, see G.12.

Figure 23. Parameters for the design of web stiffeners

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9.12.2.3 *U-frames with cross members subjected to vertical loading.* The following additional effects should be included for U-frames with cross members subjected to live loading.

(a) Additional forces F_c applied to the U-frame, in the same manner as F_u in **9.12.2.2**, resulting from the interaction between the bending of the cross members and vertical stiffeners, which may be taken as:

$$F_c = \frac{3EI_1\theta}{d_2^2}$$

where

I_1 and d_2 are as defined in **9.6.5**

θ is the rotation in radians of the cross member at its junction with the main beam under consideration, under the loading used in calculating σ_{fc} (see **9.12.2.2**).

NOTE. θ may be calculated neglecting any interaction between the cross member and vertical stiffeners of the U-frame. The average value of θ should be used for cross members within a non-uniformly loaded portion of the span.

(b) For all highway and railway bridges, except deck type highway bridges with the cross members formed entirely by a reinforced concrete deck up to grade 30 concrete (see Parts 4 and 7), the lateral flexure of a compression flange due to loading on a cross member should be considered. A method of determining the resulting transverse moment, and of combining it with other effects, may be obtained from appendix E.

9.12.3 *Continuous restraint provided by deck*

9.12.3.1 *Deck at compression flange level.* When a deck is continuously connected to the main beams at the level of the compression flange, the deck and its connections should be capable of withstanding a lateral restraining force equal to the greater of 2.5% of the force in the flange at the point of maximum bending moment in the absence of forces resulting from direct transverse loading, or 1.25% of the force in the flange at the point of maximum bending moment plus all forces arising from direct transverse loading. This lateral restraining force should be uniformly distributed along the span of the main beams.

9.12.3.2 *Deck not at compression flange level.* Where the effective length of the main beams is determined in accordance with **9.6.6**, the deck and webs and their connections should be designed to resist, in addition to the effects of wind and other applied forces, the effects due to the following.

(a) Horizontal forces f_u per unit length, acting normal to the compression flange at the level of its centroid given by:

$$f_u = \left(\frac{\sigma_{fc}}{\sigma_{ci} - \sigma_{fc}}\right)\frac{\ell_e}{667\delta}$$

where

ℓ_e and δ are as derived in **9.6.6**

σ_{fc} and σ_{ci} are defined in **9.12.2.2**.

When there are several interconnected beams, two such forces f_u should be applied, in the same or opposite direction, in such a way as to produce the most severe effect in the part being considered.

(b) Horizontal forces f_c per unit length, applied in the same manner as f_u in (a), resulting from the interaction of bending of the deck and the main beam webs which may be taken as:

$$f_c = \frac{Et_w^3\theta}{4d_2^2}$$

where

t_w and d_2 are as defined in **9.6.6**

θ is the rotation in radians of the deck at its junction with the web of the main beam under consideration, under the loading used in calculating σ_{fc} (see **9.12.2.2**).

NOTE. θ may be calculated neglecting any interaction between the deck and webs of the main beam. The average value of θ should be used within a non-uniformly loaded portion of the span.

(c) For all highway or railway bridges, except deck type highway bridges with the cross members formed entirely by reinforced concrete deck of up to grade 30 concrete (see Parts 4 and 7), the lateral flexure of a compression flange due to loading on a cross member should be considered. A method of determining the resulting transverse moment, and of combining it with other effects, may be obtained from appendix E.

9.12.4 *Restraint at supports*

9.12.4.1 *Restraining forces.* All beams, including cantilever beams, designed in accordance with **9.6**, should be restrained against rotation about their own axes at each support in accordance with the following provisions and **9.12.4.2**, as appropriate.

The restraining system should be capable of resisting, in addition to the co-existent effects of wind, frictional and other applied forces, two equal and opposite forces F_R applied normal to the beam and in the planes of its two flanges.

Where several beams are restrained by a common lateral member, two pairs of such forces should be taken, in the same or opposite directions, such as to produce the most severe effect in the part under consideration.

The value of each force F_R should be taken as the larger of F_1 or F_2, as given in (a) and (b) as follows.

(a) $$F_1 = \frac{\alpha_{LT}EDt_{f\,max}R_v\sigma_{fc}}{87.5W(\sigma_{ci} - \sigma_{fc})}$$

where

α_{LT} is dependent on the parameter ℓ_e/r_y and is given in figure 24

ℓ_e is determined in accordance with **9.6**

r_y is defined in **9.7.2**

D is the overall depth of the beam at the support

$t_{f\,max}$ is the maximum thickness of compression flange in the spans on either side of the support under consideration

9.12.2.3 U-frames with cross members subjected to vertical loading

In item (a), delete the expression for 'F_c' and the definitions for I_1, d_2 and θ'and substitute the following:

$$F_c = \frac{\theta d_2}{\delta + \dfrac{l^3_u}{15\,EI_c}}$$

where

d_2, δ, 1_u and I_c are as defined in **9.6.5**

θ is the rotation in radians of the cross member at its junction with the main beam under consideration assuming that the cross member is simply supported, under the loading used in calculating σ_{fc} (see **9.12.2.2**). The average value of θ should be used for cross members within a non-uniformly loaded portion of the span.

9.12.3.2 Deck not at compression flange level

Add at end of (a):-

When a bridge with compression flanges restrained by the webs is to be assessed using measured deviations of the flanges from straightness, the horizontal forces, per unit length f_u shall be calculated either by non-linear elastic analysis with the measured deviations allowed for in the initial geometry or from the above equation with 1.2 Δ_{Fmax} replacing l_e/667

where

Δ_{Fmax} is the maximum value of Δ_F obtained in accordance with Table 5 in Part 6 with a gauge length G equal to l_e traversed along the critical parts of the flange.

R_v is the vertical reaction from the bearing
W is the total vertical load on the spans adjacent to the support under consideration
σ_{fc} is the maximum compressive stress in the flange, averaged over the whole flange, either at the support under consideration or in the span either side of it
σ_{ci} is as defined in **9.12.2.2**.

(b) F_2 is determined from (1) to (4) as appropriate.

(1) When the compression flange is restrained laterally between points of support by an effective system of bracing, or by a continuous deck in the plane of the compression flange (see **9.12.3.1**), F_2 should be taken as 2.5% of the maximum force in the compression flange either at the support under consideration or in the span on either side of it.

(2) When the compression flange is not restrained laterally between points of support:

$$F_2 = \frac{0.004\sigma_{ci}M}{(\sigma_{ci} - \sigma_{fc})D}$$

where

σ_{fc} and σ_{ci} and D are as defined in (a)
M is the largest bending moment occurring either at the support under consideration or in the span on either side of it, whether sagging or hogging.

(3) When the compression flange is restrained laterally between points of support by a system of U-frames complying with **9.12.2**:

$$F_2 = 2(F_u + F_c)$$

where

F_u is as derived in **9.12.2.2**
F_c is as derived in **9.12.2.3**(a).

(4) When the beam is continuously restrained by the deck so that its effective length is determined in accordance with **9.6.6.2**:

$$F_2 = 2(f_u + f_c)(\ell_{e1} + \ell_{e2})$$

where

f_u and f_c are as derived in **9.12.3.2**
ℓ_{e1} and ℓ_{e2} are the effective lengths of the beam on either side of the support under consideration.

9.12.4.2 *Stiffness.* Where bearing stiffeners are used to provide the sole torsional restraint at the support sections of the beams, they should meet the stiffness criteria of (a) or (b) as follows, in addition to the criteria of **9.14** relevant to their function as bearing stiffeners.

(a) *Stiffener acting as cantilever.* A bearing stiffener acting as a cantilever from the bottom flange level should be such that:

$$I_X \geqslant 0.67\alpha_{LT}D^3 t_{f\,max}\frac{R_v}{W}$$

where

I_X is the second moment of area of the effective stiffener section about its X-X axis determined in accordance with 9.14.2 (see figure 27)
α_{LT}, D, $t_{f\,max}$, R_v and W are as defined in **9.12.4.1**.

(b) *Stiffener as part of U-frame.* A bearing stiffener acting as part of a U-frame should be such that:

$$\frac{1}{\alpha_{LT}E t_{f\,max}} \geqslant \delta$$

where

α_{LT} and $t_{f\,max}$ are as defined in **9.12.4.1**
δ is as derived in **9.6.5**.

9.13 Transverse web stiffeners other than at supports

9.13.1 ***General.*** Webs of plate girders, box girders and rolled beams should be provided with transverse stiffeners at all points where these are necessary for the adequacy of the web plate and the longitudinal stiffeners, if any.

A transverse web stiffener should be provided at all locations where a web connects with a cross beam and where a sloping flange changes direction.

Each end of the transverse stiffener should be stopped off or shaped to allow space for a root fillet or weld connecting the web to the flange where these occur, with a clearance not exceeding five times the thickness of the web, as shown in figure 1. The stiffener should extend over the whole remaining depth of the web and be fitted closely to the flange at each point of application of a concentrated load to the flange.

Where cut-outs are provided in transverse stiffeners to allow the passage of longitudinal stiffeners, at least one side of the opening in the transverse stiffener should be cleated to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat, or at least one-third of the perimeter of the cut-out should be connected to the longitudinal stiffener by welding.

A transverse web stiffener may form part of a cross beam, cross frame or U-frame.

9.13.2 ***Effective section of transverse web stiffeners.*** The effective stiffener section should comprise the stiffener with a portion of web plate of width b_{we} on each side of the stiffener connection centreline taken as the lesser of:

$$16t_w \text{ or } \frac{a}{2}$$

where

t_w is the thickness of the web plate
a is the spacing of transverse web stiffeners.

Where a stiffener outstand is stopped clear of the flange, the effective stiffener section between the end of the outstand and the flange should be taken as a portion of web plate only of width b_{we} on each side of the stiffener connection centreline for applying the provisions of **9.13.5.2**.

9.12.4.1 Restraining forces

Add at end of (b) (2):-

In assessment where the imperfections of the beam have been measured, the value of F_2 shall be calculated from:-

$$F_2 = \frac{4M}{D}\left[\frac{\sigma_{fc}/\sigma_{ci}}{2\left(1-\left(\frac{\sigma_{fc}}{\sigma_{ci}}\right)^2\right)}\frac{D}{L}\theta_o + \left(\frac{1}{1-\left(\frac{\sigma_{fc}}{\sigma_{ci}}\right)^2}\right)\frac{\Delta}{L}\right]$$

where

θ_o is the twist of the mid span of the beam relative to the mean of the departures from verticality at its supports, each twist being measured as Δ_D/D in accordance with 4(b) in Table 5 in BS5400 Part 6.

$$\Delta = \frac{\sigma_{fc}}{\sigma_{ci}}\left(\frac{\Delta_{DSmean}}{1.5}\right) + \Delta_{Fmean}$$

Δ_{DSmean} is the mean of the value of Δ_D at the two end supports and Δ_{Fmean} is the mean of the values of Δ_F for the top and bottom flanges respectively measured in accordance with 4(a) in Table 5 in BS5400 Part 6 over a gauge length L_G. Δ_{Fmean} shall be taken as positive and θ_o shall be taken as positive when it corresponds to a displacement of the compression flange relative to the tension flange in the same direction as Δ_{Fmean}.

Add at end of (b) (3):-

In assessment where the imperfections of the beam have been measured F_2 shall be taken as F_2^1 where:

$$F_2^1 = \frac{1}{2}\Sigma(F_u + F_c) + F_2$$

F_2 is derived from the assessment addition in **9.12.4.1 (b) (2)**

$\Sigma(F_u+F_c)$ is the sum of the Forces F_u and F_c for each of the U-frames within a length 1.2 l_e of the compression flange derived in accordance with **9.12.2.2 and 9.12.2.3(a)**

9.12.4.2 Stiffness

(a) Stiffener acting as cantilever

Add at end:

If the maximum compressive stress within a length l_e adjacent to a support derived in the assessment of a beam is less than σ_{lc} as given in **9.8.1** the second moment of area of a bearing stiffener acting as a cantilever, I_x, shall exceed the lesser of the foregoing limit or

$$\frac{9t_{fmax}D^3}{\left(\frac{l_e}{r_y}\right)^3 v^4\left[1-\left(\frac{\lambda_{LT}}{\lambda''_{LT}}\right)^2\right]}\cdot\frac{R_v}{W}$$

where

t_{fmax} is as defined in **9.12.4.1**

l_e, D, r_y and λ_{LT} are all as defined in **9.7.2**

v is as defined in the notes to Table 9

λ''_{LT} is the value of λ_{LT} to be used to obtain a value of σ_{lc} equal to the maximum compressive stress by use of Figure 10.

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R_v is the vertical reaction from the bearing

W is the total vertical load on the spans adjacent to the support under consideration

σ_{fc} is the maximum compressive stress in the flange, averaged over the whole flange, either at the support under consideration or in the span either side of it

σ_{ci} is as defined in **9.12.2.2**.

(b) F_2 is determined from (1) to (4) as appropriate.

(1) When the compression flange is restrained laterally between points of support by an effective system of bracing, or by a continuous deck in the plane of the compression flange (see **9.12.3.1**), F_2 should be taken as 2.5 % of the maximum force in the compression flange either at the support under consideration or in the span on either side of it.

(2) When the compression flange is not restrained laterally between points of support:

$$F_2 = \frac{0.004\sigma_{ci}M}{(\sigma_{ci} - \sigma_{fc})D}$$

where

σ_{fc} and σ_{ci} and D are as defined in (a)

M is the largest bending moment occurring either at the support under consideration or in the span on either side of it, whether sagging or hogging.

(3) When the compression flange is restrained laterally between points of support by a system of U-frames complying with **9.12.2**:

$$F_2 = 2(F_u + F_c)$$

where

F_u is as derived in **9.12.2.2**

F_c is as derived in **9.12.2.3**(a).

(4) When the beam is continuously restrained by the deck so that its effective length is determined in accordance with **9.6.6.2**:

$$F_2 = 2(f_u + f_c)(\ell_{e1} + \ell_{e2})$$

where

f_u and f_c are as derived in **9.12.3.2**

ℓ_{e1} and ℓ_{e2} are the effective lengths of the beam on either side of the support under consideration.

9.12.4.2 *Stiffness.* Where bearing stiffeners are used to provide the sole torsional restraint at the support sections of the beams, they should meet the stiffness criteria of (a) or (b) as follows, in addition to the criteria of **9.14** relevant to their function as bearing stiffeners.

(a) *Stiffener acting as cantilever.* A bearing stiffener acting as a cantilever from the bottom flange level should be such that:

$$I_X \geq 0.67\alpha_{LT}D^3 t_{f\,max}\frac{R_v}{W}$$

where

I_X is the second moment of area of the effective stiffener section about its X-X axis determined in accordance with 9.14.2 (see figure 27)

α_{LT}, D, $t_{f\,max}$, R_v and W are as defined in **9.12.4.1**.

(b) *Stiffener as part of U-frame.* A bearing stiffener acting as part of a U-frame should be such that:

$$\frac{1}{\alpha_{LT}Et_{f\,max}} \geq \delta$$

where

α_{LT} and $t_{f\,max}$ are as defined in **9.12.4.1**

δ is as derived in **9.6.5**.

9.13 Transverse web stiffeners other than at supports

9.13.1 ***General.*** Webs of plate girders, box girders and rolled beams should be provided with transverse stiffeners at all points where these are necessary for the adequacy of the web plate and the longitudinal stiffeners, if any.

A transverse web stiffener should be provided at all locations where a web connects with a cross beam and where a sloping flange changes direction.

Each end of the transverse stiffener should be stopped off or shaped to allow space for a root fillet or weld connecting the web to the flange where these occur, with a clearance not exceeding five times the thickness of the web, as shown in figure 1. The stiffener should extend over the whole remaining depth of the web and be fitted closely to the flange at each point of application of a concentrated load to the flange.

Where cut-outs are provided in transverse stiffeners to allow the passage of longitudinal stiffeners, at least one side of the opening in the transverse stiffener should be cleated to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat, or at least one-third of the perimeter of the cut-out should be connected to the longitudinal stiffener by welding.

A transverse web stiffener may form part of a cross beam, cross frame or U-frame.

9.13.2 ***Effective section of transverse web stiffeners.*** The effective stiffener section should comprise the stiffener with a portion of web plate of width b_{we} on each side of the stiffener connection centreline taken as the lesser of:

$$16t_w \text{ or } \frac{a}{2}$$

where

t_w is the thickness of the web plate

a is the spacing of transverse web stiffeners.

Where a stiffener outstand is stopped clear of the flange, the effective stiffener section between the end of the outstand and the flange should be taken as a portion of web plate only of width b_{we} on each side of the stiffener connection centreline for applying the provisions of **9.13.5.2**.

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(b) Stiffener as part of U-frame

Add at end:-

If the maximum compressive stress within a length le adjacent to a support derived in the assessment of a beam restrained by U-frames at its supports is less than σ_{lc} as given in 9.8.1 the value of α_{LT} may be taken as:-

$$\frac{27}{v^4\left(\frac{l_e}{r_y}\right)^3\left[1-\left(\frac{\lambda_{LT}}{\lambda''_{LT}}\right)^2\right]}$$

but not greater than the value obtained from Figure 24 factored by l/v^4, where v, l_e, r_y, λ_{LT} and λ''_{LT} are as defined in (a) above.

Add new item (c)

(c) Other support stiffeners

If the bearing stiffeners are not rigidly supported on their bearings or of a type other than a cantilever the requirements of **9.12.4.2(b)** shall be met, with δ calculated as for U-frames.

Add new clause 9.12.5:

9.12.5 Restraint to plastic hinges

All structures which are assessed using plastic methods of analysis in accordance with 7.4 shall provide lateral restraint close to all locations of plastic hinges which may occur under the various load cases. Such restraint shall be provided within a distance along the member from the theoretical plastic hinge locations not exceeding half the depth of the member.

9.13.1 General

Add at end:

In the assessment of existing arrangements where transverse stiffeners are not provided in accordance with the above, local effects shall be considered. Detailed analyses shall be carried out to cater for local effects resulting from the following:-

a) absence of web stiffeners where cross beams connect to the web;

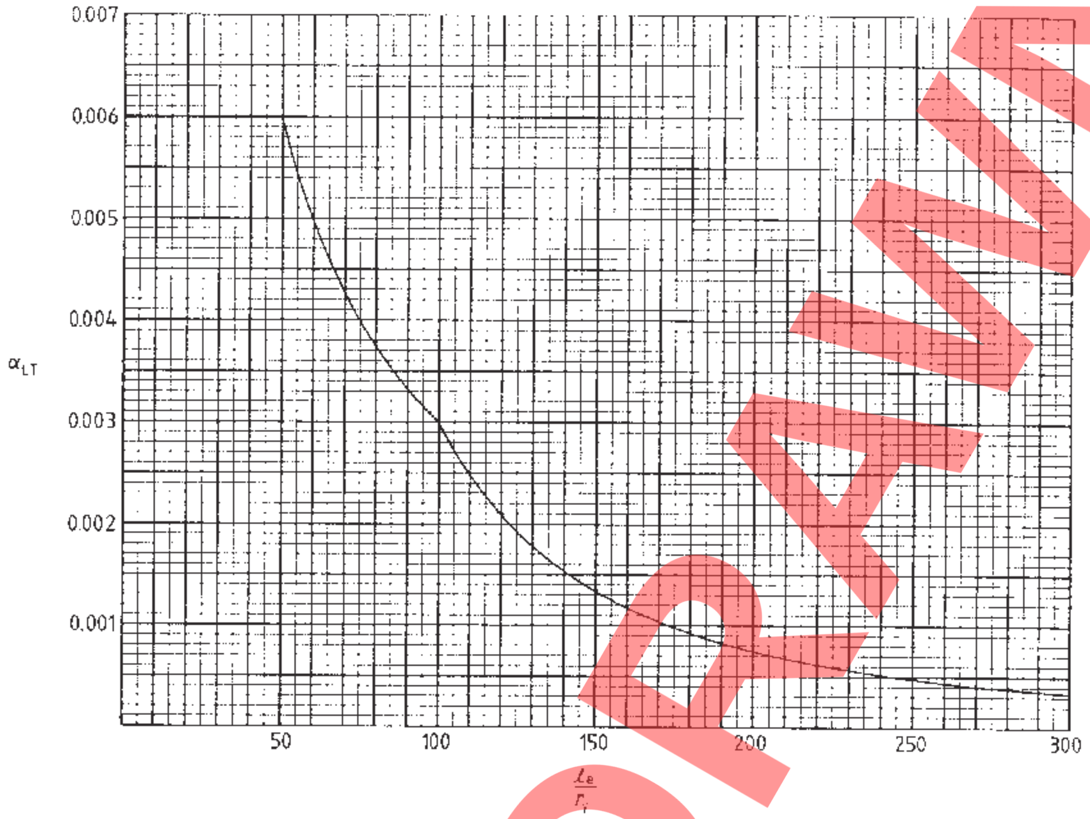
b) absence of web stiffeners where a sloping flange changes direction;

c) where the stiffener does not extend over the whole depth of the web or is not fitted closely to the flange at a point of application of a concentrated load to the flange;

d) where cut outs are not properly connected to the longitudinal stiffener

The assessment may utilise the relevant aspects of **9.14.6** and **9.15.6**.

As an alternative to the provisions of **9.13.3** to **9.13.6**, the adequacy of the transverse stiffeners or deep webs with longitudinal stiffeners may be assessed using the methods of **9.15.6**. In this case the longitudinal stiffeners shall also be checked at the serviceability limit state as for the flanges in accordance with **9.10.3.3**.



NOTE 1. ℓ_e is the effective length for lateral torsional buckling.

NOTE 2. r_y is the radius of gyration of the whole beam about its Y-Y axis (see figure 1).

NOTE 3. For basis of curve, see **G.13.**

Figure 24. Coefficient α_{LT} for torsional restraint at supports

9.13.3 *Loading on transverse web stiffeners*

9.13.3.1 *Effects to be considered.* A transverse stiffener should be designed to resist the following load effects, where these are present:

(a) axial force due to tension field action, in accordance with **9.13.3.2**;

(b) axial force representing the destabilizing influence of the web, determined in accordance with **9.13.3.3**;

(c) axial force due to transfer of load through a cross frame or cross beam;

(d) axial force due to load applied at flange level;

(e) axial force due to curvature of flange, in accordance with **9.13.3.4**;

(f) axial force due to change of slope of flange;

(g) bending moment about an axis in or parallel to the plane of the web, arising from eccentricity of axial force, or from flexure of a cross beam, cross frame, U-frame or deck.

NOTE. When only (b) is applicable the stiffener may be designed in accordance with **9.13.6**.

9.13.3.2 *Axial force due to tension field action.* Tension field action should be assumed to occur in any web panel when the average shear stress in the web panel, τ (see **9.5.1**), is greater than τ_o given by:

$$\tau_o = 3.6E\left[1 + \left(\frac{b}{a}\right)^2\right]\left(\frac{t_w}{b}\right)^2\sqrt{1 - \frac{\sigma_1}{2.9E}\left(\frac{b}{t_w}\right)^2}$$

$$\text{when } \sigma_1 < 2.9E\left(\frac{t_w}{b}\right)^2, \text{ or}$$

$$\tau_o = 0 \text{ when } \sigma_1 \geqslant 2.9E\left(\frac{t_w}{b}\right)^2$$

where

- a is the panel length, i.e. the dimension in the direction of the main longitudinal stress (see figure 19)
- b is the panel width, i.e. the dimension perpendicular to the direction of the main longitudinal stress (see figure 19)

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t_w is the thickness of the web

σ_1 is the average longitudinal stress in the web panel, to be taken as positive when compressive, calculated without redistribution of moment or axial force to the flanges.

Tension field action should be assumed to cause a compressive force F_{tw} in the adjacent transverse stiffener over its entire length, given by:

$F_{tw} = (\tau - \tau_o) t_w a$ or

$(\tau - \tau_o) t_w \ell_s$ whichever is the smaller,

where

ℓ_s is the length of the transverse stiffener, measured along the stiffener as the clear distance between the flanges of the beam.

When F_{tw} differs on the two sides of a transverse stiffener the average value may be taken.

When there are longitudinal stiffeners on the web, the average of the two smallest values of τ_o occurring in the web panels to one side of the transverse stiffener should be used in determining the value of F_{tw} for that side.

The force F_{tw} is to be taken as acting in the mid-plane of the web.

9.13.3.3 *Axial force representing the destabilizing influence of the web.* In order to resist buckling of the web plate the effective stiffener section should be assumed to carry, along its centroidal axis, a compressive force F_{wi} given by:

$$F_{wi} = \frac{\ell_s^2}{a_{max}} t_w k_s \sigma_R$$

where

ℓ_s is defined in **9.13.3.2**

a_{max} is the maximum spacing of transverse stiffeners which would ensure the adequacy of the web and longitudinal stiffeners. It may conservatively be taken as a (see figure 1)

t_w is the thickness of the web

k_s is obtained from figure 23 using the slenderness parameter λ determined from

$$\lambda = \frac{\ell_s}{r_{se}} \sqrt{\frac{\sigma_{ys}}{355}}$$

r_{se} is the radius of gyration of the effective stiffener section about the centroidal axis X-X (see figure 1)

$$\sigma_R = \tau_R + \left(1 + \frac{\Sigma A_s}{\ell_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right)$$

τ_R is equal to τ or τ_o, whichever is less. (see **9.13.3.2**)

ΣA_s is the sum of the cross-sectional areas of all the longitudinal stiffeners on the web not including any adjacent web plate

σ_1 is the average longitudinal stress in the web, taken as positive when compressive, calculated without any redistribution to the flanges (see figure 25)

σ_b is the maximum value of the stress in the web due to bending alone, calculated without any redistribution of moment to the flanges, as permitted by **9.5.4**, and always taken as positive (see figure 25).

For a longitudinally stiffened web, the force F_{wi} should be factored by η_s

where

$$\eta_s = \frac{1}{1 + 0.4 \dfrac{(\Sigma I_s) \ell_s^3}{I a^3}}$$

ΣI_s is the sum of the moments of inertia of the effective section of all the longitudinal web stiffeners in depth ℓ_s derived in accordance with 9.11.5.1

I is the moment of inertia of the effective section of the transverse stiffener.

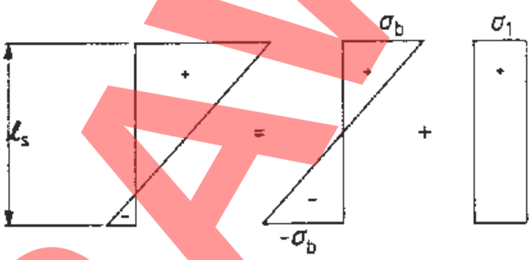


Figure 25. Longitudinal stress in webs with transverse stiffeners

9.13.3.4 *Axial force due to curvature.* The web plate included in the effective stiffener section should be considered to be subjected to an axial force F_f given by:

$$F_f = \frac{\sigma B_f t_f b_{we}}{R_f \cos \beta}$$

where

σ, B_f, t_f, R_f and β are as defined in **9.5.7**

b_{we} is as defined in **9.13.2**.

9.13.4 ***Distribution of axial loading.*** The force in a stiffener due to load applied at flange level, or due to curvature or change of slope of a stressed flange, or due to transfer of load through a cross frame, should be assumed to vary uniformly along the length of the stiffener from the value at the point of application to zero at the remote end of the stiffener.

The force in a stiffener due to tension field action or restraint of web buckling should be assumed as constant over the length of the stiffener.

9.13.5 ***Strength of transverse web stiffeners***

9.13.5.1 *Yielding of web plate.* The maximum equivalent stress σ_e, in the portion of web plate included in the effective stiffener section due to all the relevant forces and moments listed in **9.13.3.1**, except in item (b), together with coexistent shear and longitudinal stresses, should not exceed:

$$\frac{\sigma_{yw}}{\gamma_m \gamma_{f3}}$$

where

$\sigma_e = \sqrt{(\sigma_1 + K\sigma_b)^2 + \sigma_{es2}^2 - \sigma_{es2}(\sigma_1 + K\sigma_b) + 3\tau_R^2}$

σ_1 is as defined in 9.13.3.3

σ_b is the longitudinal bending stress in the web

σ_{es2} is the transverse stress in the web plate forming part of the effective stiffener section

9.13.3.3 Axial force representing the destabilising influence of the web

Add at end:

In the assessment of the adequacy of a transverse web stiffener, where allowance is to be made for initial departures from straightness, Δ_{sx}, measured in accordance with Part 6, k_s shall be calculated as described in **9.11.5.2**

τ_R is the lesser of τ and τ_o defined in **9.13.3.3**

k is as defined in **9.11.3**(b)

σ_{yw} is the nominal yield stress of the web plate.

9.13.5.2 *Yielding of stiffener.* The maximum stress in the stiffener at every point along its entire length between the flanges due to all the relevant forces and moments listed in **9.13.3.1**, except in item (b), should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the stiffener material.

In areas where cut-outs are provided an appropriate reduced effective section should be taken.

Where the end of a stiffener is fitted closely to the flange of a beam, whether to meet the provisions of **9.13.1** or otherwise, the bearing stress over the area in contact should not exceed:

$$\frac{1.33\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

In calculating this stress, the effective bearing area should be taken to consist of only those portions of the area of the stiffener and web plate that are:

(a) in contact with the flange;

(b) clear of the weld or root fillet at the web flange junction;

(c) within dispersal lines drawn at 60° from the line of application of any local load through the thickness of a flange plate (see figure 26).

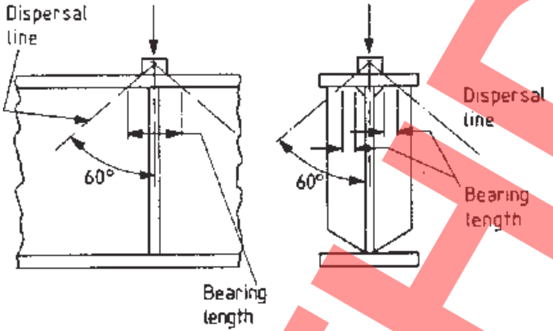


Figure 26. Dispersal of load through a transversely stiffened web

9.13.5.3 *Buckling of effective stiffener section.* The effective stiffener section defined in **9.13.2** should be such that:

$$\frac{P}{A_{se}\sigma_{ls}} + \frac{M_{xs}}{Z_x \sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P and M_{xs} are, respectively, the total maximum force on the effective stiffener section and the total maximum moment about the centroidal axis parallel to the web, due to all the relevant load effects given in **9.13.3.1** within the middle-third of the stiffener length

σ_{ls} is determined from figure 23 using the value of the slenderness parameter λ determined from

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is as defined in **9.13.3.2**

A_{se} and r_{se} are the area and radius of gyration respectively of effective stiffener section

Z_x is the lowest elastic section modulus of the effective stiffener section about the X-X axis over the middle-third of the length of the stiffener

σ_{ys} is the nominal yield stress of the stiffener material.

9.13.6 ***Transverse web stiffeners without applied loading.*** Where a transverse stiffener is subjected only to the load effect of **9.13.3.1**(b), it need only be designed to satisfy the following:

$$\sigma_R \leq \frac{A_{se} a_{max} \sigma_{ls}}{\eta_s k_s t_w l_s^2 \gamma_m \gamma_{f3}}$$

where

σ_R, a_{max}, k_s, and t_w are as defined in **9.13.3.3**

A_{se} and σ_{ls} are as defined in **9.13.5.3**

ℓ_s is as defined in **9.13.3.2**.

η_s = 1 for webs without longitudinal stiffeners and is defined in **9.13.3.3** for longitudinally stiffened webs

9.14 Load bearing support stiffeners

9.14.1 ***General.*** Webs of plate girders and rolled beams shall be provided with a system of load bearing stiffeners at each support position. Hereafter these will be referred to as bearing stiffeners.

The section of a bearing stiffener should be symmetrical about the mid-plane of the web. When this condition is not met, the effect of the resulting eccentricity should be taken into account.

The ends of a bearing stiffener should be closely fitted or adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web (see figure 1).

Where cut-outs are provided in bearing stiffeners, to allow the passage of longitudinal stiffeners, at least one side of the opening in the bearing stiffener should be cleated to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat, or at least one-third of the perimeter of the cut-out should be connected to the longitudinal stiffener by welding.

A bearing stiffener may be used to provide torsional restraint to a beam at its support, as required by **9.12.4.2**; it may also form part of a cross beam, cross frame or U-frame.

9.14.2 ***Effective section for bearing stiffeners***

9.14.2.1 *Single leg stiffener.* The effective stiffener section should be taken to comprise the stiffener with a portion of web plate on each side having a width not exceeding the following (see figure 27(b)):

(a) half the spacing of transverse stiffeners;

(b) the distance to the transverse edge of the web plate at the end of a beam;

(c) $16t_w$, where t_w is the thickness of the web plate.

9.13.5.3 Buckling of effective stiffener section.

Add at end:

Where in assessment of the adequacy of a transverse web stiffener allowance is to be made for initial departures from straightness, Δ_{sx}, measured in accordance with Part 6, σ_{ls} shall be calculated as described in **9.11.5.2**.

9.14.1 General

Add at end:

In the assessment of existing arrangements where bearing stiffeners are not provided in accordance with the above, local effects shall be considered. Detailed analyses shall be carried out to cater for local effects resulting from the following as appropriate:

a) where the stiffener does not extend over the whole depth of the web or is not fitted closely to the flange;

b) where cut outs are not properly connected to the longitudinal stiffener.

Where bearing stiffeners are not provided, in accordance with the above, the adequacy of the web under transverse loading shall be checked in accordance with **9.14.6.**

9.14.2.2 *Multileg stiffeners.* The effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs, and a portion of web plate not exceeding the widths given in **9.14.2.1** on the outer sides of the outer legs.

If the spacing of the legs of adjacent stiffeners is greater than $25t_w$, the legs should be treated as independent stiffeners according to **9.14.2.1**.

9.14.3 *Loading on bearing stiffeners*

9.14.3.1 *Effects to be considered.* A stiffener should be designed to resist the following load effects, where these are present.

(a) An axial force equal to the algebraic sum of the reaction from the support bearing and the vertical components of the forces in the adjacent bottom flange or flanges, due to slope or curvature.
(b) An axial force representing the destabilizing influence of the web, determined in accordance with **9.14.3.2**.
(c) Axial force due to transfer of load through a cross frame or cross beam.
(d) Bending moments about the X-X or Y-Y axes (as shown in figure 27) arising from eccentricity of axial force about the relevant axes, determined in accordance with **9.14.3.3**, or from flexure of a cross beam, cross frame, U-beam or deck.
(e) Bending moment about the X-X axis due to transverse forces F_R determined and applied in accordance with **9.12.4.1**. This moment should be assumed to be applied to any bearing stiffener which, as a cantilever or part of a U-frame, is the sole means of providing the restraint required by **9.12.4**.
(f) Bending moment about the Y-Y axis due to shear in the web adjacent to an end bearing stiffener (see **9.14.3.4**) in the case of beams with intermediate transverse web stiffeners.

9.14.3.2 *Axial force representing the destabilizing influence of the web.* In order to resist web buckling, the effective section of a bearing stiffener at an internal support of a continuous beam should be assumed to carry an additional compressive force F_{wi} along its centroidal axis equal to:

$$F_{wi} = \frac{l_s^2}{a_{max}} t_w k_s \sigma_R$$

where

$$\sigma_R = \left(1 + \frac{\Sigma A_s}{l_s t_w}\right)\left(\sigma_1 + \frac{\sigma_b}{6}\right)$$

t_w, a_{max}, k_s, ΣA_s, σ_1 and σ_b are as defined in **9.13.3.3**
l_s is as defined in **9.13.3.2**.

For a longitudinally stiffened web, F_{wi} may be factored by η_s, where η_s is as given in **9.13.3.3**.

9.14.3.3 *Eccentricity.* Load effects due to eccentricities arising from the following causes should be taken into account, when relevant:

(a) movements of the beam relative to the bearing due to changes in temperature;
(b) changes in the point or line of contact at the spherical or cylindrical surface of a bearing due to slope of the beam when deflected by load;
(c) uneven seating which may occur on a flat bearing surface;
(d) inaccuracy which may occur in positioning of the beam relative to the bearing.

The following values of eccentricity may be assumed to satisfy the provisions of (c) and (d):

(1) half the width of the flat bearing surface plus 10 mm for flat-topped rocker bearing in contact with flat bearing surface; or
(2) 3 mm for radiused upper bearing resting on flat or radiused lower part; or
(3) 10 mm for flat upper bearing resting on radiused lower part.

9.14.3.4 *End supports.* When the shear stress τ in the web of a beam at a section adjacent to an end support stiffener is greater than τ_o, the support stiffener, together with any additional stiffening which may be provided at the end of the beam (see figure 1), should be designed to resist an additional bending moment M_y about the axis through the centroid of the end section perpendicular to the web, i.e. the Y-Y axis shown in figure 1, given by:

$$M_y = \frac{8(\tau - \tau_o) t_w l_s^2}{\theta_d}$$

where

$$\tau_o = 3.6E\left[1 + \left(\frac{b}{a}\right)^2\right]\left(\frac{t_w}{b}\right)^2$$

t_w, a and b are as defined in **9.13.3.2**
l_s is the length of the bearing stiffener, measured along the stiffener as the clear distance between the flanges of the beam
θ_d is the slope in degrees of the diagonal line joining the top of the bearing stiffener to the bottom of the adjacent transverse stiffener, as shown in figure 27.

If there are longitudinal stiffeners on the web, the average of the two smallest values of τ_o in the web panels in the section should be used.

NOTE. For beams without intermediate transverse web stiffeners, M_y may be taken as zero.

9.14.4 *Strength of bearing stiffeners*

9.14.4.1 *Yielding of the web plate.* The maximum equivalent stress σ_e, determined from **9.13.5.1**, in the portion of web plate included in the effective stiffener section due to all the relevant forces and moments listed in **9.14.3.1**, except item (b), together with coexistent longitudinal stresses in the web plate, should not exceed:

$$\frac{\sigma_{yw}}{\gamma_m \gamma_{f3}}$$

where

σ_{yw} is the nominal yield stress of the web plate.

9.14.4.2 *Yielding of stiffener.* The maximum stress in the stiffener itself, at every point along its length, due to all relevant load effects listed in **9.14.3.1**, except item (b),

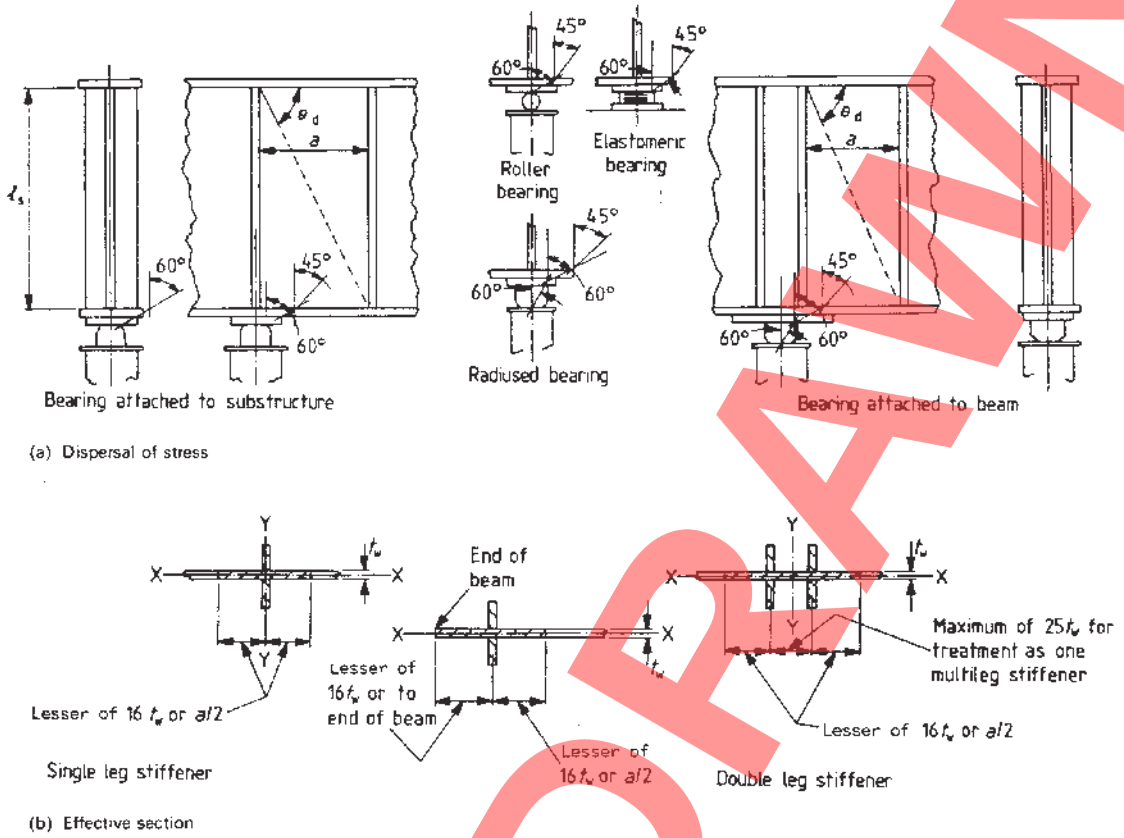
9.14.3.3 Eccentricity.

Add at end:

In assessing the adequacy of an existing stiffener, where any error in positioning and any unevenness of seating on a flat bearing is measured, the following values of eccentricity shall be taken into account in respect of (c) and (d) above.

(1) half the width of the flat bearing surface plus the measured error in positioning for flat topped rocker bearing in contact with flat bearing surface: or

(2) the measured error in positioning for radiused upper bearing resting on flat or radiused lower part or for flat upper bearing resting on radiused lower part.



NOTE. Dispersal lines lie continuously in metal from their origin. Where a 60° dispersal angle is shown, this is the maximum that may be assumed.

Figure 27. Bearing stiffeners

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calculated on the basis of the effective section as determined under **9.14.2**, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the stiffener material.

In areas where cut-outs are provided an appropriate reduced effective section should be taken.

The bearing stress over the area of the end of a bearing stiffener in contact with a flange should not exceed:

$$\frac{1.33\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

In calculating this stress, the effective bearing area should consist of only those portions of the area of the stiffener and the web that are:

(a) in contact with the flange; and
(b) clear of the weld or root fillet at the web flange junction; and
(c) within the uninterrupted dispersal lines drawn at a maximum of 60° to the line of application of the bearing reaction from the bearing contact area (see figures 26 and 27).

9.14.4.3 *Buckling of effective stiffener section.* The effective stiffener section, as defined in **9.14.2**, should be such that:

$$\frac{P}{A_{se}\sigma_{ls}} + \frac{M_{xs}}{Z_x\sigma_y} + \frac{M_{ys}}{Z_y\sigma_y} + \frac{M_y}{Z_{cy}\sigma_y} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P is the maximum force on the stiffener within the middle third of its length

M_{xs} and M_{ys} are the maximum bending moments on the stiffener about the X-X and Y-Y axes, respectively, (see figure 27) due to all the relevant load effects listed in **9.14.3.1**, except item (f), within the middle-third of the length of the stiffener

A_{se} is the area of the effective stiffener section

σ_{ls} is as defined in 9.13.5.3

Z_x and Z_y are the appropriate section moduli of the effective stiffener section

M_y is as defined in **9.14.3.4**

Z_{cy} is the section modulus of the end section, as described in **9.14.3.4**

σ_y is the nominal yield stress of the web plate or the stiffener, as appropriate.

9.14.5 ***Stiffness of bearing stiffeners.*** A bearing stiffener, acting as a cantilever from the bottom flange level or as part of a U-frame, which is the sole means of providing the restraint required by **9.12.4.1**, should be in accordance with **9.12.4.2**(a) or (b), as appropriate.

9.15 Cross beams and other transverse members in stiffened flanges

9.15.1 *General*

9.15.1.1 *Loading.* Where transverse members are provided on flanges or decks they should be designed for the following:

(a) to transfer local loading from the flange or deck to the web of main beams;
(b) to distribute loading transversely to the main beams;
(c) to withstand forces arising from a longitudinal change of slope of a flange.

NOTE. In box girders, a flange transverse member may also be part of an internal cross frame or diaphragm.

9.15.1.2 *Compression flanges.* In addition to the provisions of **9.15.1.1**, compression flange transverse members should have sufficient stiffness and strength to prevent overall buckling of the flange. A member designed in accordance with **9.15.3** and **9.15.5** may be deemed to satisfy this provision.

9.15.1.3 *Transverse member support.* All flange transverse members required by **9.15.1.1** or **9.15.1.2** should be supported by transverse web stiffeners at main beam webs, unless a special investigation is undertaken to show such stiffening to be unnecessary. Transverse members required for a flange projecting from an outer main beam should be continuous with the transverse member between main beam webs.

9.15.2 *Effective section for transverse members*

9.15.2.1 *Effective section for stiffness.* In determining the stiffness of a transverse member for global analysis and for overall buckling of a compression flange, an effective width of attached flange should be assumed to act with the member, on each side of the web of the cross member where available, and should be taken as the lesser of:

(a) half the spacing of transverse members; or
(b) either:
(1) one-eighth of the distance between main beam webs, for a portion between such webs; or
(2) one-sixth of the cantilever length, for a cantilever portion.

This effective width should be taken as constant over each relevant portion of the transverse member.

NOTE. When the flange consists of composite reinforced concrete, the reinforcement within the effective width, but not the concrete, may be taken into account in calculating the overall buckling strength of the compression flange, but both reinforcement and concrete may be taken into account for global analysis.

9.15.2.2 *Effective section for strength and stress calculation.* For calculating stresses in, or the strength of, a transverse member, the effective section should be taken as for stiffness (see **9.15.2.1**), but with the following modifications:

(a) when, in a non-composite transverse member, the attached flange is in compression parallel to its axis (e.g. in the sagging moment zone of a top flange transverse member) the effective width on each side of the web of the cross member should not exceed one-quarter of the transverse member spacing;
(b) between main beam webs, in the portion subjected to hogging bending moments, the effective width on each side of the web of the cross member should not locally exceed one-fourteenth of the distance between main beam webs;
(c) when the effective widths on two sides of a main beam web are unequal, an average value should be taken for the section at the main beam web;

9.14.4.3 Buckling of effective stiffener section

In the defnition for P, delete 'within the middle third of its length'.

Add new clause 9.14.6:

9.14.6 Unstiffened web at supports

9.14.6.1 Strength of web

The strength of an unstiffened web shall be taken as the limiting value of patch load P, as determined in accordance with Appendix D.

9.14.6.2 Buckling resistance of web

The buckling resistance P_D of an unstiffened web over a bearing shall be taken as:

$$P_D = \frac{\sigma_c \, b_{eff} \, t_w}{\gamma_m \, \gamma_{f3}}$$

where

σ_c is the ultimate compressive stress about an axis along the centre line of the web obtained from σ_c/σ_y in accordance with Curve C of Figure 37; **NOTE**: In using Figure 37, l_e shall be determined taking account of the lateral and rotational restraint of the flange.

b_{eff} is the effective breadth of web obtained from

$$b_{eff} = \sqrt{\left(d^2 + s^2\right)}$$

but not greater than the width available (see Figure 27).

d is the overall depth of the beam;

s is the bearing length.

γ_m is taken as 1.05 for ultimate limit state.

9.15.1.2 Compression flanges

Add at end:

Compression flange transverse members which do not comply with the requirements of **9.15.3** and **9.15.5** shall be assessed in accordance with **9.15.6**.

(d) in the case of composite flanges, the area of concrete in tension should be ignored;
(e) the effective thickness of the cross member web should be determined in accordance with 9.4.2.5.

9.15.2.3 *Compact section.* Irrespective of the effective width of flange, transverse members attached to a continuous deck may be taken as compact if their webs and flange outstands meet the appropriate provisions of 9.3.7.

9.15.3 *Stiffness of transverse members in compression flanges*

9.15.3.1 *Transverse member segment types.* In order to satisfy the stiffness provisions of 9.15.1.2 for a transverse member supporting a compression flange, the entire length of the effective member should be divided, for analysis, into the following segments (see figure 28):

(a) type I segments between interior webs of main beams;
(b) type II segments comprising any cantilever and the adjacent length to the first interior beam web.

NOTE For cross sections with only two main beam webs, one segment of type II will encompass the full length of transverse member, including cantilevers (if any).

9.15.3.2 *Stiffness of transverse members.* Transverse members should be designed such that:

$$I_{be} \geq \frac{9\sigma_f^2 a B^4 A_f^2}{16 K E^2 I_f}$$

where

I_{be} is the average second moment of area of the effective transverse member (see 9.15.2.1) between main beam webs

σ_f is the longitudinal compressive stress in the flange of the main beam, averaged across the width of the segment between main beam webs

B is the spacing of the main beam webs at the level of the transverse member

a is the transverse member spacing (or mean of adjacent spacings)

A_f is the area per unit width of the cross section of the flange of the main beam

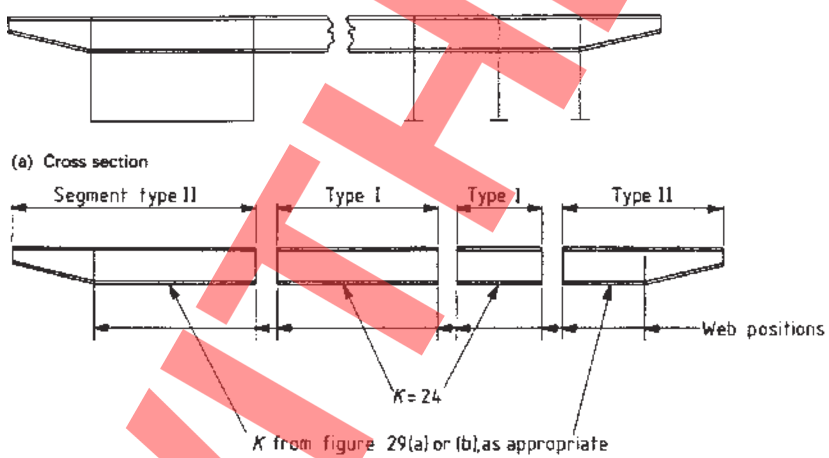
I_f is the second moment of area per unit width of the flange of the main beam about the centroidal axis of the flange

NOTE 1. A_f and I_f should include any longitudinal stiffeners, and, in the case of a concrete or composite flange, concrete areas divided by the modular ratio. Where there are longitudinal stiffeners, A_f and I_f should be calculated on the basis of the effective section of the member derived in accordance with 9.4.2.

K = 24 for type I segments (see 9.15.3.1) with open longitudinal stiffeners (but see note 2 for closed stiffeners), or,
for type II segments (see 9.15.3.1), K should be obtained from figure 29(a) or (b) for cantilever overhangs on one or both sides respectively, for the appropriate values of I_{bc}/I_{be} and B_c/B provided that the following limitations are taken into account:

(a) longitudinal stiffeners are of open type (but see note 2 for closed stiffeners); and
(b) I_f and σ_f are constant over the whole segment including cantilevers; and
(c) either:
(1) there are no edge members at the cantilever tips; or
(2) the edge member is unable to carry longitudinal stress because of its structural detailing; or
(3) the radius of gyration of the edge member about its centroidal axis is not less than 1.65 times the radius of gyration of the flange of the main beam

B_c and I_{bc} are the length and the average effective second moment of area, respectively, of the



(a) Cross section

(b) Cross section divided into segments between webs, with cantilevers taken as integral parts of the adjacent internal segment (see 9.15.3.1)

NOTE. K should be taken from appendix F when any of the requirements of 9.15.3.2 are not met.

Figure 28. Segments of transverse members continuous over three or more webs

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cantilever portion of the transverse member (see figure 29).

NOTE 2. For closed longitudinal stiffeners, values of the buckling coefficient K may be either taken conservatively as given above or be determined more accurately from appendix F. For cases not in accordance with the limitations given in (b) or (c), values of the buckling coefficient K may be determined from appendix F.

9.15.4 *Loading on transverse members*

9.15.4.1 *Effects to be considered.* Transverse members should be designed to resist the load effects due to the following:

(a) dead and live loading placed locally over the transverse members;
(b) transverse distribution of loading between the main beams (see also **9.15.4.5**);
(c) restraint of distortion of box girders;
(d) creep and shrinkage of concrete, and differential temperature (but see **9.2.1.3** and **9.2.3.2**);
(e) longitudinal curvature of the main flange in elevation (see **9.15.4.2**);
(f) change of longitudinal slope of the main flange (see **9.15.4.3**);
(g) profile deviations from the specified profile in elevation of a compression flange (see **9.15.4.4**).

Methods of application of the loadings given in (a) to (g) are given in **9.15.4.5**.

9.15.4.2 *Flanges curved in elevation.* When a flange of a main beam is curved in elevation, the transverse member should be designed to resist a load equal to:

$$\frac{P_f a}{R_f}$$

where

P_f is the longitudinal force in the flange
a is the distance between transverse members
R_f is the radius of curvature of the flange.

The direction of the load on the transverse member should be assumed to be such as to increase the flange curvature in a compression flange, or decrease it in a tension flange.

9.15.4.3 *Change of flange slope.* At any change in the longitudinal slope of the flange of the main beam, a transverse member should be designed to resist a load equal to the component of the longitudinal flange force in the plane of the web of the transverse member.

9.15.4.4 *Profile deviation in compression flanges.* A destabilizing force should be assumed to act on each portion of each transverse member, in either direction in the plane of its web, in such a way as to maximize the load effects at the section of the transverse member under consideration. The magnitude of this force should be taken as:

(a) $P_f/200$ per unit width, distributed uniformly along the length of portions of transverse members between adjacent main beam webs; and
(b) $P_{fc}/200$ per unit width, distributed uniformly along the length of any cantilever portions of transverse members; and
(c) $P_{fc}\,A_{sc}/160A_{fc}$ concentrated at a longitudinal stiffening member at the cantilever tips, if this is capable of transmitting longitudinal stress;

where

P_f is the average longitudinal compressive force per unit width in the relevant portion of flange between adjacent webs of main beams
P_{fc} is the average longitudinal compressive force per unit width in the flange attached to a cantilever portion of a transverse member
A_{sc} is the area of the cross section of the longitudinal stiffening member at the cantilever tip
A_{fc} is the area of the cross section per unit width of the flange attached to the cantilever portion of the transverse members, inclusive of any longitudinal stiffeners. In the case of a concrete or composite flange, the area of the concrete divided by the modular ratio should be included.

9.15.4.5 *Application of loading*

9.15.4.5.1 Loadings which cause substantially the same effects on a series of transverse members along the length of the flange (e.g. the effects of dead load, or of creep and shrinkage of concrete) should be considered to be applied directly to the transverse member under consideration.

9.15.4.5.2 Loadings which may cause substantially different effects on adjacent transverse members (e.g. the effect of traffic loading on a deck supported by the transverse members) should be distributed over several transverse members. Any suitable elastic method (e.g. a grillage analysis) may be used to determine the maximum effects on the transverse member under consideration, neglecting the destabilizing effect of longitudinal flange stress in compressive flanges.

NOTE. A non-linear analysis including the destabilizing effect of longitudinal flange stress may be used to determine the maximum effects of the load distribution on the transverse member under consideration, in which case, when applying **9.15.4.5.4**, $i_2 = 1.0$.

9.15.4.5.3 For tension flanges, the effects from **9.15.4.5.1** and **9.15.4.5.2** should be superimposed.

9.15.4.5.4 For compression flanges, the effects from **9.15.4.5.1** and **9.15.4.5.2** should be multiplied by the destabilizing factors given below and then superimposed as follows.

(a) Factor for effects from **9.15.4.5.1**:

$$i_1 = 1 + \frac{\ell_i}{L_f}\,\frac{\sqrt{I_{be\,min}}}{3\sqrt{I_{be}} - \sqrt{I_{be\,min}}}$$

(b) Factor for effects from **9.15.4.5.2**:

$$i_2 = \frac{3\sqrt{I_{be}}}{3\sqrt{I_{be}} - \sqrt{I_{be\,min}}}$$

where

$$\ell_i = B\left(\frac{24 I_f a}{K I_{be}}\right)^{0.25}$$

L_f is the length of compression flange of main beam between points of contraflexure
$I_{be\,min}$ is the minimum value of I_{be} to satisfy **9.15.3.2**
I_{be}, B, a, I_f and K are as defined in **9.15.3.2**.

NOTE. i_1 and i_2 may be conservatively taken as $(2L_f + \ell_i)/2L_f$ and 1.5, respectively.

9.15.4.4 Profile deviation in compression flanges

Add at end:

When a survey of the deflections of the transverse members has been carried out, the factors of 200 and 160 in the denominator of (a), (b) and (c) above shall be replaced by

$$\frac{G}{3\Delta_c} \text{ and } \frac{G}{3.75\Delta_c}$$

respectively where G and Δ_c are defined in Table 5 of Part 6. The actual value of Δ_c to be used shall be the largest measured value at any point of the span of the transverse member being considered and shall not be taken as less than 3mm. in any circumstances.

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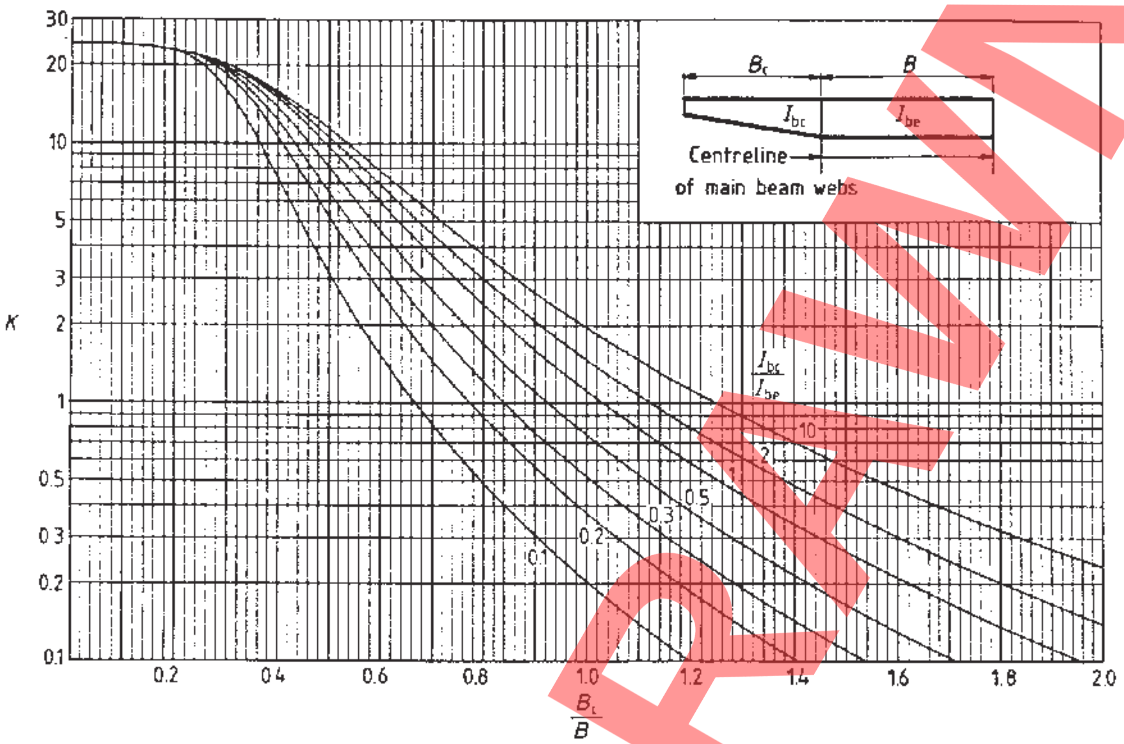
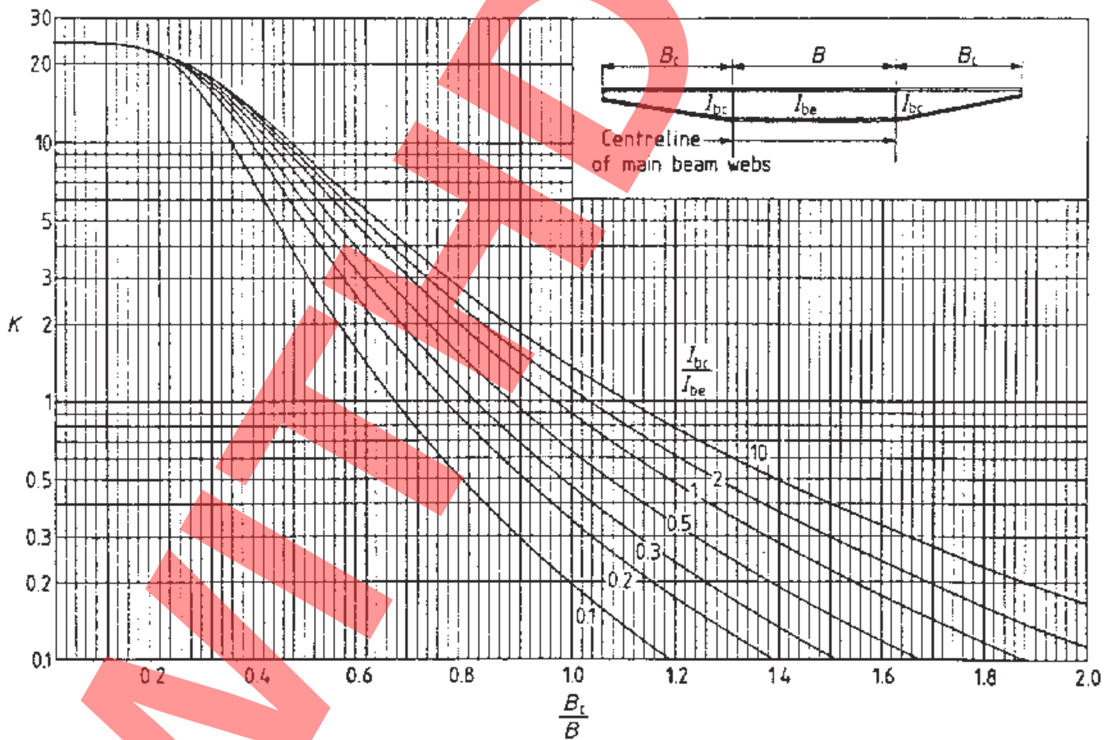


(a) With cantilevers on one side only (see 9.15.3)



(b) With cantilevers on both sides (see 9.15.3)

NOTE. For basis of curves, see G.14.

Figure 29. Buckling coefficient K for transverse members

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9.15.5 *Strength of transverse members.* The capacity of a transverse member, or the limiting stress in the member under the action of the loadings, applied as in **9.15.4**, should be determined from **9.9** or **9.10** and **9.11**, as appropriate, using the effective section defined in **9.15.2.2**.

9.16 Intermediate internal cross frames in box girders

9.16.1 *General.* This section applies to ring or braced intermediate cross frames provided in box girders to restrict distortion of the cross section, (see figure 30), subject to the limitations and provisions of **9.16.2**.

NOTE. The design of plated intermediate diaphragms is not covered by this Part, but they may be used provided special analysis is undertaken.

9.16.2 *Limitations*

9.16.2.1 *Girder layout.* Girders should be nominally symmetric (i.e. ignoring cross fall or superelevation), of rectangular or trapezoidal cross section, and with webs in single planes inclined at less than 45° from the vertical.

Girders should be of single cell form, with or without interconnecting cross members and cantilevers.

Girders should not be subject to internal pressure effects due to sealing.

9.16.2.2 *Cross frames.* The cross frame should be in a single plane, within ±5° of the normal to the axis of the girder in elevation and within ±10° in plan, and within ±5° of a vertical plane.

9.16.2.3 *Corner stiffening.* Each corner of the cross frame (i.e. at the intersections of flange and web of main box) should be stiffened as necessary to withstand the bending moments acting at the corner. In the absence of such stiffening, no effective width of box web or flange (see **9.16.4.1**) should be assumed to act with the cross frame at this point.

9.16.3 *Load effects to be considered.* The design should be such as to resist load effects given in **9.13.3** and **9.15.4** with the relevant loads applied as described in **9.13.4** and **9.15.4.5**.

9.16.4 *Ring cross frames*

9.16.4.1 *Effective section of transverse members.* When calculating the stiffness of a flange transverse member forming part of a ring frame, its effective section should be determined in accordance with **9.15.2.1**.

When calculating flexural stresses or moment capacity in the plane of the frame, the effective section of a flange transverse member should be determined in accordance with **9.15.2.2**.

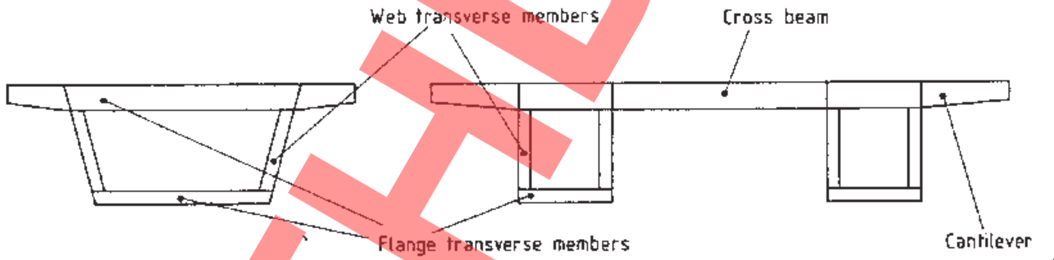
When determining the effective section of a flange transverse member forming part of a ring frame for the purposes of calculating axial stresses or axial load capacity:

(a) a width of flange plate equal to 16 times the flange plate thickness (if available) should be taken to act each side of the web of the transverse member, when the axial effect is compressive; or

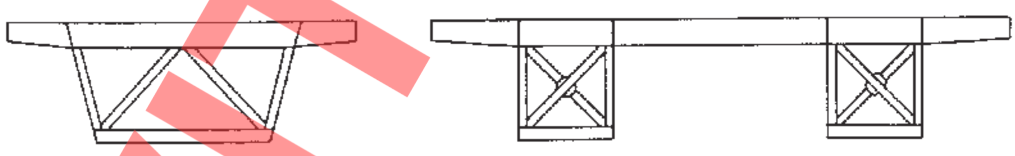
(b) the effective section should be determined in accordance with **9.15.2.2**, when the axial effect is tensile.

When determining the effective section of a web transverse member forming part of a ring frame, the effective section should be determined in accordance with **9.13.2** for all purposes.

9.16.4.2 *Analysis.* Global load effects and stresses in ring cross frames should be derived from analysis undertaken



(a) Ring cross frames



(b) Braced cross frames

Figure 30. Internal intermediate cross frames in box girders

Add new clause 9.15.6

9.15.6 Compression nange transverse members with insufficient stiffness to prevent overall buckling of the Bange, or with insufficient strength

9.15.6.1 General

When the stiffness of the effective transverse member (when assessed in accordance with **9.15.3**) is not capable of restricting the buckling wavelength of the flange to the spacing between cross frames, assessment may be made by an appropriate method that caters for overall buckling of flanges. Appropriate methods are by means of a full analysis in accordance with **9.15.6.2** or by utilising the critical stress of the flange in accordance with **9.15.6.3** of the accompanying Advice Note.

For both approaches the effective sections to be used shall be as set down in **9.15.2**, and the effects to be considered shall be as set down in **9.15.4.**

Further to the above, advantage may be taken of the reduced destabilising effects that can be obtained by utilising more exact stress proportions and distributions in the flange. Also allowance may be made for measured imperfections and the effect of the mode of buckling on imperfections assumed. These procedures may also be applied in cases where strength is initially assessed as insufficient.

9.15.6.2 Structural model when a full analysis is utilised

The model shall be in the form of a non-linear analysis that fully takes into account the stiffness of the orthotropic deck systems and the magnified stresses resulting from deformation of the cross girders due to combined actions of longitudinal deck stresses and imperfections of the cross girders. The structural model to be analysed shall include an initially deformed shape of the flange. In the absence of a survey of the actual flange, the relevant deformations specified in Item 5 of Table 5 of B.S. 5400: Part 6 shall be used with the peak values occurring at the mid points of each span of the transverse member or at the cantilever tips, and with a smooth curve between. Alternate spans of a particular transverse member shall be deformed up and down.

Adjacent transverse members along the bridge shall be deformed up and down either alternately, or alternately in groups of two, three ... etc., whichever eventually gives the highest forces and moments. Consideration shall also be given to having an undeformed transverse member between the up and down groups.

The Computer program shall be capable of analysing displacement in all six primary degrees of freedom (three linear, three rotational) and shall take into account the change of geometry under load. Member material behaviour may be linear elastic. No allowance for plasticity shall be made in the analysis.

The compressive load in the flange shall be applied at the end and side boundaries of the model, as appropriate. The transverse loads on the flange and transverse members shall be those specified in **9.15.4.1**(a) to (f) insofar as they are not otherwise taken into account in the model. Loading between transverse members shall be applied directly at the appropriate position and not distributed as described in **9.15.4.5**. The loads shall not be magnified by a destabilising factor as this will be taken account of in the non-linear analysis.

If a survey is made of the actual deformations, these may be used directly subjected to a minimum of 3mm but increased in the model by a factor of 1.2. If the result of the analysis shows a deflected form radically different from the measured form, further analyses shall be made with the initial deformation conforming more closely to the final pattern.

9.16.1 General

Add at end:

Assessment of plated intermediate diaphragms shall be assessed in accordance with **9.18**.

9.16.2.1 Girder layout

Add at end:

Cross frames in girders not complying with the above shall be assessed in accordance with **9.16.6**.

9.15.5 ***Strength of transverse members.*** The capacity of a transverse member, or the limiting stress in the member under the action of the loadings, applied as in **9.15.4**, should be determined from **9.9** or **9.10** and **9.11**, as appropriate, using the effective section defined in **9.15.2.2**.

9.16 Intermediate internal cross frames in box girders

9.16.1 ***General.*** This section applies to ring or braced intermediate cross frames provided in box girders to restrict distortion of the cross section, (see figure 30), subject to the limitations and provisions of **9.16.2**.

NOTE. The design of plated intermediate diaphragms is not covered by this Part, but they may be used provided special analysis is undertaken.

9.16.2 ***Limitations***

9.16.2.1 *Girder layout.* Girders should be nominally symmetric (i.e. ignoring cross fall or superelevation), of rectangular or trapezoidal cross section, and with webs in single planes inclined at less than 45° from the vertical.

Girders should be of single cell form, with or without interconnecting cross members and cantilevers.

Girders should not be subject to internal pressure effects due to sealing.

9.16.2.2 *Cross frames.* The cross frame should be in a single plane, within ±5° of the normal to the axis of the girder in elevation and within ±10° in plan, and within ±5° of a vertical plane.

9.16.2.3 *Corner stiffening.* Each corner of the cross frame (i.e. at the intersections of flange and web of main box) should be stiffened as necessary to withstand the bending moments acting at the corner. In the absence of such stiffening, no effective width of box web or flange (see **9.16.4.1**) should be assumed to act with the cross frame at this point.

9.16.3 ***Load effects to be considered.*** The design should be such as to resist load effects given in **9.13.3** and **9.15.4** with the relevant loads applied as described in **9.13.4** and **9.15.4.5**.

9.16.4 ***Ring cross frames***

9.16.4.1 *Effective section of transverse members.* When calculating the stiffness of a flange transverse member forming part of a ring frame, its effective section should be determined in accordance with **9.15.2.1**.

When calculating flexural stresses or moment capacity in the plane of the frame, the effective section of a flange transverse member should be determined in accordance with **9.15.2.2**.

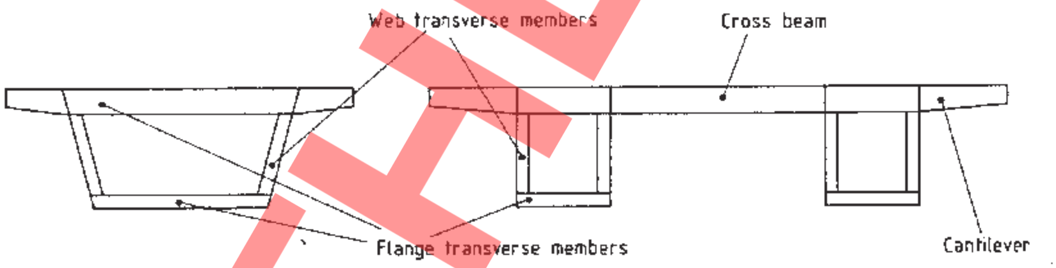
When determining the effective section of a flange transverse member forming part of a ring frame for the purposes of calculating axial stresses or axial load capacity:

(a) a width of flange plate equal to 16 times the flange plate thickness (if available) should be taken to act each side of the web of the transverse member, when the axial effect is compressive; or

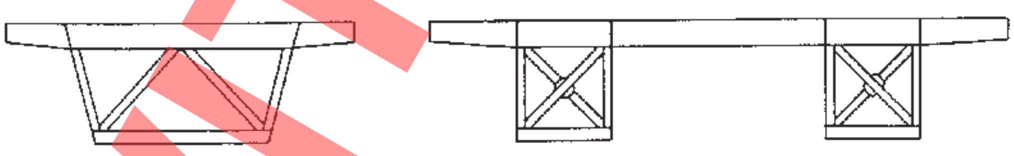
(b) the effective section should be determined in accordance with **9.15.2.2**, when the axial effect is tensile.

When determining the effective section of a web transverse member forming part of a ring frame, the effective section should be determined in accordance with **9.13.2** for all purposes.

9.16.4.2 *Analysis.* Global load effects and stresses in ring cross frames should be derived from analysis undertaken



(a) Ring cross frames



(b) Braced cross frames

Figure 30. Internal intermediate cross frames in box girders

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9.16.2.2 Cross frames

Add at end:

Cross frames not complying with these limitations shall be assessed in accordance with this Standard with the exception of Appendix B, provided that the analytical models used fully account for the plane direction of the frames and interconnection between the frames and longitudinal members.

9.16.3 Load effects to be considered

Add at end:

The forces and stresses due to torsion to be carried by a frame shall be determined from elastic analysis (see **9.16.4.2**) or, for boxes with any web inclination and provided that the cross frames comply with **9.16.2.2**, shall be derived in accordance with Appendix **B.3.4**.

Where the webs of the box girder are inclined to the vertical, horizontal components of load induced in top and bottom transverse members shall also be considered.

in accordance with **9.4.1**, using the effective section properties defined in **9.16.4.1**, and incorporating the local load effects referred to in **9.16.3**.

9.16.4.3 *Strength of ring cross frame members.* Components of ring cross frames attached to the web or flange plates of main beams should comply with the strength provisions of **9.13.5**, **9.13.6** and **9.15.5**, as appropriate.

9.16.5 *Braced cross frames*

9.16.5.1 *General.* Braced cross frames should be designed to satisfy all the provisions of **9.16.4** and in addition those of **9.16.5.2** and **9.16.5.3**.

9.16.5.2 *Load effects to be considered.* Additional account should be taken of bending moments induced by eccentricities at connections, incomplete triangulation and application of loading other than at points of intersection between members.

9.16.5.3 *Design of braced cross frame members.* The design of compression and tension members of braced cross frames, not attached to the web or flange plate of main beams, should be in accordance with clauses **10** and **11**, respectively.

9.17 Diaphragms in box girders at supports

9.17.1 ***General.*** Diaphragms should be provided at supports of box girders to transfer applied loads to the bearings. Subject to the limitations and provisions of **9.17.2**, unstiffened and stiffened diaphragms should be designed in accordance with **9.17.5** and **9.17.6**, respectively, on the basis of the loadings and effective sections given in **9.17.3** and **9.17.4**, respectively.

The diaphragm/web junctions should meet the provisions of **9.17.7**. Deck cross beams and/or cantilevers supporting the deck and located in the plane of a diaphragm should meet the provisions of **9.17.8**.

The geometric notation used is shown in figure 31.

9.17.2 *Limitations*

9.17.2.1 *Box girders.* Box girders should be of nominally rectangular cross section or of nominally trapezoidal cross section with webs in single planes inclined at less than 45° from the vertical, and when unstiffened, should be nominally symmetrical about a vertical axis (i.e. ignoring cross fall or superelevation).

Box girders should be of a single cell form with or without interconnecting cross members and cantilevers

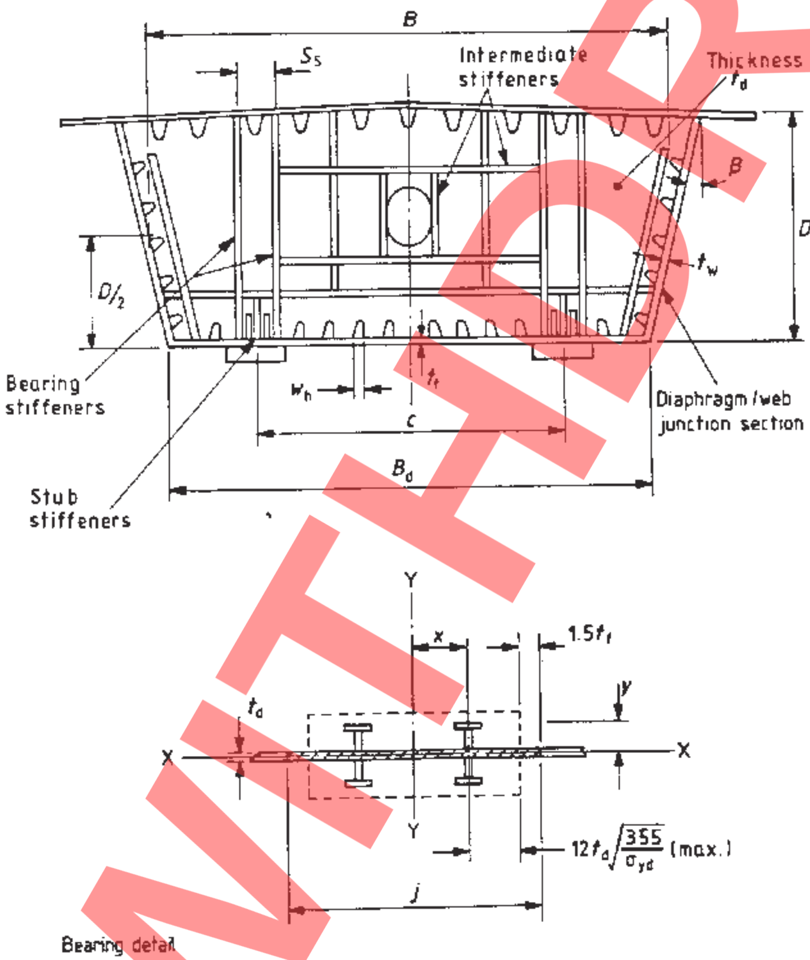

Figure 31. Geometric notation for diaphragms

Add new clause 9.16.4.4:

9.16.4.4 Ring frame corners

The strength of the connection between web transverse members and flange transverse members shall be adequate to transfer the forces and moments from one member to the other. In determining the strength of the connection, the web and flange shall only be considered to act with the transverse member if the corner is stiffened, see **9.16.2.3**

Add new clauses 9.16.6 and 9.16. 7:

9.16.6 Cross frames not complying with limitations

Where the adequacy of cross frames to girders not complying with the limitations defined in **9.16.2** is to be assessed, the strengths of the components of the frames shall be determined from this Standard subject to the following requirements: Global analysis shall be undertaken in accordance with **7.1** and **7.2**. The structure shall be analysed either by a finite element method with all its primary components modelled or an equivalent grillage provided that the elastic properties of the equivalent members are derived from finite element analysis of the box girders.

Analysis to determine load effects from local loads and reactions including distortional effects shall be undertaken using a finite element method on a model of sufficient extent to ensure that the effects calculated are insensitive to assumed end conditions.

9.16.7 Cross girder stiffness

Where distortional and warping stresses in the box girders are calculated in accordance with Appendix B the stiffness of a cross girder shall comply with the requirements of **B.3.4**. Where the stiffness requirements are not complied with, the stresses shall be derived in accordance with **8.3.**

9.17 Diaphragms in box girders at supports

9.17.1 General

Add at end:

Where the adequacy of a stiffened diaphragm not complying with the limitations given in **9.17.2** is to be assessed, the assessment shall be undertaken in accordance with Appendix L.

Alternatively, diaphragms may be assessed in accordance with Appendix L.

and should not be subject to internal pressure effects due to sealing.

9.17.2.2 *Diaphragms and bearings.* The plane of the diaphragm should be within $\pm 5°$ to the normal to the axis of the girder in elevation, within $\pm 10°$ in plan, and within $\pm 5°$ of a vertical plane.

The diaphragm should be in a single plane, except as permitted in **9.17.2.4** for starter plates.

Each diaphragm should be supported on a single or twin bearings under each box.

Bearings under unstiffened diaphragms should be symmetrically placed about the vertical axis of the diaphragm.

The contact width j of a stiffened diaphragm above a bearing, as defined in figure 31, should not exceed half the depth of the diaphragm with a single bearing nor one-quarter of the depth of the diaphragm with twin bearings.

A bearing below a stiffened diaphragm should not extend across the width of the diaphragm beyond the line of attachment of a bearing stiffener by more than $12\sqrt{355/\sigma_{yd}}$ times the thickness of the diaphragm plate, where σ_{yd} is the nominal yield stress of the diaphragm plate.

9.17.2.3 *Cross beams and cantilevers.* Where the deck projects beyond the box web and is supported on cross beams and/or cantilevers which are in the plane of a diaphragm, the flanges of such members should provide a continuous load path through each box web and across the diaphragm for the forces they are required to carry. These members should be assumed to be supported by the diaphragm/box web junctions (see **9.17.7** and **9.17.8**).

9.17.2.4 *Starter plates.* Where starter plates are to be used to connect a diaphragm to the box walls, they should either be:

(a) positioned in the plane of the diaphragm and be butt-welded or connected by double cover plates to the diaphragm; or

(b) lap jointed to the diaphragm, provided that a suitable system of stiffening is designed to withstand, in addition to any other load effects, all the moments resulting from the eccentricity of connection.

9.17.2.5 *Stiffeners to diaphragms.* All stiffeners to plate diaphragms should be in accordance with **9.3.4** with b taken as the spacing of stiffeners, or the distance between the stiffener and the box wall, as appropriate. For boxes with sloping walls, the distance between the stiffener and the box wall should be taken at the centre of the length of the stiffener between points of effective restraint.

Bearing stiffeners should be symmetrically placed about the diaphragm plate, unless a special analysis is made of the effects of any eccentricity with respect to that plate.

9.17.2.6 *Plating in diaphragms.* The thickness of plating in an unstiffened diaphragm should be uniform throughout.

9.17.2.7 *Openings in unstiffened diaphragms.* Openings in unstiffened diaphragms should be in accordance with the following:

(a) only one circular opening may be provided on each side of the vertical centreline of the diaphragm within the upper-third of the height of the diaphragm;

(b) the diameter of any such opening should not exceed the least of:

(1) $6t_d$
(2) $D/20$, or
(3) $B/20$

where

t_d is the diaphragm plate thickness
D is the depth of the diaphragm (see figure 31)
B is the width of the diaphragm taken as the average of the widths at the top and bottom flange levels for boxes with sloping webs;

(c) cut-outs for longitudinal stiffeners on the box walls should have the stiffeners connected to the diaphragm plate either by:

(1) welding, along at least one-third of the perimeter of the cut-out, or
(2) cleating to the longitudinal stiffener with at least two bolts or rivets per side of the connection, or by full perimeter welding of the cleat.

In addition, the length of the free edge of any cut-out should not exceed $8t_d\sqrt{355/\sigma_{yd}}$, when any part of this free edge is within a distance $10t_d\sqrt{355/\sigma_{yd}}$ from any part of a bearing plate,

where

σ_{yd} is the nominal yield stress of the diaphragm plate
t_d is the diaphragm plate thickness.

9.17.2.8 *Openings in stiffened diaphragms.* Openings in stiffened diaphragms should be in accordance with the following.

(a) With the exception of openings permitted in item (d), openings should not be positioned within the areas shown shaded in figure 32.

(b) Unstiffened openings should be circular and of diameter not exceeding the least of:

(1) $6t_d$, or
(2) $a/20$, or
(3) $b/20$,

except when

$$\sigma_e \leqslant \frac{\sigma_{yd}}{2\gamma_m\gamma_{f3}}$$

for which the limiting diameter is twice the limits given in (1), (2) and (3),

where

a and b are the panel dimensions

$$\sigma_e = \sqrt{\sigma_{d1}^2 + \sigma_{d2}^2 - \sigma_{d1}\sigma_{d2} + 3\tau^2}$$

σ_{d1}, σ_{d2} and τ are the stresses in the diaphragm plate derived in accordance with **9.17.6.2**
σ_{yd} is the nominal yield stress of the diaphragm plate.

Not more than one such opening should be positioned in a single plate panel.

(c) Stiffened openings should:

(1) be framed on all sides by stiffeners;
(2) have circular corners of radius at least one-quarter of the least dimension of the hole, with no re-entrant corners;
(3) be positioned such that the distance of any edge from an adjacent wall of the box is at least 0.7 times the maximum dimension of the hole parallel to the

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wall, plus the distance from the wall to the tips of any cut-outs in the diaphragm for longitudinal stiffeners (see figure 32), unless the adjacent plate is designed for secondary in-plane stresses.

(d) Cut-outs for longitudinal stiffeners should be in accordance with **9.17.2.7**(c).

9.17.3 *Loading on diaphragms*

9.17.3.1 *Derivation*. The load effects in diaphragms and associated parts of box girders should be derived from global analysis undertaken in accordance with **9.4.1**.

The design methods of **9.17.5** and **9.17.6** use strength provisions that are compatible only with the assumed methods of stress derivation contained therein. Stresses derived by finite element analyses should not be substituted directly for these derived stresses.

9.17.3.2 *Effects to be considered*. Diaphragms should be designed to resist, with due account being taken of any lack of symmetry in the cross section or in the bearing arrangement, the combined effects of the following.

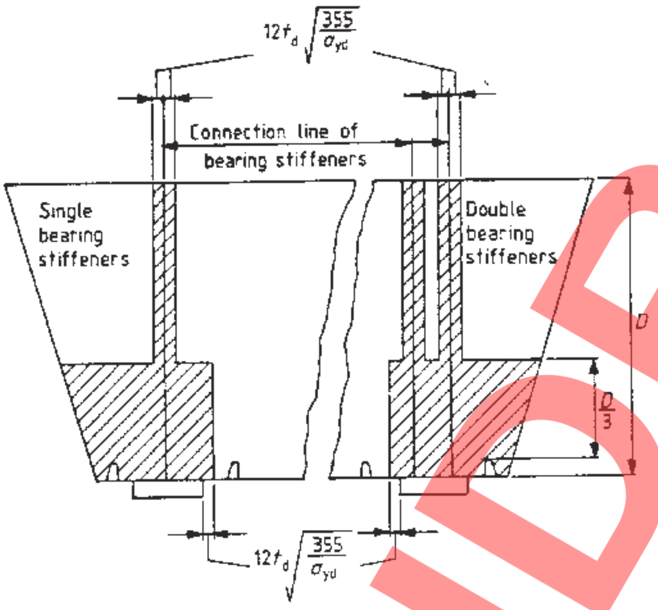
(a) All externally applied loads and the associated bearing reactions.

(b) Changes in bearing reactions and web shears due to:

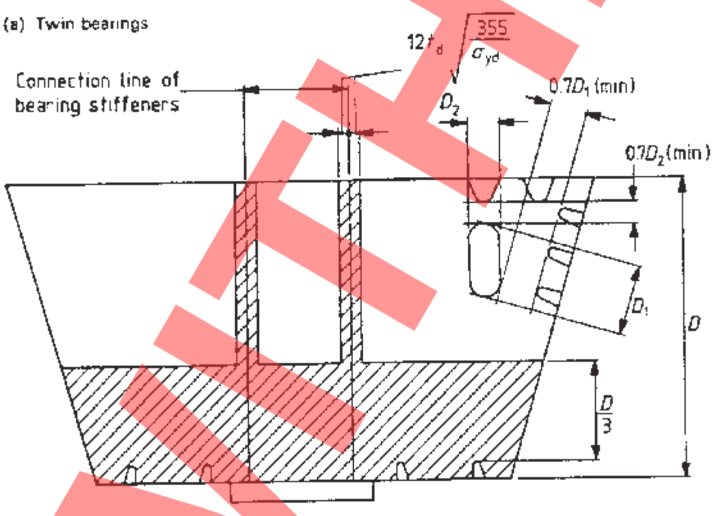
(1) creep, shrinkage and differential temperature;

(2) settlement and other movement of supports.

NOTE. Transverse effects due to (1) may be neglected.



(a) Twin bearings



(b) Single central bearing and positioning of large openings

NOTE 1. Openings are not permitted in shaded areas

NOTE 2. Dimensions are taken from top of cut-outs where present.

Figure 32. Openings in stiffened diaphragms

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(c) Errors in installation of bearings, comprising:

(1) bearing misalignment in plan;
(2) errors in level of a single bearing, or in the mean levels of more than one bearing at any support;
(3) bearing inclination;
(4) departures from common planarity of twin or multiple bearings.

NOTE. Installation errors in (1), (2) and (3), within the tolerances given in Part 9*, are allowed for in **9.17.5** and **9.17.6** and their load effects need not be assessed separately.

(d) Changes in longitudinal slope of box flanges at the diaphragm.
(e) Errors of longitudinal camber in continuous construction. Allowance for this may be made by assuming, at the bearings, a vertical displacement of a support relative to two adjacent supports of 1/5000 times the sum of the adjacent spans.
(f) Out-of-plane moments due to:

(1) longitudinal movements of the bridge;
(2) changes in slope of the bridge;
(3) eccentricity due to bearing misalignment along the span or due to the shape of the bearing; the combined eccentricity for these may be taken as given in **9.14.3.3**;
(4) interconnection between deck and diaphragm stiffeners;
(5) any intended eccentricity of the centroidal axes of the effective section of the bearing stiffeners with respect to the diaphragm plate.

9.17.4 ***Effective sections***

9.17.4.1 *General*. For determining the stresses in a diaphragm, the effective elastic section modulus and effective area of a vertical cross section, and the effective vertical and horizontal shear areas, should be derived in accordance with **9.17.4.2** and **9.17.4.3**. For determining the stresses in stiffeners, their effective sections shall be derived in accordance with **9.17.4.4** or **9.17.4.5**, as appropriate.

In **9.17.4.2** and **9.17.4.3** the references to transverse tension and compression apply to directions normal to the longitudinal axis of the girder.

9.17.4.2 *Vertical sections*

9.17.4.2.1 *General*. The determination of the effective area A_e and the effective section modulus Z_e of a vertical cross section of a diaphragm, should be based on effective areas of box flanges and diaphragm plate as given in **9.17.4.2.2** to **9.17.4.2.5**.

9.17.4.2.2 *Effective flange width*. In calculating an effective area of a box flange, an effective width w_e should be determined separately for each side of the diaphragm and should not exceed any of the following:

(a) one-quarter of the distance of the section under consideration from the nearest web/flange junction; or
(b) half the distance to an adjacent diaphragm or cross beam for any flange in transverse tension, or for a composite flange in transverse compression; or
(c) outside an end diaphragm, the actual width of plate provided; or
(d) $12t_f\sqrt{355/\sigma_{yf}}$ for a non-composite flange in transverse compression. This limit may be increased to one-quarter of the distance to an adjacent diaphragm or cross beam provided that the transverse compressive stress (using the increased width) does not exceed the lesser of:

(1) one-quarter of the longitudinal compressive strength of the flange; or

(2) $0.5\left(\frac{t_f}{b}\right)^2 E$

where

t_f is the thickness of the flange plate
σ_{yf} is the nominal yield stress of the flange plate
b is the spacing of the longitudinal flange stiffeners or the distance between box webs for an unstiffened flange.

9.17.4.2.3 *Effective flange area*. The effective area of a box flange should be determined as follows.

(a) The effective area of steel plate on each side of the diaphragm should be taken as:

$K_c t_f w_e$

where

t_f is the flange thickness
w_e is the effective width on the appropriate side of the diaphragm derived from **9.17.4.2.2**
K_c is a coefficient taken as 1.0, except in the case of a non-composite flange in transverse compression with an effective width greater than $12t_f\sqrt{355/\sigma_{yf}}$, when the value of K_c should be obtained from figure 5 with the dimension a taken as the spacing of longitudinal flange stiffeners and dimension b taken as the distance from the diaphragm to an adjacent cross beam or diaphragm. In using figure 5, the restrained curve should be used for diaphragms at internal supports of continuous beams and the unrestrained curve for diaphragms at end supports.

(b) Any transverse flange stiffeners within the effective width should be ignored.
(c) In composite construction, the effective flange area may include the area of steel reinforcement within the total effective width, and, if subjected to transverse compression, may also include the transformed area of concrete within the total effective width.

9.17.4.2.4 *Diaphragm plate*. Holes within the vertical section of a diaphragm should be deducted. When a stiffened opening is provided, diaphragm plating extending within the framing stiffeners by more than $8t_d\sqrt{355/\sigma_{yd}}$ should be ignored, where t_d and σ_{yd} are the thickness and nominal yield stress, respectively, of the diaphragm plate.

9.17.4.2.5 *Inclined webs*. In the case of box girders with inclined webs, no part of the webs should be included in the vertical section of a diaphragm.

9.17.4.3 *Shear area*. The effective vertical and horizontal shear areas, A_{ve} and A_{he}, should be taken as the net areas of a vertical and horizontal cross section, respectively, of diaphragm plating only.

* In course of preparation.

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9.17.4.4 *Diaphragm stiffeners.* The effective section of a stiffener on a diaphragm should be taken to comprise the stiffener with widths of diaphragm plate on each side of the stiffener where available, not exceeding the lesser of:

(a) half the distance from the stiffener to an adjacent stiffener or to the wall of the box; or

(b) $12\sqrt{355/\sigma_{yd}}$ times the thickness of the diaphragm plate, where σ_{yd} is the nominal yield stress of the diaphragm plate.

Additionally, for a bearing stiffener, the effective width of plate assumed on the side towards the web should not exceed half the distance from the stiffener to the web/bottom flange junction.

The sectional area of discontinuous diaphragm stiffeners should be ignored.

9.17.4.5 *Diaphragm/web junction.* The effective section of this part should be taken to comprise:

(a) a width of web plating each side of the diaphragm (where available) of up to 16 times the web thickness; and

(b) the area of a stiffener, together with a width of diaphragm plate equal to $25t_d$, when there is a stiffener on the diaphragm parallel to the web within $25t_d$ of the web, or a width of diaphragm plate equal to $12t_d\sqrt{355/\sigma_{yd}}$ when there is no stiffener parallel to and within $25t_d$ of the web, where t_d and σ_{yd} are the thickness and nominal yield stress, respectively, of the diaphragm plate.

9.17.5 *Unstiffened diaphragms*

9.17.5.1 *General.* Unstiffened diaphragms in accordance with **9.17.2.1** to **9.17.2.4**, **9.17.2.6** and **9.17.2.7**, should be designed to meet the yield criteria of **9.17.5.4** and the buckling criterion of **9.17.5.5** using reference stress values of **9.17.5.2** and the buckling coefficients of **9.17.5.3**. Web/flange junctions should additionally be in accordance with **9.17.7.3**.

9.17.5.2 *Reference values of in-plane stresses*

9.17.5.2.1 *General.* The stresses in an unstiffened diaphragm, resulting from the load effects given in **9.17.3**, should be determined at the reference point indicated in figure 33, in accordance with **9.17.5.2.2** to **9.17.5.2.4**, for each of the appropriate reference stresses required.

9.17.5.2.2 *Vertical stresses.* The reference value of the in-plane vertical stress σ_{R1} should be taken as follows:

(a) for a diaphragm with a single central bearing:

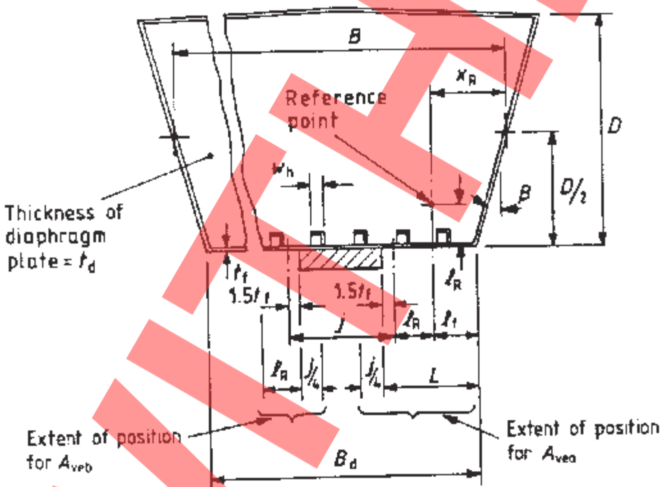
$$\sigma_{R1} = \frac{R_v(1 + 4e/t_d)}{(j - \Sigma w_h)t_d} + 0.77\left(\frac{T_b j}{2I_{yd}}\right)$$

(b) for a diaphragm with a pair of twin symmetrical bearings:

$$\sigma_{R1} = \frac{R_v(1 + 4e/t_d)}{(j - \Sigma w_h)t_d}$$

where

- R_v is the total vertical load transmitted by the diaphragm to one bearing (including the effects of torque on twin bearings)
- T_b is the torsional reaction at a single central bearing
- j is the width of contact of the bearing pad plus 1.5 times the thickness of the bottom flange at each end if available (see figure 31)
- Σw_h is the sum of the widths of any cut-outs for stiffeners within the width j at the level immediately above the flange
- t_d is the diaphragm thickness
- I_{yd} is the second moment of area of the diaphragm plate of width j, excluding cut-outs, about the Y-axis (see figure 31)
- e is the eccentricity of bearing reaction along the span (see **9.14.3.3**).



NOTE. ℓ_R is taken as the least of:

$$8t_d\sqrt{\frac{355}{\sigma_{yd}}},\ \frac{D}{8}\ \text{or}\ \frac{L}{2}$$

Figure 33. Reference point and notation for unstiffened diaphragms

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9.17.5.2.3 *Horizontal stresses*. The reference value of the in-plane horizontal stress σ_{R2} should be taken as:

$$\sigma_{R2} = \left[\left(\frac{K_d \Sigma R_v}{2} + \frac{T}{B}\right)x_R + Q_{fv}\frac{\ell_f}{2}\right]\frac{1}{Z_e} + \frac{\Sigma R_v \tan\beta}{2A_e}$$

where

K_d is a factor to allow for the effects of boundary shears and should be taken as 2.0 in the absence of any special analysis

ΣR_v is the total vertical force transmitted by the diaphragm to the bearings

Q_{fv} is the vertical force transmitted to the diaphragm by the portion of the bottom flange over a width ℓ_f when there is a change of flange slope

ℓ_f is the horizontal distance from the reference point to the nearest edge of the bottom flange

B is as defined in **9.17.2.7**

T is the torque transmitted to the diaphragm in shear through the box walls and from cross beam and/or cantilever loading

x_R is the distance parallel to the bottom flange from the reference point to the web mid-point (see figure 33)

Z_e and A_e are the effective section modulus and the effective area respectively of the diaphragm and flanges at the vertical cross section through the reference point, derived in accordance with **9.17.4.2**

β is the inclination of the box web to the vertical.

9.17.5.2.4 *Shear stresses*

(a) Except as required by (b), the reference value of the in-plane shear stress τ_R should be taken as follows:

$$\tau_R = \left(\frac{\Sigma R_v}{2} + Q_{fv} + \frac{T}{2B}\right)\frac{1}{A_{vea}} + \frac{Q_h}{A_{he}}$$

where

ΣR_v, Q_{fv} and T are as defined in **9.17.5.2.3**

B is as defined in **9.17.2.7**

Q_h is the shear force due to transverse horizontal loads on the bridge transmitted from the top flange to the diaphragm

A_{vea} is the minimum value of the effective vertical shear area, as given in **9.17.4.3**, for any section of diaphragm plating taken between the web and a point $j/4$ inside the outer edge of the bearing (see figure 33)

j is as defined in **9.17.5.2.2**

A_{he} is the effective horizontal shear area, as given in **9.17.4.3** for the section of diaphragm plating through the reference point.

(b) In addition, in the case of diaphragms on twin symmetrical bearings where there is a change in slope of the bottom flange, an alternative value τ_{RI} should be derived from:

$$\tau_{RI} = \left(\frac{T}{c} + \frac{Q_{bv}}{2} - \frac{T}{2B}\right)\frac{1}{A_{veb}} + \frac{Q_h}{A_{he}}$$

where

T is as defined in **9.17.5.2.3**

Q_{bv} is the total vertical force transmitted to the diaphragm by the portion of the bottom flange between the inner edges of the bearings when there is a change in flange slope

c is the distance between centres of bearings

A_{veb} is the minimum value of the effective vertical shear area, as given in **9.17.4.3**, for any section of diaphragm plating taken within a distance ℓ_R from the inner edge of a bearing (i.e. towards the diaphragm centreline) and a distance $j/4$ inside the same inner edge of the bearing (see figure 33)

ℓ_R is as defined in figure 33.

This value τ_{RI} should be adopted if it exceeds the value of τ_R determined in (a).

9.17.5.3 *Buckling coefficient*. In checking the adequacy of an unstiffened plate diaphragm, a coefficient K is required which is given by:

$$K = K_1 K_2 K_3 K_4$$

where

$$K_1 = 3.4 + \frac{2.2D}{B_d}$$

$$K_2 = 0.4 + \frac{j}{2B_d} \text{ for single central bearings, or}$$

$$= 0.4 + \frac{c - j/3}{B_d} \text{ for twin bearings}$$

$$K_3 = 1.0 - \frac{\beta}{100}$$

$$K_4 = 1.0 - \frac{fP_d}{\Sigma R_v + T/\ell_b}\left(\frac{2B}{B_d} - 1\right)$$

D, B_d, B and β (in degrees) are as defined in figure 33

j is as defined in **9.17.5.2.2**

f = 0.55 when $D/B \leqslant 0.7$, or
= 0.86 when $D/B \geqslant 1.5$ with intermediate values found by linear interpolation

ℓ_b = $j/2$ for single central bearings, or
= c for twin bearings

ΣR_v and T are as defined in **9.17.5.2.3**

c is the distance between centres of bearings

$$P_d = W_d + \Sigma\left(\frac{P}{K_5}\right)$$

W_d is the total uniformly distributed load applied to the top of the diaphragm

P is any local load applied to the top of a diaphragm

$$K_5 = 0.4 + \frac{w}{2B - B_d}$$

w is the actual width of the load P, plus an allowance for the dispersal through a concrete flange at an angle of 45° to the vertical, and through a steel flange at an angle of 60° to the vertical.

9.17.5.4 *Yielding of diaphragm plate*. The value of σ_{R1} and $\sqrt{\sigma_{R2}^2 + 3\tau_R^2}$ should not exceed the lesser of:

$$\frac{\sigma_{yd}}{\gamma_m \gamma_{f3}}, \text{ or}$$

$$\frac{\sigma_{yd}}{\gamma_m \gamma_{f3}}\left[1.2 - \frac{(\Sigma R_v + T/\ell_b)D}{1.25KE t_d^3}\right]$$

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where

σ_{R1}, σ_{R2} and τ_R are the reference values of stress as derived in **9.17.5.2.2**, **9.17.5.2.3** and **9.17.5.2.4**, respectively
ΣR_v and T are as defined in **9.17.5.2.3**
ℓ_b and K are as derived in **9.17.5.3**
D is as defined in figure 33
t_d and σ_{yd} are, respectively, the thickness and nominal yield stress of the diaphragm plate.

9.17.5.5 *Buckling of diaphragm plate*. The value of $\Sigma R_v + T/\ell_b$ should not exceed:

$$\frac{0.7KEt_d^3}{D\gamma_m\gamma_{f3}}$$

where

ℓ_b and K are as derived in **9.17.5.3**
t_d is the thickness of the diaphragm plate;
D is as defined in figure 33
ΣR_v and T are as defined in **9.17.5.2.3**.

9.17.6 *Stiffened diaphragms*

9.17.6.1 *General*. Diaphragms in accordance with **9.17.2.1** to **9.17.2.6** and **9.17.2.8** and stiffened by an orthogonal system of stiffeners, generally as indicated in figure 31, should be designed such that the diaphragm plate meets the yield criterion of **9.17.6.4** and the buckling criterion of **9.17.6.5**, using the appropriate stresses determined from **9.17.6.2**.

In addition, all types of stiffeners, as defined in (a), (b) and (c) below, should be designed such that they meet the yield criterion of **9.17.6.6** and the buckling criterion of **9.17.6.7**, using the appropriate stresses determined from **9.17.6.3**.

Web/flange junctions should, additionally, be in accordance with **9.17.7.3** and **19.17.7.4**.

Stiffening may consist of (see figure 31):

(a) bearing stiffeners, which span from a box flange immediately above a bearing, to the flange at deck level;
(b) stub stiffeners, which are short vertical stiffeners above bearings;
(c) intermediate stiffeners, which may be either primary or secondary. Stiffeners spanning between box walls or, if horizontal, between a box web and a bearing stiffener, or between bearing stiffeners should be treated as primary. All other stiffeners should be treated as secondary.

9.17.6.2 *Stresses in diaphragm plates*

9.17.6.2.1 *General*. Relevant stress components should be calculated at the corners of each plate panel, using the appropriate section properties obtained from **9.17.4**, in accordance with **9.17.6.2.2** to **9.17.6.2.4**. When secondary bending stresses have been calculated in accordance with **9.17.2.8**(c) (3) they should be added to these components.

9.17.6.2.2 *Vertical stresses*. Vertical stresses σ_{d1} may be neglected with the exception of those due to:

(a) a change in slope of the main girder flange; and
(b) local wheel loads applied above the diaphragm, which should be calculated in accordance with **9.11.4.1**.

9.17.6.2.3 *Horizontal stresses*. Horizontal stresses σ_{d2} should be calculated under the action of the following.

(a) The in-plane primary moment M on the diaphragm which should be taken as:

$$M = (K_d Q_v + 2Q_T)x_w + K_d Q_c x_c + \Sigma_{i=1,n}(P_i x_i) - R_v x_b + \left(\frac{Q_{tv}\ell_f}{2}\right)$$

where (as shown in figure 34)

Q_v is the total vertical component of symmetric shear transmitted into the diaphragm from one web
Q_T is the vertical component of torsional shear transmitted into the diaphragm from one web, given by $T/2B$

NOTE. In calculating the effects in plate panels occurring in a region bounded by an inclined girder web, a girder top flange, and a vertical line passing through the bottom flange/web junction, an appropriate portion of Q_v and Q_T should be taken on the assumption that they are uniformly distributed over the depth of the girder web.

x_w is the horizontal distance, from the section under consideration to the mid point of the web
Q_c is the vertical component of any cross beam or cantilever shear
x_c is the horizontal distance from the section under consideration to the root of the cross beam or cantilever
P_i is a locally applied deck load between the section under consideration and the web
x_i is the horizontal distance from the section under consideration to the locally applied deck load P_i
R_v is the total vertical load transmitted to one bearing by the diaphragm
x_b is the distance from the section under consideration to the inner edge of the nearest bearing plus $j/4$, for sections between twin bearings, or is zero for all other sections, and for diaphragms with a single bearing
K_d, Q_{tv} and ℓ_f are as defined in **9.17.5.2.3**
j is as defined in **9.17.5.2.2**.

The horizontal bending stress σ_{2b} should be taken as:

$$\sigma_{2b} = \frac{M}{Z_e}$$

where

Z_e is the effective section modulus of a vertical cross section of the diaphragm and flanges, at the point under consideration, derived in accordance with **9.17.4.2**.

(b) The horizontal component of girder shear when the webs are inclined. The horizontal stress σ_{2q} from this component should be taken as:

$$\sigma_{2q} = \frac{Q_v \tan\beta}{A_e}$$

where

Q_v is as defined in (a)
A_e is the effective area of a vertical cross section of the diaphragm and flanges, at the point under consideration, derived in accordance with **9.17.4.2**
β is the inclination of the box web to the vertical.

The total horizontal stress σ_{d2} at the point under consideration should be taken as:

$$\sigma_{d2} = \sigma_{2b} + \sigma_{2q}$$

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9.17.6.2.4 *Shear stresses.* Shear stresses should be calculated under the action of the shear flow q at the section of the diaphragm under consideration. This shear flow q should be taken as constant over the net depth or width of the diaphragm, and as follows:

(a) In sections between a box web and an outer bearing stiffener:

$$q = \frac{Q_v + Q_T + Q_{fv} + Q_c + \Sigma P_i}{D_e} + \frac{Q_h}{B_e}$$

(b) In sections between inner bearing stiffeners where there are twin bearings:

$$q = \left(\frac{Q_v}{4} + \frac{Q_{bv}}{2} + \frac{T}{c} - Q_T\right)\frac{1}{D_e} + \frac{Q_h}{B_e}$$

(c) In sections between pairs of bearing stiffeners above one of a pair of bearings, up to the height of longitudinal flange stiffener cut-outs:

$$q = \left(\frac{5Q_v}{8} + \frac{T}{2c}\right)\frac{1}{D_e} + \frac{Q_h}{j - \Sigma w_h}$$

(d) In sections between pairs of bearing stiffeners above a single bearing, up to the height of longitudinal flange stiffener cut-outs:

$$q = \left(\frac{Q_v}{4} + \frac{T}{s_s} - Q_T\right)\frac{1}{D_e} + \frac{Q_h}{j - \Sigma w_h}$$

where

Q_v, Q_T, Q_c and P_i are as defined in **9.17.6.2.3**
Q_{fv} and T are as defined in **9.17.5.2.3**
Q_h, Q_{bv} and c are as defined in **9.17.5.2.4**
D_e and B_e are the net depth and width of the diaphragm at the point under consideration
j and Σw_h are as defined in **9.17.5.2.2**
s_s is the distance between stiffener centroids.

The shear stress τ in the sections referred to in (a), (b), (c) or (d) should be taken as:

$$\tau = \frac{q}{t_d}$$

where

t_d is the thickness of the diaphragm plate in the panel under consideration.

In sections other than those referred to in (a), (b), (c), or (d) τ may be neglected.

9.17.6.3 *Stresses in diaphragm stiffeners*

9.17.6.3.1 *General.* Stresses in stiffeners should be determined in accordance with **9.17.6.3.2** to **9.17.6.3.4**, using the appropriate effective stiffener sections obtained from **9.17.4.4**.

The stiffener types: bearing, stub, primary intermediate and secondary intermediate, are as defined in **9.17.6.1**.

9.17.6.3.2 *Vertical stresses in bearing stiffeners.* The vertical stress σ_{1s} in a bearing stiffener should be taken as:

$$\sigma_{1s} = \frac{P_s}{A_{se}}$$

where

P_s is the total vertical force in the group of bearing stiffeners
A_{se} is the effective cross-sectional area of the group of bearing stiffeners, derived in accordance with **9.17.4.4.**

NOTE. Both values are taken at the level under consideration.

In the absence of openings in the diaphragm between the group of bearing stiffeners and the adjacent web, the vertical force P_s may be assumed to vary linearly from the value of the reaction at the bearing to the value of any reaction transmitted from the deck to the top of the bearing stiffener.

If there are any openings in the diaphragm between the group of bearing stiffeners and the adjacent web, no variation of load over the depth of such openings should be assumed. The variation over the remaining parts of the diaphragm should be assumed to be linear of constant slope. In the case of a diaphragm above a single bearing, an additional vertical stress σ_{1sT} in a bearing stiffener

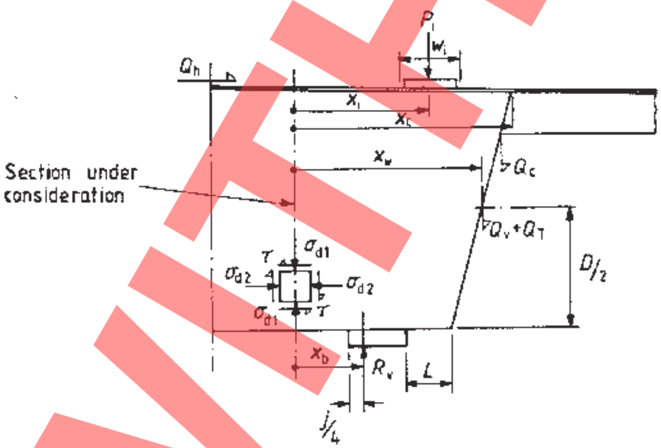

Figure 34. Load effects and notation for stiffened diaphragms

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should be taken as:

$$\sigma_{1sT} = \frac{T_s x}{I_{yse}}$$

where

T_s is the value of the moment in the plane of the diaphragm on the group of bearing stiffeners

x is the horizontal distance of the stiffener under consideration from the centroidal axis, normal to the plane of the diaphragm, of the stiffener group (see figure 31)

I_{yse} is the effective second moment of area of the stiffener group about the same centroidal axis, derived in accordance with **9.17.4.4**.

NOTE. All values are taken at the point under consideration. T_s may be assumed to vary linearly, from the torsional reaction above the bearing, to zero at the top flange level.

Where stub stiffeners are used, the stress calculated as above may be reduced locally by including the area of such stiffeners, provided their connections to the diaphragm plate are adequate to transfer their share of the bearing reaction.

9.17.6.3.3 *Bending stresses in bearing stiffeners.* The bending stress σ_{bs} in a bearing stiffener due to an out-of-plane moment should be taken as:

$$\sigma_{bs} = \frac{M_s y}{I_{xse}}$$

where

M_s is the proportion of the out-of-plane moment carried by the group of bearing stiffeners

y is the distance of the extreme fibre of the stiffener under consideration from the centroidal axis, parallel to the plane of the diaphragm, of the stiffener group (see figure 31)

I_{xse} is the effective second moment of area of the stiffener group about the same centroidal axis, derived in accordance with **9.17.4.4**.

NOTE. All values are taken at the point under consideration.

A proportion of the out-of-plane moment may be assumed to be carried by the flange longitudinal stiffeners, provided due account is taken of this in their design.

Stub stiffeners should not be considered to carry any part of the out-of-plane moment carried by a bearing stiffener group unless they have an adequate out-of-plane shear connection to the bearing stiffeners and/or the box walls.

9.17.6.3.4 *Equivalent stress for buckling check.* The equivalent axial stress σ_{se}, to be used in the buckling check of all stiffeners, should be taken as the maximum value within the middle-third of the length ℓ_s of the stiffener, calculated from:

$$\sigma_{se} = \sigma_a + \frac{1}{A_{se}}\left[\frac{\sigma_q \ell_s^2 t_d k_s}{a_{max}}\left(1 + \frac{\Sigma A_s}{\ell_s t_d}\right) + \tau_h t_d h_h\right]$$

where, for all stiffeners,

A_{se} is the effective cross-sectional area of the stiffener derived in accordance with **9.17.4.4**

ℓ_s is the length of the stiffener between points of effective restraint

t_d is the thickness of the diaphragm plate

k_s is obtained from figure 23 using the slenderness parameter

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

r_{se} is the radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm derived in accordance with 9.17.4.4

σ_{ys} is the nominal yield stress of the stiffener

ΣA_s is the sum of the areas of all stiffeners which intersect the stiffeners being designed, within the length ℓ_s not including any adjacent diaphragm plate

σ_{d2} is derived in **9.17.6.2.3**, for the level being considered, and taken as positive when compressive

σ_{2s} is the average value of σ_{d2} within the middle-third of the length ℓ_s.

σ_a, σ_q, a_{max}, τ_h and h_h are defined as follows for the appropriate type of stiffener.

For bearing stiffeners,

$\sigma_a = \sigma_{1s} + \sigma_{1sT}$

σ_{1s} and σ_{1sT} are as derived in **9.17.6.3.2**

$\sigma_q = \sigma_{2s}$

a_{max} is the maximum spacing of vertical stiffeners which would ensure the adequacy of the diaphragm plate and any horizontal stiffeners, and may conservatively be taken as the actual spacing of vertical stiffeners

τ_h and h_h are taken as zero.

For all intermediate stiffeners,

a_{max} is one-half of the sum of the panel widths on each side of the stiffener. Where the widths vary over the length ℓ_s, the average value of the middle-third should be used

τ is the average shear stress in the panels on either side of the stiffener

τ_h is zero except in the case of the stiffeners framing openings where τ_h is the shear stress which would occur in the plating adjacent to the stiffener if the opening had been fully plated

h_h is zero except in the case of the stiffeners framing openings where h_h is the dimension of the opening parallel to the stiffener.

NOTE. In calculating a_{max} and σ_q no account should be taken of any opening in the diaphragm adjacent to the stiffener (i.e. it should be assumed that a plate of thickness t_d fills the opening).

For horizontal intermediate stiffeners only,

$\sigma_a = \sigma_{d2}$

$\sigma_q = \tau$

For vertical intermediate stiffeners only,

$\sigma_a = 0$

$$\sigma_q = \tau + \sigma_{2s} + \frac{\sigma_{2b\,max} - \sigma_{2b\,min}}{12}$$

$\sigma_{2b\,max}$ and $\sigma_{2b\,min}$ are the maximum and minimum values of σ_{2b}, derived as in **9.17.6.2.3**, within the length ℓ_s and taken as positive when compressive

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9.17.6.4 *Yielding of diaphragm plate*. Plate panels between stiffeners, or between stiffeners and the box walls, should be designed such that at all points in every panel:

$$\sigma_{d1}^2 + \sigma_{d2}^2 - \sigma_{d1}\sigma_{d2} + 3\tau^2 \leqslant \left(\frac{\sigma_{yd}}{\gamma_m \gamma_{f3}}\right)^2$$

where

$\sigma_{d1} = \sigma_{1s} + \sigma_{1sT}$ for parts of plate panels forming part of the effective section of any bearing stiffener, or is the vertical in-plane stress due to local deck loads and change in flange slope, if relevant, for all remaining parts of plate panels

σ_{1s} is defined in **9.17.6.3.2**

σ_{1sT} is as derived in **9.17.6.3.2**, but with the value of x in that clause taken as the dimension from the centroidal axis to the extreme fibre of the effective section of the stiffener group

σ_{d2} is defined in **9.17.6.2.3**

τ is defined in **9.17.6.2.4**

σ_{yd} is the nominal yield stress of the diaphragm plate.

9.17.6.5 *Buckling of diaphragm plate*

9.17.6.5.1 Plate panels need not be checked for buckling provided that:

(a) the cross section of the girder is nominally rectangular;
(b) the ratio of the depth of the diaphragm D to the minimum plate thickness t_d is less than:

$$80\sqrt{\frac{355}{\sigma_{yd}}}$$

(c) the overhang L (see figure 33 or 34) from the outer edge of the bearing to the box web is less than $D/2$;
(d) stiffening is limited to the bearing stiffeners themselves, and any member providing continuity of cross beam or cantilever flanges through the diaphragm;
(e) there is no change in flange slope at the diaphragm.

9.17.6.5.2 If any of the provisions of **9.17.6.5.1** are not satisfied, all plate panels should meet the buckling criterion given in **9.11.4**, but with the following qualifications.

(a) For panels adjacent to an inclined web the panel dimension a should be taken as the maximum horizontal dimension of the panel.
(b) A plate panel of non-constant thickness should be assumed to be of its minimum thickness throughout.
(c) All plate panels adjacent to the box webs or flanges or to boundary stiffeners not more than $25t_d$ from the box walls or to large cut-outs should be treated as unrestrained. Other panels may be treated as restrained.
(d) For a plate panel without horizontal stiffeners, bounded on three sides by the main beam web and the two main beam flanges, the shear stress coefficient K_q should not be taken higher than:

$$\frac{1.3 \times 10^6}{\sigma_{yd}}\left(\frac{t_d}{b}\right)^2\left[1 + \left(\frac{b}{a}\right)^2\right]$$

where

a and b are the length and width of the panel, respectively

t_d and σ_{yd} are the thickness and nominal yield stress, respectively, of the diaphragm plate;

(e) σ_{d2} should be taken as the main longitudinal stress in the plate panel, and hence for the purposes of meeting the buckling criterion of **9.11.4**, σ_{d1} and σ_{d2} as derived in **9.17.6.4** should be taken as σ_2 and σ_1, respectively, in **9.11.4**.

9.17.6.6 *Yielding of diaphragm stiffeners*. A bearing stiffener section should be designed such that, at any point along its length.

$$\sigma_{1s} + \sigma_{1sT} + \sigma_{bs} \leqslant \frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{1s} and σ_{1sT} are as defined in **9.17.6.3.2**

σ_{bs} is as defined in **9.17.6.3.3**

σ_{ys} is the nominal yield stress of the stiffener.

The bearing stress at the point of contact with a flange should be in accordance with **9.14.4.2**.

9.17.6.7 *Buckling of diaphragm stiffeners*. The stiffener section should be such that, at any point within the middle-third of the length of the stiffener:

$$\frac{\sigma_{se}}{\sigma_{ls}} + \frac{\sigma_{bs}}{\sigma_{ys}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{se} is as defined in **9.17.6.3.4**

σ_{bs} is as defined in **9.17.6.3.3** for a bearing stiffener, or is taken as zero for an intermediate stiffener

σ_{ls} is obtained from figure 23 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is the length of the stiffener between points of effective restraint

r_{se} is the radius of gyration of the effective section of the stiffener about its centroidal axis parallel to the plane of the diaphragm derived in accordance with **9.17.4.4**.

9.17.7 ***Diaphragm/web junctions***

9.17.7.1 *General*. The diaphragm/web junction should be designed as a stiffener to the box web, spanning between box flanges, unsupported in the plane of the diaphragm, and with effective section derived as in **9.17.4.5**.

9.17.7.2 *Loading effects to be considered*. The junction should withstand the effects of the following.

(a) All loads transmitted to the diaphragm from the cross beams and/or cantilevers in the plane of the diaphragm. Such loads should be assumed to be applied at the centroidal axis of the effective section, and to vary linearly from a maximum at the top of the junction, to zero at the bottom.
(b) Any forces resulting from tension field action in the adjacent web panels (see **9.13.3.2**). Such forces should be assumed to be applied in the plane of the box web, and to be constant over the height of the junction.
(c) An axial force representing the destabilizing influence of the web (see **9.14.3.2**). This force should be assumed to be applied at the centroidal axis of the effective section, and to be constant over the height of the junction.

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9.17.7.3 *Strength of diaphragm/web junction*

9.17.7.3.1 The maximum stress at any point on the cross section of the junction, at any section in its length, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.3.2 The effective junction section should be such that:

$$\frac{P}{A_{se}\sigma_{ts}} + \frac{M}{Z_{se}\sigma_{ys}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

P and M are, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects specified in **9.17.7.2**, within the middle-third of the length of the junction

A_{se} is the effective area of the junction section (see **9.17.4.5**)

Z_{se} is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (see **9.17.4.5**)

σ_{ts} is obtained from figure 23 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is the total length of the junction section

r_{se} is the radius of gyration of the effective junction section about its centroidal axis parallel to the web derived in accordance with 9.17.4.5

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.4 *Junction restraint provided by diaphragm stiffeners.* Diaphragm/web junctions should be designed in accordance with **9.17.7.1** to **9.17.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress σ_{se} in such stiffeners (see **9.17.6.3.4**) is increased by an amount equal to:

$$\frac{0.025P}{nA_{se}}$$

where

P is as defined in **9.17.7.3**

n is the number of full width horizontal stiffeners

A_{se} is the effective area of the horizontal stiffeners derived in accordance with **9.17.4.4**.

NOTE. In this case ℓ_s in **9.17.7.3** may be taken as the distance between such stiffeners.

9.17.8 ***Continuity of cross beams and cantilevers*** When continuity of cross beams and cantilevers is provided in the plane of a diaphragm, in accordance with **9.17.2.3**, that portion within the box walls should be in accordance with the following.

(a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

(b) If the member providing the continuity in (a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **9.17.6**, it should be designed to withstand, in addition to the load given in (a), an axial force equal to $A_{se}\sigma_{se}$

where

A_{se} is the effective cross-sectional area of the continuity member derived in accordance with **9.17.4.4**

σ_{se} is as specified in **9.17.6.3.4**.

(c) The member providing the continuity in (a) should be designed as a compression member in accordance with **10.1** to **10.6**, and should be assumed to be unrestrained out of the plane of the diaphragm unless provided with effective intermediate restraint. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **9.17.6.3**, a force equal to 2.5% of the maximum axial load in the continuity member including that given in (b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{\sigma_{ts}} + \frac{\sigma_{bs} + \sigma_{b2}}{\sigma_{ys}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{b2} is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle-third of the lengths of the stiffener

σ_{bs}, σ_{ts}, σ_{se} and σ_{ys} are as defined in **9.17.6.7**.

10. Design of compression members

10.1 General. This clause covers the design of straight members of uniform cross section subjected to axial compression or to combined compression and bending.

10.2 Limit state

10.2.1 ***Ultimate limit state.*** Members subjected to axial compression or to combined compression and bending should be designed to satisfy the provisions of clause **10** for the ultimate limit state.

10.2.2 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

10.2.3 ***Serviceability limit state.*** Non-compact truss members (see **10.6.3**) which are not in accordance with item (b) of **12.2.3** should also satisfy the provisions of clause **10** for the serviceability limit state.

10.3 Limitations on shape

10.3.1 ***Unstiffened outstand.*** Unless the free edge of a plate or other outstand is stiffened, the ratio b_o/t_o should not exceed:

$$12\sqrt{\frac{355}{\sigma_y'}}$$

Add new clause 9.18:

9.18 Intermediate diaphragms in box girders

9.18.1 General

This section shall apply to intermediate plated diaphragms provided in box girders to transfer deck loads to the webs, to resist forces due to local changes in slope of the flanges and to restrict distortion of the cross-section.

9.18.2 Limitations

The limitations given in **9.17.2** other than those related to bearings shall be applicable to intermediate diaphragms. Assessment of unstiffened and stiffened intermediate diaphragms shall be carried out in accordance with **9.18.5** and **9.18.6** respectively.

9.18.3 Loading on diaphragms

9.18.3.1 Derivation

The load effects in intermediate diaphragms and associated parts of box girders shall be derived from global and local analysis in accordance with **7.1, 7.2** and **9.4.1**.

9.18.3.2 Effects to be considered

Intermediate diaphragms shall be assessed with due account taken of the application of the load effects given in **9.13.3** and **9.15.4.**

In this context the diaphragm/web junction shall be considered to be equivalent to a web stiffener.

9.18.4 Effective sections

The effective sections of intermediate diaphragms to be used in deriving stresses shall be in accordance with **9.17.4.**

9.18.5 Unstiffened intermediate diaphragms

9.18.5.1 General

Unstiffened diaphragms complying with the limitations of **9.18.2** shall be assessed to the yield criterion of **9.18.5.4** and the buckling criterion of **9.18.5.3** using reference stress values of **9.18.5.2** and buckling coefficients of **9.18.5.3**. Web/diaphragm junctions shall be assessed in accordance with **9.18.7.** Diaphragm stiffness shall be assessed in accordance with **9.18.8,** to ascertain the relevant distribution of warping and distortional stresses in the box girder or diaphragm.

9.18.5.2 Reference values of in-plane stresses

9.18.5.2.1 General

The stresses in an unstiffened diaphragm resulting from the load effects given in **9.18.3** shall be determined in accordance with **9.18.5.2.2** to **9.18.5.2.4.**

9.18.5.2.2 Vertical stresses

The reference value of the in-plane vertical stress σ_{R1} shall be taken as the greater of σ_{R1T} and σ_{R1B}

where

σ_{R1T} is the maximum value of compressive vertical stress on the effective horizontal section of the diaphragm plating beneath the top flange due to deck loading

σ_{R1B} is the maximum value of compressive vertical stress on the effective horizontal section of the diaphragm plating above the bottom flange due to change in slope of the flange or other applied vertical loading

9.18.5.2.3 Horizontal stresses

By reference to Figure 9.18.5A the reference value of the in-plane horizontal stress at a section distance S from the centre of the web, σ_{R2}, shall be taken as the greater of:

$$\left[\sigma_{R2T} + [\sigma_2]\right]$$

or

$$\left[\sigma_{R2B} + [\sigma_2]\right]$$

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9.17.7.3 *Strength of diaphragm/web junction*

9.17.7.3.1 The maximum stress at any point on the cross section of the junction, at any section in its length, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.3.2 The effective junction section should be such that:

$$\frac{P}{A_{se}\sigma_{ts}} + \frac{M}{Z_{se}\sigma_{ys}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

P and M are, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects specified in **9.17.7.2**, within the middle-third of the length of the junction

A_{se} is the effective area of the junction section (see **9.17.4.5**)

Z_{se} is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (see **9.17.4.5**)

σ_{ts} is obtained from figure 23 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is the total length of the junction section

r_{se} is the radius of gyration of the effective junction section about its centroidal axis parallel to the web derived in accordance with **9.17.4.5**

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.4 *Junction restraint provided by diaphragm stiffeners.* Diaphragm/web junctions should be designed in accordance with **9.17.7.1** to **9.17.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress σ_{se} in such stiffeners (see **9.17.6.3.4**) is increased by an amount equal to:

$$\frac{0.025P}{nA_{se}}$$

where

P is as defined in **9.17.7.3**

n is the number of full width horizontal stiffeners

A_{se} is the effective area of the horizontal stiffeners derived in accordance with **9.17.4.4**.

NOTE. In this case ℓ_s in **9.17.7.3** may be taken as the distance between such stiffeners.

9.17.8 ***Continuity of cross beams and cantilevers***

When continuity of cross beams and cantilevers is provided in the plane of a diaphragm, in accordance with **9.17.2.3**, that portion within the box walls should be in accordance with the following.

(a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

(b) If the member providing the continuity in (a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **9.17.6**, it should be designed to withstand, in addition to the load given in (a), an axial force equal to $A_{se}\sigma_{se}$

where

A_{se} is the effective cross-sectional area of the continuity member derived in accordance with **9.17.4.4**

σ_{se} is as specified in **9.17.6.3.4**.

(c) The member providing the continuity in (a) should be designed as a compression member in accordance with **10.1** to **10.6**, and should be assumed to be unrestrained out of the plane of the diaphragm unless provided with effective intermediate restraint. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **9.17.6.3**, a force equal to 2.5% of the maximum axial load in the continuity member including that given in (b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{\sigma_{ts}} + \frac{\sigma_{bs} + \sigma_{b2}}{\sigma_{ys}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{b2} is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle-third of the lengths of the stiffener

σ_{bs}, σ_{ts}, σ_{se} and σ_{ys} are as defined in **9.17.6.7**.

10. Design of compression members

10.1 General. This clause covers the design of straight members of uniform cross section subjected to axial compression or to combined compression and bending.

10.2 Limit state

10.2.1 ***Ultimate limit state.*** Members subjected to axial compression or to combined compression and bending should be designed to satisfy the provisions of clause **10** for the ultimate limit state.

10.2.2 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

10.2.3 ***Serviceability limit state.*** Non-compact truss members (see **10.6.3**) which are not in accordance with item (b) of **12.2.3** should also satisfy the provisions of clause 10 for the serviceability limit state.

10.3 Limitations on shape

10.3.1 ***Unstiffened outstand.*** Unless the free edge of a plate or other outstand is stiffened, the ratio b_o/t_o should not exceed:

$$12\sqrt{\frac{355}{\sigma_y'}}$$

where

$$\sigma_{R2T} = \frac{M}{Z_T}$$

$$\sigma_{R2B} = \frac{M}{Z_B}$$

$$M \;= M_e \;+ F_1\,Y_T \;+ F_2\,Y_B + \frac{Q}{2D}\,(B_T - B_B)\left(Y_B - \frac{D}{2}\right) - Q_T S$$

$$F_1 = \frac{Q_v}{D}\left[S + \frac{(B_T - B_B)}{4}\right]\left\{1 - \frac{1}{B_T}\left[S + \frac{(B_T - B_B)}{4}\right]\right\} + \frac{Q_T}{D}\left[S + \frac{(B_T - B_B)}{4}\right]$$

$$F_2 = \frac{Q_v}{D}\left[S + \frac{(B_T - B_B)}{4}\right]\left\{1 - \frac{1}{B_B}\left[S + \frac{(B_T - B_B)}{4}\right]\right\} + \frac{Q_T}{D}\left[S + \frac{(B_T - B_B)}{4}\right]$$

M_e is the bending moment at the section under consideration due to externally applied loads transmitted to the diaphragm (see 9.15.4) and changes in slope of the bottom flange calculated treating the diaphragm as a simply-supported beam spanning between the mid points of the webs.

Y_T and Y_B are the distances to the top and bottom diaphragm/flange junctions from the centroid of the effective diaphragm section.

Q $= Q_V + Q_T$.

Q_v is one half the total resultant load transmitted to the diaphragm (see **9.15.4**).

$$Q_T = \left(\frac{T}{B_B + B_T}\right)$$

T is the torque about the centre line of the diaphragm due to any eccentricity of externally applied loads transmitted to the diaphragm (see **9.15.4**).

$$[\sigma_2] = (V_T - V_B)\,\frac{Tan\beta}{2A_e}$$

V_T is the total factored vertical load applied to the top of the diaphragm

V_B is the total factored vertical load applied upwards to the bottom of the diaphragm (in either case the coincident values of V_T and V_B shall be taken as those causing the maximum combined stress in strength assessment).

Z_B Z_T are the effective section moduli of the diaphragm and flanges on the diaphragm centre line with respect to the bottom flange and the top flange respectively

t_D, σ_{yd} are as defined in **9.17.5.4**

A_e is the effective area of the diaphragm and flanges at the vertical section under consideration

β is the greater angle of inclination to the vertical of either web

B, B_T, B_B are as defined in Figure 9.18.5A.

9.17.7.3 *Strength of diaphragm/web junction*

9.17.7.3.1 The maximum stress at any point on the cross section of the junction, at any section in its length, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.3.2 The effective junction section should be such that:

$$\frac{P}{A_{se}\sigma_{ts}} + \frac{M}{Z_{se}\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P and M are, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects specified in **9.17.7.2**, within the middle-third of the length of the junction

A_{se} is the effective area of the junction section (see **9.17.4.5**)

Z_{se} is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (see **9.17.4.5**)

σ_{ts} is obtained from figure 23 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is the total length of the junction section

r_{se} is the radius of gyration of the effective junction section about its centroidal axis parallel to the web derived in accordance with 9.17.4.5

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.4 *Junction restraint provided by diaphragm stiffeners*. Diaphragm/web junctions should be designed in accordance with **9.17.7.1** to **9.17.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress σ_{se} in such stiffeners (see **9.17.6.3.4**) is increased by an amount equal to:

$$\frac{0.025P}{nA_{se}}$$

where

P is as defined in **9.17.7.3**

n is the number of full width horizontal stiffeners

A_{se} is the effective area of the horizontal stiffeners derived in accordance with 9.17.4.4.

NOTE. In this case ℓ_s in **9.17.7.3** may be taken as the distance between such stiffeners.

9.17.8 ***Continuity of cross beams and cantilevers*** When continuity of cross beams and cantilevers is provided in the plane of a diaphragm, in accordance with **9.17.2.3**, that portion within the box walls should be in accordance with the following.

(a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

(b) If the member providing the continuity in (a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **9.17.6**, it should be designed to withstand, in addition to the load given in (a), an axial force equal to $A_{se}\sigma_{se}$

where

A_{se} is the effective cross-sectional area of the continuity member derived in accordance with **9.17.4.4**

σ_{se} is as specified in **9.17.6.3.4**.

(c) The member providing the continuity in (a) should be designed as a compression member in accordance with **10.1** to **10.6**, and should be assumed to be unrestrained out of the plane of the diaphragm unless provided with effective intermediate restraint. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **9.17.6.3**, a force equal to 2.5% of the maximum axial load in the continuity member including that given in (b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{\sigma_{ts}} + \frac{\sigma_{bs} + \sigma_{b2}}{\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{b2} is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle-third of the lengths of the stiffener

σ_{bs}, σ_{ts}, σ_{se} and σ_{ys} are as defined in 9.17.6.7.

10. Design of compression members

10.1 General. This clause covers the design of straight members of uniform cross section subjected to axial compression or to combined compression and bending.

10.2 Limit state

10.2.1 ***Ultimate limit state.*** Members subjected to axial compression or to combined compression and bending should be designed to satisfy the provisions of clause **10** for the ultimate limit state.

10.2.2 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

10.2.3 ***Serviceability limit state.*** Non-compact truss members (see **10.6.3**) which are not in accordance with item (b) of **12.2.3** should also satisfy the provisions of clause **10** for the serviceability limit state.

10.3 Limitations on shape

10.3.1 ***Unstiffened outstand.*** Unless the free edge of a plate or other outstand is stiffened, the ratio b_o/t_o should not exceed:

$$12\sqrt{\frac{355}{\sigma_y'}}$$

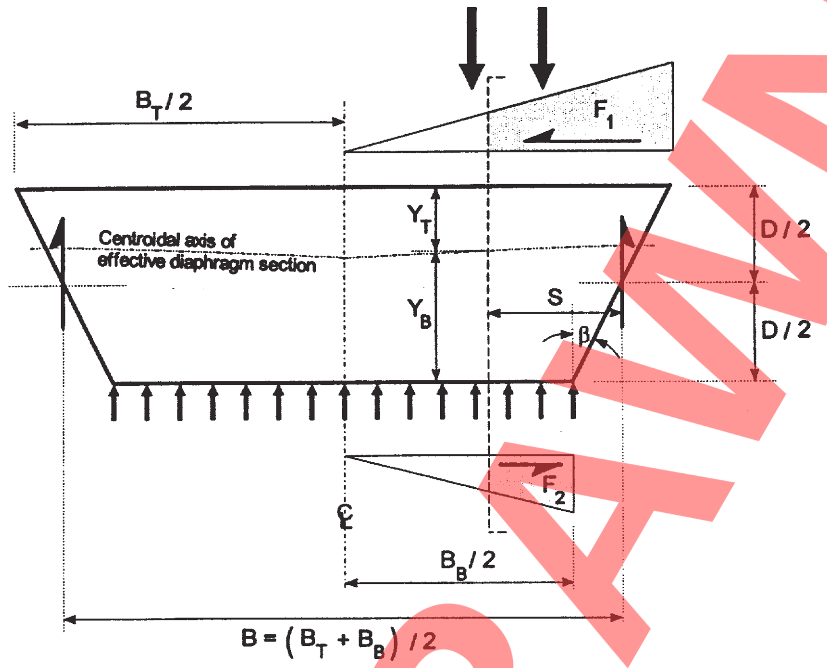

Figure 9.18.5A

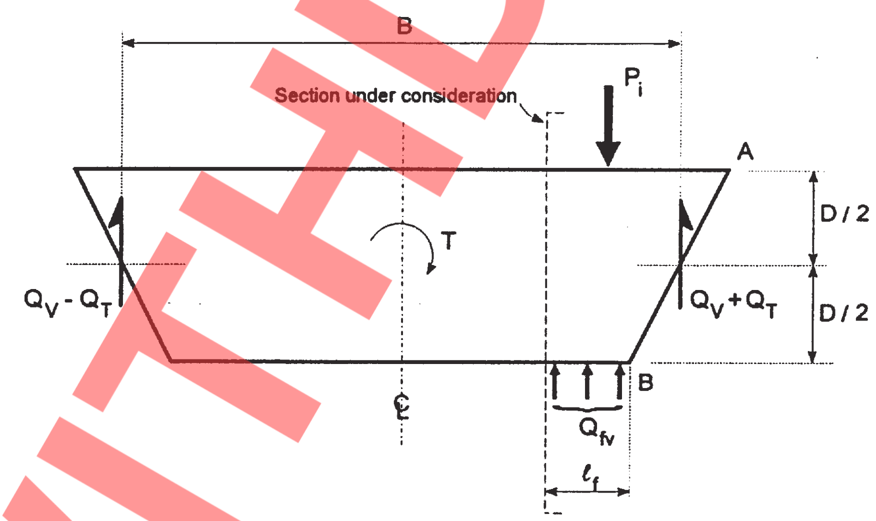

Figure 9.18.5B

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9.17.7.3 *Strength of diaphragm/web junction*

9.17.7.3.1 The maximum stress at any point on the cross section of the junction, at any section in its length, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.3.2 The effective junction section should be such that:

$$\frac{P}{A_{se}\sigma_{ts}} + \frac{M}{Z_{se}\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P and M are, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects specified in **9.17.7.2**, within the middle-third of the length of the junction

A_{se} is the effective area of the junction section (see **9.17.4.5**)

Z_{se} is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (see **9.17.4.5**)

σ_{ts} is obtained from figure 23 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is the total length of the junction section

r_{se} is the radius of gyration of the effective junction section about its centroidal axis parallel to the web derived in accordance with **9.17.4.5**

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.4 *Junction restraint provided by diaphragm stiffeners.* Diaphragm/web junctions should be designed in accordance with **9.17.7.1** to **9.17.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress σ_{se} in such stiffeners (see **9.17.6.3.4**) is increased by an amount equal to:

$$\frac{0.025P}{nA_{se}}$$

where

P is as defined in **9.17.7.3**

n is the number of full width horizontal stiffeners

A_{se} is the effective area of the horizontal stiffeners derived in accordance with **9.17.4.4**.

NOTE. In this case ℓ_s in **9.17.7.3** may be taken as the distance between such stiffeners.

9.17.8 ***Continuity of cross beams and cantilevers***
When continuity of cross beams and cantilevers is provided in the plane of a diaphragm, in accordance with **9.17.2.3**, that portion within the box walls should be in accordance with the following.

(a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

(b) If the member providing the continuity in (a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **9.17.6**, it should be designed to withstand, in addition to the load given in (a), an axial force equal to $A_{se}\sigma_{se}$

where

A_{se} is the effective cross-sectional area of the continuity member derived in accordance with **9.17.4.4**

σ_{se} is as specified in **9.17.6.3.4**.

(c) The member providing the continuity in (a) should be designed as a compression member in accordance with **10.1** to **10.6**, and should be assumed to be unrestrained out of the plane of the diaphragm unless provided with effective intermediate restraint. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **9.17.6.3**, a force equal to 2.5% of the maximum axial load in the continuity member including that given in (b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{\sigma_{ts}} + \frac{\sigma_{bs} + \sigma_{b2}}{\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{b2} is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle-third of the lengths of the stiffener

σ_{bs}, σ_{ts}, σ_{se} and σ_{ys} are as defined in **9.17.6.7**.

10. Design of compression members

10.1 General. This clause covers the design of straight members of uniform cross section subjected to axial compression or to combined compression and bending.

10.2 Limit state

10.2.1 ***Ultimate limit state.*** Members subjected to axial compression or to combined compression and bending should be designed to satisfy the provisions of clause **10** for the ultimate limit state.

10.2.2 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

10.2.3 ***Serviceability limit state.*** Non-compact truss members (see **10.6.3**) which are not in accordance with item (b) of **12.2.3** should also satisfy the provisions of clause 10 for the serviceability limit state.

10.3 Limitations on shape

10.3.1 ***Unstiffened outstand.*** Unless the free edge of a plate or other outstand is stiffened, the ratio b_o/t_o should not exceed:

$$12\sqrt{\frac{355}{\sigma_y'}}$$

9.18.5.2.4 Shear stresses

The reference value of the in-plane shear stress *TR* shall be taken as follows:

$$\tau_R = \left(Q_v + Q_T - \Sigma P_i + Q_{fv}\right)\frac{1}{A_{ve}}$$

where, as shown in figure 9.18.5.B

- Q_v and Q_T are as defined in **9.18.5.2.3**.
- A_{ve} is the minimum effective vertical shear area, as given in **9.17.4.3**.
- ΣP_i is the sum of the vertical applied loads transmitted to the diaphragm between the section considered and the edge of the top flange at point A.
- Q_{fV} is the vertical force transmitted to the diaphragm by the portion of the bottom flange over a width l_f when there is a change of flange slope.
- l_f is the horizontal distance from the section considered to the edge of the bottom flange at point B.

The value of τ_R to be used in yield checks in accordance with **9.18.5.4** is the maximum value within the middle third of the median width, B, of the diaphragm. Additionally, the value on the sections adjacent to the webs shall be applied in yield checks with $\sigma 2 = 0$. For buckling checks τ_R shall be taken as the average shear stress in the diaphragms.

9.18.5.3 Buckling of diaphragm plate

The diaphragm plate shall be assessed in accordance with the criterion given in **9.11.4.4** using the buckling coefficients for an unrestrained panel given in clause **9.11.4.3** in which the stresses defined in **9.11.3** shall be taken as the following:

$\sigma_1 \quad = [\sigma_2]$
$\sigma_b \quad = \sigma_{R2T}$ or σ_{R2B}, whichever is compressive
$t \quad = \tau_R$
$\sigma_2 \quad = \sigma_{R1}$

The panel dimension ‘b’ in Figure 19 shall be taken as the depth of the diaphragm (D in Figure 33) and the dimension ‘a’ shall be taken as the maximum width between box webs.

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9.18.5.4 Yielding of diaphragm plate

The values of σ_{R1}, $\sqrt{3\tau_R}$ and $\sqrt{\sigma_{R2}^2 + 3\tau_R^2}$

shall not exceed $\dfrac{\sigma_{yd}}{\gamma_m \, \gamma_{f3}}$

9.18.6 Stiffened intermediate diaphragms

9.18.6.1 General

Intermediate diaphragms stiffened by an orthogonal system of stiffeners shall be assessed in accordance with the yield and buckling criteria for the plating given in **9.18.6.3.1**. Stiffeners shall be assessed in accordance with the yield and buckling criteria given in **9.18.6.3.2**. Stiffeners which span between box walls shall be treated as primary. All other stiffeners shall be treated as secondary. Web/diaphragm junctions shall be assessed in accordance with **9.18.7**. Diaphragm stiffnesses shall be assessed in accordance with **9.18.8,** to ascertain the treatment of warping and distortional stresses in the box girder or diaphragm.

9.18.6.2 Values of in-plane stresses

9.18.6.2.1 General

The stresses in a stiffened diaphragm resulting from the load effects given in **9.18.3** shall be determined in accordance with **9.18.6.2.2** to **9.18.6.2.4.**

9.18.6.2.2 Vertical stresses

Vertical stresses, σ_d, due to concentrated loads applied to the deck shall be calculated assuming dispersion of load at 45° from the width of contact and diminishing linearly to zero from the level of intersection of the lines of dispersion with the web to the bottom of the diaphragm. Stresses due to changes in slope of the bottom flange shall be calculated from the vertical components of flange force and be assumed to diminish linearly up the height of the diaphragm. The vertical stresses due to top and bottom loads are to be added.

9.18.6.2.3 Horizontal stresses

The horizontal stresses shall be derived in accordance with **9.17.6.2.3.** In-plane bending, σ_{2b}’ shall be

9.17.7.3 *Strength of diaphragm/web junction*

9.17.7.3.1 The maximum stress at any point on the cross section of the junction, at any section in its length, should not exceed:

$$\frac{\sigma_{ys}}{\gamma_m \gamma_{f3}}$$

where

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.3.2 The effective junction section should be such that:

$$\frac{P}{A_{se}\sigma_{ts}} + \frac{M}{Z_{se}\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

P and M are, respectively, the maximum force on the effective junction section and the maximum moment about the centroidal axis parallel to the web due to all the effects specified in **9.17.7.2**, within the middle-third of the length of the junction

A_{se} is the effective area of the junction section (see **9.17.4.5**)

Z_{se} is the lowest section modulus of the effective junction section about the centroidal axis parallel to the web (see **9.17.4.5**)

σ_{ts} is obtained from figure 23 using the slenderness parameter:

$$\lambda = \frac{\ell_s}{r_{se}}\sqrt{\frac{\sigma_{ys}}{355}}$$

ℓ_s is the total length of the junction section

r_{se} is the radius of gyration of the effective junction section about its centroidal axis parallel to the web derived in accordance with **9.17.4.5**

σ_{ys} is the nominal yield stress of the junction section.

9.17.7.4 *Junction restraint provided by diaphragm stiffeners.* Diaphragm/web junctions should be designed in accordance with **9.17.7.1** to **9.17.7.3**, except that full width horizontal stiffeners in the diaphragm may be assumed to offer restraint to the junction in the plane of the diaphragm, provided that the equivalent axial stress σ_{se} in such stiffeners (see **9.17.6.3.4**) is increased by an amount equal to:

$$\frac{0.025P}{nA_{se}}$$

where

P is as defined in **9.17.7.3**

n is the number of full width horizontal stiffeners

A_{se} is the effective area of the horizontal stiffeners derived in accordance with **9.17.4.4**.

NOTE. In this case ℓ_s in **9.17.7.3** may be taken as the distance between such stiffeners.

9.17.8 ***Continuity of cross beams and cantilevers*** When continuity of cross beams and cantilevers is provided in the plane of a diaphragm, in accordance with **9.17.2.3**, that portion within the box walls should be in accordance with the following.

(a) The force in the member providing continuity to the bottom flange of the transverse member should be taken as the moment in the transverse member at the box wall divided by the distance between the mid-plane of the top and bottom flanges of the member. If the force is different at the two box walls a linear variation along the length may be assumed.

(b) If the member providing the continuity in (a) is also required as a horizontal stiffener for a diaphragm designed in accordance with **9.17.6**, it should be designed to withstand, in addition to the load given in (a), an axial force equal to $A_{se}\sigma_{se}$

where

A_{se} is the effective cross-sectional area of the continuity member derived in accordance with **9.17.4.4**

σ_{se} is as specified in **9.17.6.3.4**.

(c) The member providing the continuity in (a) should be designed as a compression member in accordance with **10.1** to **10.6**, and should be assumed to be unrestrained out of the plane of the diaphragm unless provided with effective intermediate restraint. If these restraints are provided by bearing or primary vertical diaphragm stiffeners, such stiffeners should each be designed to resist, in addition to all other forces given in **9.17.6.3**, a force equal to 2.5% of the maximum axial load in the continuity member including that given in (b), if appropriate. This force should be applied, out of the plane of the diaphragm, at the point of intersection of the continuity member and the stiffener providing the restraint. The stiffener should be designed to satisfy the criterion:

$$\frac{\sigma_{se}}{\sigma_{ts}} + \frac{\sigma_{bs} + \sigma_{b2}}{\sigma_{ys}} \leq \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_{b2} is the bending stress induced in the stiffener by the above force, taken as the maximum value within the middle-third of the lengths of the stiffener

σ_{bs}, σ_{ts}, σ_{se} and σ_{ys} are as defined in **9.17.6.7**.

10. Design of compression members

10.1 General. This clause covers the design of straight members of uniform cross section subjected to axial compression or to combined compression and bending.

10.2 Limit state

10.2.1 ***Ultimate limit state.*** Members subjected to axial compression or to combined compression and bending should be designed to satisfy the provisions of clause **10** for the ultimate limit state.

10.2.2 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

10.2.3 ***Serviceability limit state.*** Non-compact truss members (see **10.6.3**) which are not in accordance with item (b) of **12.2.3** should also satisfy the provisions of clause 10 for the serviceability limit state.

10.3 Limitations on shape

10.3.1 ***Unstiffened outstand.*** Unless the free edge of a plate or other outstand is stiffened, the ratio b_o/t_o should not exceed:

$$12\sqrt{\frac{355}{\sigma_y'}}$$

calculated by treating the diaphragm with the associated effective widths of flanges as a simply supported beam spanning between the box webs (span B) and horizontal stresses, σ_{2q}, due to inclination of webs to the vertical shall be calculated in accordance with **9.18.5.2.3**.

9.18.6.2.4 Shear stresses

The values of the in-plane shear stresses, τ, on any section shall be taken as the reference value τ_R as defined in **9.18.5.2.4**.

9.18.6.2.5 Stresses in diaphragm stiffeners

The equivalent stress in a stiffener for buckling check shall be determined from **9.17.6.3.4** as appropriate for intermediate stiffeners with σ_{2b}, σ_{2q} and τ calculated in accordance with **9.18.6.2.3** and **9.18.6.2.4**. Except that σ_a for vertical intermediate stiffeners is not necessarily zero but shall include loading effects due to tension field in accordance with **9.13.3.2** and **9.13.4.** Loading from clause **9.13.3.3** shall be excluded. All additional load effects as defined in **9.18.3.2** shall be considered.

9.18.6.3 Strength criteria

9.18.6.3.1 Diaphragm plating

Plate panels between stiffeners or between stiffeners and box walls shall be assessed inaccordance with the criteria in **9.17.6.4** and **9.17.6.5.**

9.18.6.3.2 Stiffeners

Diaphragm stiffeners shall be assessed in accordance with the criterion given in **9.17.6.7.**

9.18.7 Intermediate diaphragmAveb junctions

The intermediate diaphragm web junction shall be assessed as a stiffener to the box web spanning between box flanges, unsupported in the plane of the diaphragm, in accordance with **9.17.7.2** to **9.17.7.4** using effective sections derived in accordance with **9.17.4.5.**

9.18.8 Intermediate diaphragm stiffness

Where distortional and warping stresses in the box girders are calculated in accordance with Appendix B the stiffness of an intermediate diaphragm shall comply with the requirements of **B.3.4** Where the stiffness requirements are not complied with, the stress shall be derived in accordance with **8.3.**

10.3.1 Unstiffened Outstand

Delete the existing definition for σ_y' *and substitute following definition:-*

σ_y' is the lesser of the nominal yield stress of the material or such lower value of yield stress as would be necessary to meet the strength criteria of the subsequent clauses.

where

b_o is the width of outstand measured from the edge to the nearest line of rivets or bolts connecting it to the supporting part of the member, or to the toe of a root fillet of a rolled section, or, in the case of a welded construction, to the surface of the supporting part of the member (see figure 35(a))

t_o is the mean thickness of the outstand, or the aggregate thickness of the outstand where two or more parts are joined together in accordance with **14.5** or **14.6**

σ_y' is the lesser of the nominal yield stress of the material, or 1.5 times the maximum stress in the member for the ultimate limit state.

10.3.2 ***Stiffened outstand.*** Unless the free edges of stiffened outstands are interconnected tranversely by means of battens, lacing or perforated plates in accordance with **10.8**, **10.9** or **10.10** respectively, the ratio b_o/t_o should not exceed:

$$14\sqrt{\frac{355}{\sigma_y'}}$$

where

b_o, t_o and σ_y' are as defined in **10.3.1** (see figure 35(b)).

10.3.3 ***Circular hollow sections.*** The ratio of outside diameter to wall thickness of a circular hollow section should not exceed:

$$100\sqrt{\frac{355}{\sigma_y'}}$$

where

σ_y' is as defined in **10.3.1**.

10.4 Effective lengths

10.4.1 ***General.*** The effective length of a compression member ℓ_e may be determined either from table 10 or from **10.4.2** for single angles and **12.4** or **12.5** for trusses, or may be determined by an elastic critical buckling analysis. Alternatively, for a strut which is effectively held in position and fully or partially restrained in direction, ℓ_e may be taken as $k_1 L$

where

k_1 is obtained from figure 7(a) taking I_c as the second moment of area of the member about its appropriate centroidal axis

L is the length of the member between end restraints.

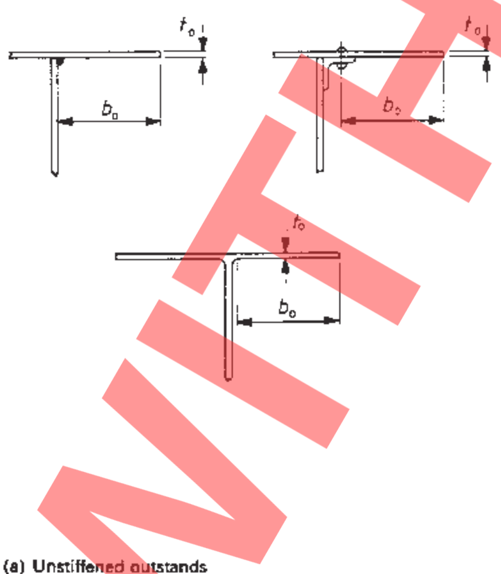
10.4.2 ***Single angle members***

10.4.2.1 *Discontinuous members.* The effective length ℓ_e of a single angle discontinuous member connected by bolts, rivets or welds to a gusset or to a section, provided that they offer effective restraint in the plane considered, should be taken as the length of the member measured between centres of fastenings or groups of fastenings at the ends.

10.4.2.2 *Intersecting members.* The effective length ℓ_e of a single angle bracing member intersected by, and connected to, another such member should be taken as:

(a) *in the plane of bracing*:
0.85 × (the greatest distance between centres of adjacent intersections);

(b) *in any other plane*:
0.7 × (the distance along the bracing member between centroids of the main members).



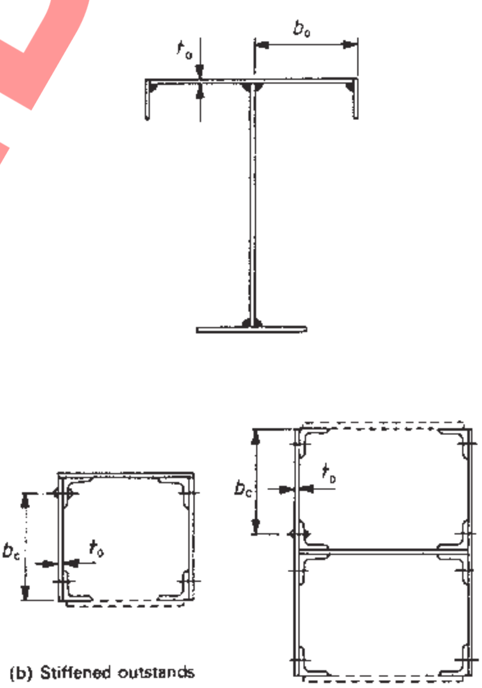


Figure 35. Limitations on shape for compression members

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10.3.3 Circular Hollow Section

Delete the expression and substitute

$$60\left(\frac{355}{\sigma_y}\right)$$

Add new sub-clause 10.3.4:

10.3.4 Assessment of sections not complying with shape limitations

Outstands not complying with **10.3.1** or **10.3.2** shall be assessed in accordance with **9.3.2**. Circular hollow sections not complying with **10.3.3** shall be assessed in accordance with **9.3.6**. This means that a lower value of yield stress shall be determined such that compliance with the strength criteria of **10.6** and **10.3.1, 10.3.2** or **10.3.3** as appropriate is achieved. This lower value of yield stress shall be used in all subsequence assessment of strength, in accordance with **9.3.1**.

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Table 10. Effective length ℓ_e for compression members

Restraint condition	Effective length, ℓ_e
Effectively held in position and restrained in direction at both ends	$0.7L$
Effectively held in position at both ends and restrained in direction at one end	$0.85L$
Effectively held in position at both ends, but not restrained in direction	L
Effectively held in position and restrained in direction at one end; partially restrained in direction but not held in position at the other end	$1.5L$
Effectively held in position and restrained in direction at one end; not held in position or restrained in direction at the other end	$2.0L$

10.5 Effective section

10.5.1 ***General.*** In determining the effective section of a member, consideration should be given to the adequacy of the end fixings to distribute the load effects into all parts of the section.

10.5.2 ***Effective areas***

10.5.2.1 *Members other than circular hollow sections.* The effective area A_e of a compression member, other than a circular hollow section, should be taken as:

$$A_e = \Sigma K_c (k_h A_c)$$

where

K_c = 1.0 for all outstands in accordance with **10.3.1** or **10.3.2** or is determined from the relevant curve of figure 36 for each component plate of the section with edges supported by adjacent components

NOTE. In using figure 36:

b is the unsupported width of plate between adjacent lines of bolts or rivets connecting the plate to supporting parts of the member, or, for welded members, between the surfaces of the supporting parts, or, for rolled sections, clear between root fillets

t is the thickness of the plate, or, if two or more plates are adequately connected together in accordance with **14.5** or **14.6**, the aggregate thickness of such plates.

A_c is the net area of each component part of the member, derived from the gross area, less a deduction across a section perpendicular to the centreline of the member for open holes or clearance holes for pins, black bolts or countersunk bolts. Holes carrying rivets, HSFG, close tolerance or turned barrel bolts, or fully filled plug holes, need not be deducted

k_h = 1.0 for a section free from holes or for a section with one or more holes greater than 40 mm in diameter, or

1.2 for a section in which holes do not exceed 40 mm in diameter, provided that $k_h A_c$ in no case exceeds the gross area of the component part.

10.5.2.2 *Circular hollow sections.* The effective area A_e of a circular hollow section should be taken as:

(a) the net area A_c, when

$$\frac{D}{t}\sqrt{\frac{\sigma_y}{355}} \leqslant 50$$

(b) $A_c\left(1.15 - 0.003\frac{D}{t}\sqrt{\frac{\sigma_y}{355}}\right)$, when $\frac{D}{t}\sqrt{\frac{\sigma_y}{355}} > 50$

where

D is the outside diameter of the section

t is the wall thickness

A_c is the net area of the section, calculated in accordance with **10.5.2.1**

σ_y is the nominal yield stress of the material.

10.6 Compression members without longitudinal stiffeners

10.6.1 ***Axial compression***

10.6.1.1 *Strength.* A member subjected to axial compression should be such that the axial load does not exceed the resistance P_D given by:

$$P_D = \frac{A_e \sigma_c}{\gamma_m \gamma_{f3}}$$

except for single angles connected by one leg (see **10.6.1.2**)

where

A_e is the effective area of the section as defined in **10.5**

σ_c is the least ultimate compressive stress for buckling about any axis to be obtained from σ_c/σ_y, in accordance with figure 37.

NOTE. In using figure 37 the values of $\frac{t}{y}$ and $\frac{\ell_e}{r}\sqrt{\frac{\sigma_y}{355}}$ are required,

where

ℓ_e is the effective length for buckling about any centroidal axis, as defined in **10.4**

r is the radius of gyration of the section about the same axis, based on the gross section of the member, but ignoring battening or lacing

y is the distance from the same axis to the extreme fibre of the section. In an unsymmetrical section the larger value of y should be used

σ_y is the nominal yield stress of the material.

10.6.1.2 *Single angles.* A single angle member connected by one leg may be designed ignoring the eccentricity of the connections with respect to the centroidal axis, unless it is connected at either end by a single bolt or rivet, when its resistance should be taken as 0.8 times the value derived in **10.6.1.1**. However, for a lacing bar, the full value, without the reduction, may be used.

10.6.1.1 Strength

Add at end:

Where in assessment of the adequacy of a compression member allowance is made for initial departures from straightness, Δ_s, measured in accordance with BS5400: Part 6, over a gauge length G equal to the clear length of the compression member, σ_c shall be calculated from the equation in Appendix G16 with η taken as:

$$\eta = \alpha(\lambda - 15) + \left(\frac{\lambda - 15}{\lambda}\right)\left[\frac{(1.2\Delta_s - 0.00012G)y}{r^2}\right]$$

but not less than zero.

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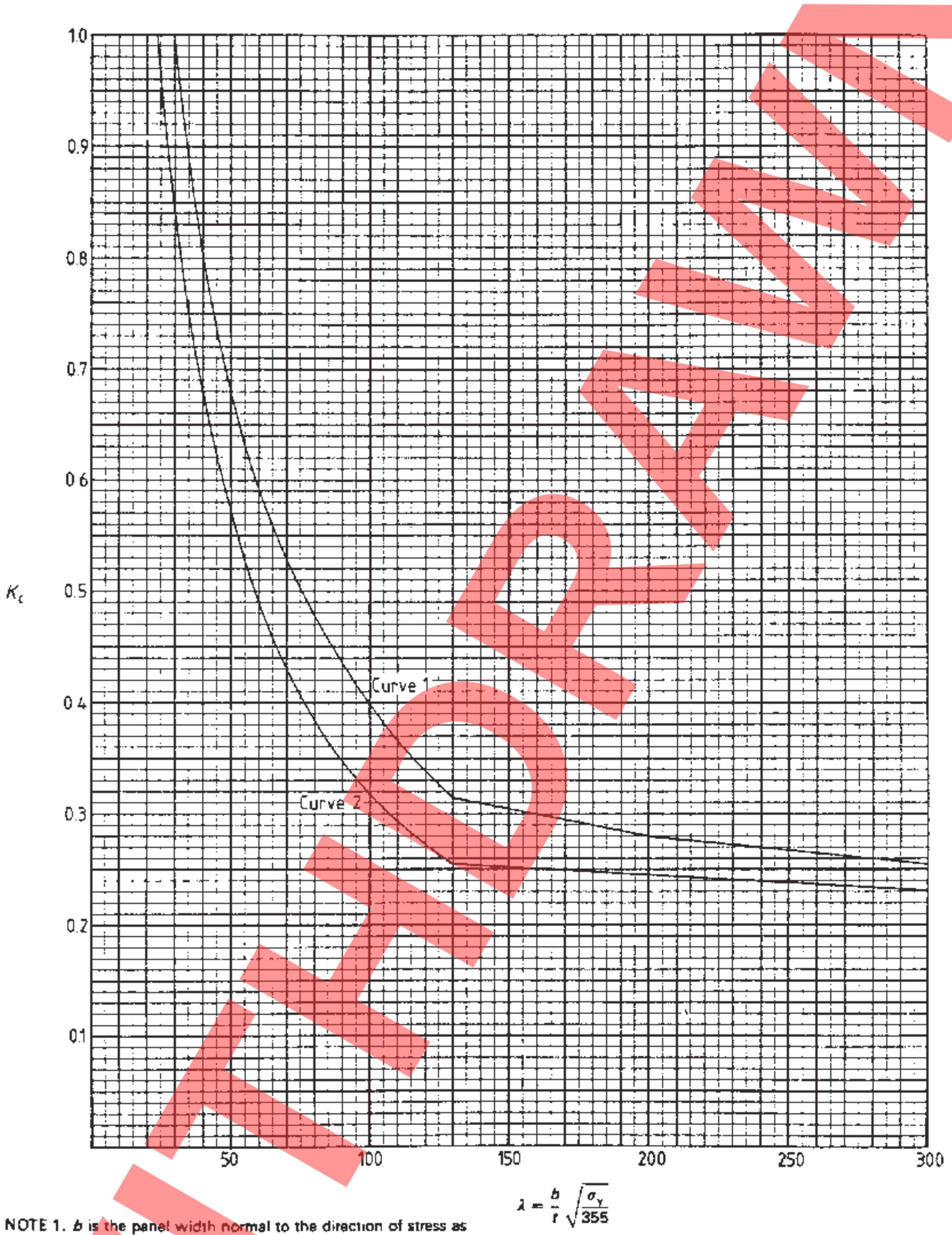
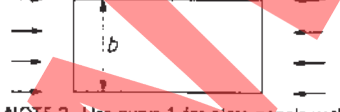


NOTE 1. b is the panel width normal to the direction of stress as follows:



NOTE 2. Use curve 1 for plate panels with bolted or riveted edges. Use curve 2 for rolled sections or plate panels in welded construction.

NOTE 3. For basis of curves, see G.15.

Figure 36. Coefficient K_c for plate panels under direct compression

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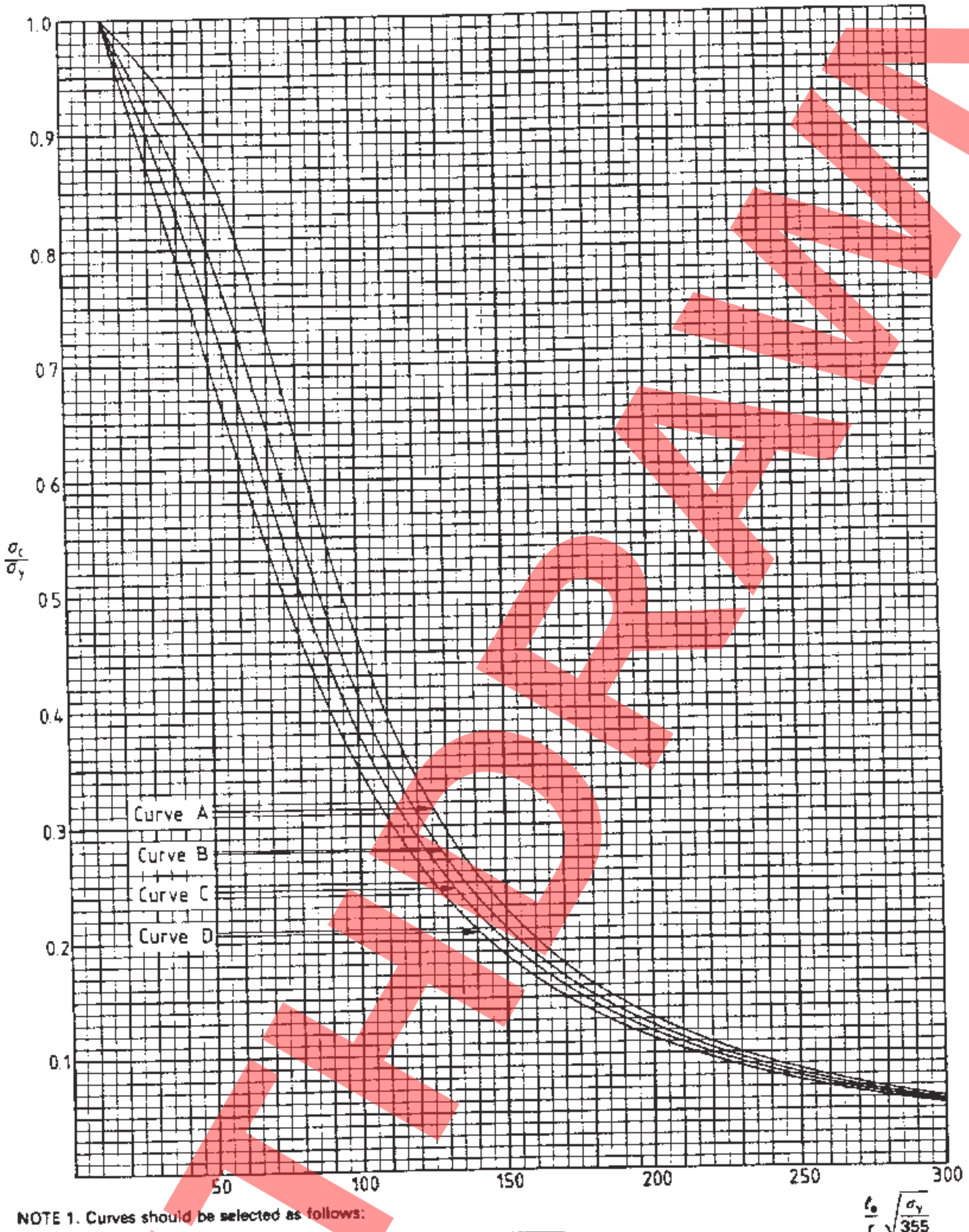

NOTE 1. Curves should be selected as follows:

	Members fabricated by welding (excluding local welding of battens, lacing, etc.)	All other members (including stress relieved welded members)
$r/y \geqslant 0.7$	Curve B	Curve A
$r/y = 0.60$	Curve C	Curve B
$r/y = 0.50$	Curve C	Curve B
$r/y \leqslant 0.45$	Curve C	Curve C
All rolled sections with flange thickness >40 mm	Curve D	
Hot finished hollow sections	Curve A	

NOTE 2. For intermediate values of r/y, linear interpolation may be used between the curves given.

NOTE 3. For the basis of curves, see G.16.

Figure 37. Ultimate compressive stress σ_c

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10.6.2 *Combined compression and bending*

10.6.2.1 *Strength.* A member subjected to coexistent compression and bending should be such that at all cross sections within the middle-third of the length of the member:

$$\frac{P_{max}}{P_D} + \frac{M_{x\,max}}{M_{Dxc}} + \frac{M_{y\,max}}{M_{Dyc}} \leqslant 1.0$$

where

P_{max}, $M_{x\,max}$, $M_{y\,max}$ are the maximum axial load, and bending moments about the X-X and Y-Y axes, respectively, (see figure 1), anywhere within the middle-third of the length of the member between points of restraint

P_D is as defined in **10.6.1**

M_{Dxc}, M_{Dyc} are the corresponding bending resistances of the member, with respect to the extreme compression fibres determined in accordance with **9.9.1**.

In addition, at all sections of the member, the maximum stress due to the applied load P and coexistent bending moments M_x and M_y should be such that:

$$\frac{P}{A_e} \pm \frac{M_x}{Z_x} \pm \frac{M_y}{Z_y} \leqslant \frac{\sigma_y}{\gamma_{m}\gamma_{f3}}$$

where

A_e is the effective area of the section, as defined in **10.5**

σ_y is the nominal yield stress of the material

Z_x and Z_y are the appropriate elastic moduli of the effective section derived in accordance with **9.4.2**.

10.6.2.2 *Eccentricity of end connections.* The bending moment resulting from any eccentricity of the end connections of the member or its components should be taken into account in determining the values of $M_{x\,max}$, $M_{y\,max}$, M_x and M_y referred to in **10.6.2.1**.

10.6.3 *Compact and stocky members.* As an alternative to the provisions of **10.6.2**, when a member is stocky and of compact cross section, the resistance in combined compression and bending may be determined on the basis of any assumed distribution of stress over the effective area of cross section, provided that the stresses so assumed are in equilibrium with the load effects and nowhere exceed:

$$\frac{\sigma_y}{\gamma_{m}\gamma_{f3}}$$

and provided that

$$\lambda_{LT} \leqslant 45\sqrt{\frac{355}{\sigma_y}}$$

where

σ_y is the nominal yield stress of the material

λ_{LT} is derived in accordance with **9.7**.

A member is defined as stocky if its slenderness ratio l_e/r does not exceed:

$$15\sqrt{\frac{355}{\sigma_y}}$$

where

l_e and r are as defined in **10.4** and **10.6.1.1**, respectively.

A cross section of a member is defined as compact if:

(a) $\frac{b}{t} \leqslant 24\sqrt{\frac{355}{\sigma_y}}$ for plates between supports

where

b and t are as defined in **10.5.2**:

(b) $\frac{b_o}{t_o} \leqslant 7\sqrt{\frac{355}{\sigma_y}}$ for outstands,

where

b_o, t_o are as defined in **10.3.1**;

(c) the outside diameter does not exceed:

$50\sqrt{\frac{355}{\sigma_y}} \times$ the wall thickness for circular hollow sections.

10.7 Compression members with longitudinal stiffeners

10.7.1 *Strength.* The stress in plate panels, longitudinally stiffened by discrete stiffeners (i.e. other than corner stiffeners) and forming walls of box-type compression members, should satisfy the provisions of **9.10.2.1** and the stresses at the centroids of the longitudinal stiffeners should satisfy the provisions of **9.10.2.3** when both these stresses are determined in accordance with 10.7.2. In determining the stresses at the centroid of the longitudinal stiffeners, the value of y_{Bs} should be appropriate to the combined stress diagram at the section considered, where y_{Bs} is as defined in **9.10.2.3**.

NOTE. When the maximum tensile stress intensity due to the bending moment is smaller than the compression stress intensity due to the axial load, the axis of zero stress will be outside the cross section of the member and in that case y_{Bs} may be greater than the depth of the member.

10.7.2 *Evaluation of stresses.* The stresses in the member should be evaluated for the following coexistent loads and moments:

(a) the applied axial load;

(b) the applied bending moments about the X-X and Y-Y axes each multiplied by a factor:

$$\frac{\sigma_E}{\sigma_E - \sigma_a}$$

where

σ_E is the Euler buckling stress of the member about the relevant axis given by $\pi^2 E r^2 / l_e{}^2$

l_e and r are as defined in **10.4** and **10.6.1**, respectively

σ_a is the axial stress based on the effective section of the member in accordance with 10.5;

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10.6.2.1 Strength

Add at end:

For assessment of the adequacy of a uniform member of I-section subject to combined bending and axial compression, the buckling criterion above shall be replaced by the alternative criterion given in **9.9.4.2.**

10.7.2 Evaluation of stresses

Add at end of (c):

Where in assessment of the adequacy of a compression member with longitudinal stiffeners allowance is to be made for measured initial departures from straightness, Δ_i shall be taken as:

Δ_i = 1.2 Δ_s determined separately for the X-X and Y-Y axes where Δ_s is the departure from straightness measured in accordance with BS5400: Part 6 over a gauge length G equal to the distance between appropriate points of restraint.

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(c) additional bending moments, acting about the X-X and Y-Y axes, respectively equal to:

$$P\Delta_i\left(\frac{\sigma_E}{\sigma_E - \sigma_a}\right)$$

where

P is the applied axial load

Δ_i $= \frac{1}{800}$ × (length of member between the appropriate points of restraint), determined separately for the X-X and Y-Y axes.

NOTE. Stresses due to (a) should be evaluated on the basis of the effective area determined in accordance with 10.5.2, and stresses due to (b) and (c) should be evaluated on the basis of the effective section determined in accordance with 9.4.2.

10.7.3 ***Shape of longitudinal stiffener.*** The shape of the longitudinal stiffeners should satisfy the provisions of 9.3.4.

10.7.4 ***Transverse stiffeners***

10.7.4.1 Transverse stiffeners may be assumed to combine with an effective width of the plating on each side not exceeding either:

(a) one-quarter of the stiffener spacing; or
(b) one-eighth of the length of the stiffener.

10.7.4.2 The effective section of the stiffener should satisfy both:

(a) the stiffness provision of 9.15.3; and

(b) the strength provision of 9.15.5 under the action of a uniformly distributed load equal to:

$\frac{1}{300}$ × (longitudinal compressive force in the stiffened panel at the cross section under consideration due to the loads and moments given in 10.7.2).

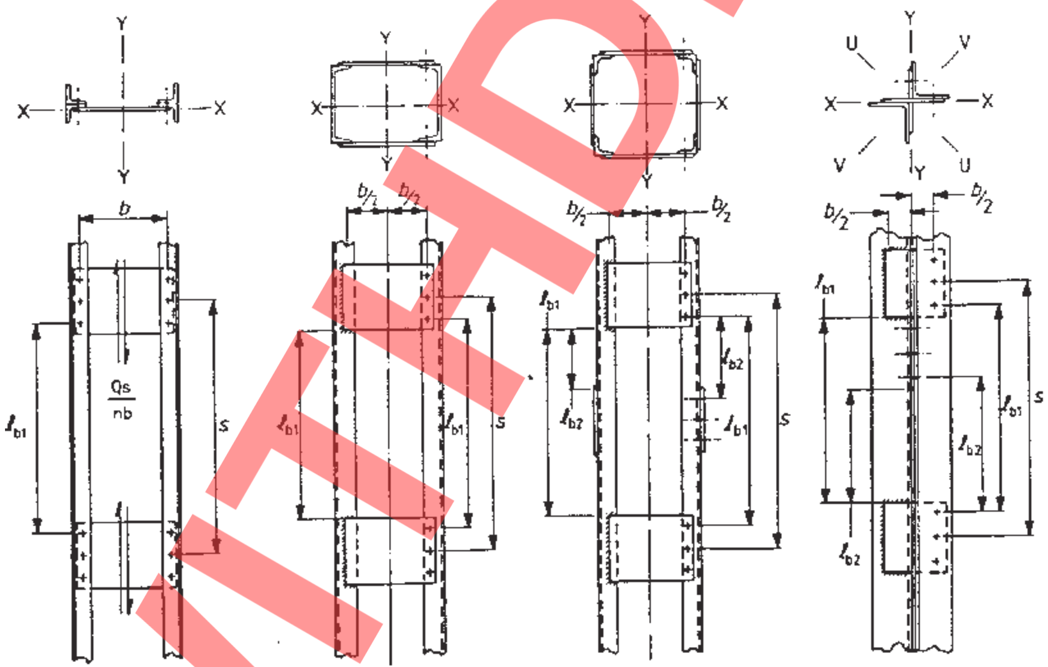
10.8 Battened compression members

10.8.1 ***General.*** A compression member consisting of two or more main components may have battens connecting the components, either in one plane, or in two or more parallel planes, or in two perpendicular planes or sets of parallel planes, as shown in figure 38.

The strength of the individual components, including battens, and their connections should be in accordance with the provisions of 9.1 to 9.9, 10.1 to 10.7, 11.1 to 11.5 and clause 14, as appropriate.

10.8.2 ***Radius of gyration of the member.*** The radius of gyration of the member about the Y-Y axis, in the case of a single plane of battens or of parallel battens (see figure 38), and about any axis in the case of a member with battens in planes at right angles, or of cruciform section, should be taken as 0.9 times the actual radius of gyration.

10.8.3 ***Spacing of battens.*** Battens should generally be spaced uniformly throughout the length of the member, except as required for intermediate restraint in accordance with 10.8.5.1(c).



NOTE. Pairs of battens are shown staggered to illustrate l_{b1} and l_{b2}. Normally all four battens should be at the same cross section.

Figure 38. Battened members

10.7.3 Limitations of stiffener shape

Add at end:

Stiffener shapes not complying with **9.3.4** shall be assessed in accordance with **9.3.1** and Appendix S. Stiffeners of shapes other than those specified shall be assessed on the basis of the nearest standard shape.

10.8.1 General

Add at end:

For assessment, where the arrangements of the member do not comply with any of the above requirements, the strengths of the battens and of the battened member shall be assessed in accordance with Clauses **10.8.5.3** and **l0.8.5.4** respectively.

10.8.2 Radius of gyration of the member

Add at end:

For assessment, where the battened member does not comply with the requirements of **10.8.1**, the radius of gyration of the member shall be taken as $\sqrt{\phi}$ times the actual radius of gyration where ϕ is as defined in **10.8.5.4**.

10.8.3 Spacing of battens

Add at end:

When the spacings of the battens exceed the limits derived from the above requirements, the strengths of the battened member shall be assessed as described in **l0.8.5.4.**

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If the member as a whole is such that $\ell_y/r_y \leqslant 0.8(\ell_x/r_x)$, the spacing of battens in a plane parallel to the X-X axis should be such that each main component of the member satisfies the following:

$$\frac{\ell_{b1}}{r_{b1}} \leqslant 0.7\lambda_{max}, \text{ and } \frac{\ell_{b2}}{r_{b2}} \leqslant 0.7\lambda_{max},$$

where

l_x/r_x and l_y/r_y are the slenderness ratios of the member about its X-X and Y-Y axes, respectively (as shown in figure 38); r_x and r_y should be calculated on the basis of the gross cross-section of the member

ℓ_{b1} is the distance between end fastenings of successive battens in planes parallel to the X-X axis

ℓ_{b2} is the distance between end fastenings of successive battens in any plane

r_{b1} is the radius of gyration of the gross cross-section of the component about an axis, parallel to the Y-Y axis, through the centroid of the component

r_{b2} is the least radius of gyration of the gross cross-section of the component

NOTE. For determining r_{b1} and r_{b2} in the case of partially enclosed sections, as shown in figure 35(b), the component should be considered to consist of the angle or outstanding plate to which the batten is attached together with half of the depth of the web between the batten and the flange plate parallel thereto.

λ_{max} is the largest value of the slenderness ratio ℓ/r with which the design resistance of the member would be sufficient to resist the applied load (see **10.6.1**).

If the member as a whole is such that $\ell_y/r_y > 0.8(\ell_x/r_x)$, the spacing of battens in a plane parallel to the X-X axis should be such that each main component of the member satisfies the following:

$$\frac{\ell_{b1}}{r_{b1}} \leqslant 0.6\lambda_{max}, \text{ and } \frac{\ell_{b2}}{r_{b2}} \leqslant 0.6\lambda_{max}$$

The spacing of battens (if any) in a plane perpendicular to the X-X axis should be determined similarly by transposing the axes.

10.8.4 ***Dimensions of battens***

10.8.4.1 *Length.* The length of each batten, measured between end fastenings in a direction parallel to the axis of the member, should not be less than three-quarters of the distance between the centroids of adjacent main components.

Furthermore, the length of each batten, measured as in **10.8.4.1**, should not be less than twice the width of the smallest main component measured parallel to the plane of battens.

10.8.4.2 *Thickness.* The thickness of each batten should not be less than one-fiftieth of the distance between the innermost lines of fastenings, except that, where both transverse edges of a batten are effectively stiffened by stiffeners having a slenderness ratio not exceeding 170, the thickness of the batten need not exceed 8 mm.

10.8.5 ***Members with single or parallel planes of battens***

10.8.5.1 *Arrangement of battens.* In any battened member, other than a member of cruciform section, battens should be placed in each battened plane as follows:

(a) at each end of the member;

(b) at not less than two intermediate positions, inclusive of any positions where battens are provided under (c);

(c) at each intermediate point, if any, where in the plane being considered, the member is provided with restraint against lateral displacement or has another member connected to it.

Battens should be placed opposite one another wherever there are two or more planes of parallel battens.

10.8.5.2 *Loads and moments on battens.* Each batten, and its fixings to the main components of the member, should be proportioned to resist, simultaneously:

(a) a longitudinal shear force equal to Qs/nb (see figure 38);

(b) a bending moment, acting in the plane of the batten, equal to $Qs/2n$;

(c) the effects of any external transverse loads on the member;

where

Q is a transverse shear force acting parallel to the plane or planes of the battens, which should be taken as:

(1) $Q = \dfrac{PP_{Ey}}{200(P_{Ey} - P)}$

for battens in a plane parallel to the X-X axis

(2) $Q = \dfrac{PP_{Ex}}{200(P_{Ex} - P)}$

for battens in a plane parallel to the Y-Y axis

P is the axial force in the member

$$P_{Ey} = \frac{\pi^2 E A_e}{(\ell_y/r_y)^2}$$

$$P_{Ex} = \frac{\pi^2 E A_e}{(\ell_x/r_x)^2}$$

ℓ_x/r_x and ℓ_y/r_y are as defined in **10.8.3**

A_e is the effective area of the whole member determined in accordance with **10.5**

In using figure 38,

s is the longitudinal spacing of battens measured between centres

n is the number of parallel planes of battens

b is the lateral distance between centroids of fastenings to the components.

10.8.6 ***Cruciform members***

10.8.6.1 *Arrangement of battens.* Battens in cruciform members should either be placed in pairs in two

10.8.4.1 Length.

Add at end:

Where the length of each batten is less than specified above, the strength of the battened member shall be assessed in accordance with **10.8.5.4**.

10.8.4.2 Thickness.

Add at end:

Where the thickness of any batten is less than that specified above the adequacy of such batten shall be assessed in accordance with **10.8.5.3**.

10.8.5.1 Arrangement of battens.

Add at end:

Where the arrangement of battens does not comply with the recommendations above the battened compression member shall be assessed in accordance with **10.8.5.4**.

10.8.5.2 Loads and moments on battens.

Add at end:

For assessment, (a) and (b) shall be modified to read:

(a) a longitudinal shear force equal to $K_b Qs/nb$

(b) a bending moment, acting in the plane of the batten, equal to $K_b Qs/2n$

where

K_b = 1/2 for end battens,
for intermediate battens

$$= \tfrac{1}{2}\left(\cos\frac{\pi}{l}x_1 + \cos\frac{\pi}{l}x_2\right)$$

l is the overall length of the battened member;

x_1, x_2 are the respective distances from one end of the member to points a distance s/2 either side of the centre line of the batten under consideration.

For members with battens and their arrangements not complying with the limits given **10.8.1**, **10.8.4** or **10.8.5.1**, the lowest values of the elastic critical buckling loads P'_{EY} and P'_{EX} shall be determined as described in **10.8.5.4** and shall be used instead of P_{EY} and P_{EX}.

Where in assessment of the adequacy of a battened member, account is to be taken of measured departure from straightness exceeding that permitted by BS5400: Part 6, the number 200 in the denominator of equations (1) and (2) above shall be reduced to

$$1/\left(3.8\frac{\Delta_s}{l_e} + \frac{1}{815}\right)$$

where

Δ_s is the departure from straightness measured over a gauge length equal to l_e.

Add new Clause 10.8.5.3:

10.8.5.3 Strength assessment of non-complying battens

Where the arrangements and sizes of the battened member do not comply with the requirements **10.8.1**, **10.8.4** or **10.8.5.1**, the battens shall be of such sizes that:

(a) the maximum bending stress does not exceed

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

(b) the maximum average shear stress =

$$\frac{\textit{longitudinal shear force}}{A_{bnet}}$$

does not exceed

$$\frac{\sigma_y}{1.5\sqrt{3}\,\gamma_m \gamma_{f3}}$$

where A_{bnet} = Net cross sectional area of the batten;

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If the member as a whole is such that $\ell_y/r_y \leqslant 0.8(\ell_x/r_x)$, the spacing of battens in a plane parallel to the X-X axis should be such that each main component of the member satisfies the following:

$$\frac{\ell_{b1}}{r_{b1}} \leqslant 0.7\lambda_{max}, \text{ and } \frac{\ell_{b2}}{r_{b2}} \leqslant 0.7\lambda_{max},$$

where

l_x/r_x and l_y/r_y are the slenderness ratios of the member about its X-X and Y-Y axes, respectively (as shown in figure 38); r_x and r_y should be calculated on the basis of the gross cross-section of the member

ℓ_{b1} is the distance between end fastenings of successive battens in planes parallel to the X-X axis

ℓ_{b2} is the distance between end fastenings of successive battens in any plane

r_{b1} is the radius of gyration of the gross cross-section of the component about an axis, parallel to the Y-Y axis, through the centroid of the component

r_{b2} is the least radius of gyration of the gross cross-section of the component

NOTE. For determining r_{b1} and r_{b2} in the case of partially enclosed sections, as shown in figure 35(b), the component should be considered to consist of the angle or outstanding plate to which the batten is attached together with half of the depth of the web between the batten and the flange plate parallel thereto.

λ_{max} is the largest value of the slenderness ratio ℓ/r with which the design resistance of the member would be sufficient to resist the applied load (see 10.6.1).

If the member as a whole is such that $\ell_y/r_y > 0.8(\ell_x/r_x)$, the spacing of battens in a plane parallel to the X-X axis should be such that each main component of the member satisfies the following:

$$\frac{\ell_{b1}}{r_{b1}} \leqslant 0.6\lambda_{max}, \text{ and } \frac{\ell_{b2}}{r_{b2}} \leqslant 0.6\lambda_{max}$$

The spacing of battens (if any) in a plane perpendicular to the X-X axis should be determined similarly by transposing the axes.

10.8.4 *Dimensions of battens*

10.8.4.1 *Length.* The length of each batten, measured between end fastenings in a direction parallel to the axis of the member, should not be less than three-quarters of the distance between the centroids of adjacent main components.

Furthermore, the length of each batten, measured as in 10.8.4.1, should not be less than twice the width of the smallest main component measured parallel to the plane of battens.

10.8.4.2 *Thickness.* The thickness of each batten should not be less than one-fiftieth of the distance between the innermost lines of fastenings, except that, where both transverse edges of a batten are effectively stiffened by stiffeners having a slenderness ratio not exceeding 170, the thickness of the batten need not exceed 8 mm.

10.8.5 *Members with single or parallel planes of battens*

10.8.5.1 *Arrangement of battens.* In any battened member, other than a member of cruciform section, battens should be placed in each battened plane as follows:

(a) at each end of the member;
(b) at not less than two intermediate positions, inclusive of any positions where battens are provided under (c);
(c) at each intermediate point, if any, where in the plane being considered, the member is provided with restraint against lateral displacement or has another member connected to it.

Battens should be placed opposite one another wherever there are two or more planes of parallel battens.

10.8.5.2 *Loads and moments on battens.* Each batten, and its fixings to the main components of the member, should be proportioned to resist, simultaneously:

(a) a longitudinal shear force equal to Qs/nb (see figure 38);
(b) a bending moment, acting in the plane of the batten, equal to $Qs/2n$;
(c) the effects of any external transverse loads on the member;

where

Q is a transverse shear force acting parallel to the plane or planes of the battens, which should be taken as:

(1) $Q = \dfrac{PP_{Ey}}{200(P_{Ey} - P)}$

for battens in a plane parallel to the X-X axis

(2) $Q = \dfrac{PP_{Ex}}{200(P_{Ex} - P)}$

for battens in a plane parallel to the Y-Y axis

P is the axial force in the member

$$P_{Ey} = \frac{\pi^2 EA_e}{(\ell_y/r_y)^2}$$

$$P_{Ex} = \frac{\pi^2 EA_e}{(\ell_x/r_x)^2}$$

ℓ_x/r_x and ℓ_y/r_y are as defined in 10.8.3

A_e is the effective area of the whole member determined in accordance with 10.5

In using figure 38,

s is the longitudinal spacing of battens measured between centres

n is the number of parallel planes of battens

b is the lateral distance between centroids of fastenings to the components.

10.8.6 *Cruciform members*

10.8.6.1 *Arrangement of battens.* Battens in cruciform members should either be placed in pairs in two

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or the average maximum shear stress does not exceed

$$\frac{K}{\gamma_m \gamma_{f3}} \left(\frac{t_b}{d_b}\right)^2 N / mm^2$$

whichever is the lesser
where

K is obtained from Table 10.8A
t_b is the thickness of the batten
d_b is the depth of the batten in the direction parallel to the axis of the member
b is as defined in **10.8.5.2.**

Table 10.8A

d_b/b	1.5	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2
K ($x10^4$)	87.3	36.8	32.2	25.1	19.1	14.1	9.9	6.7	4.2	2.3	1.0

Add new clause 10.8.5.4:

10.8.5.4 Strength assessment of non-complying battened members

Where the arrangements of the battened member do not comply with the requirements of **10.8.1**, **10.8.4** or **10.8.5.1** the compressive strength of the battened member shall be calculated in accordance with Clauses **9.1** to **9.9, 10.1** to **10.7** using effective radii of gyration defined in **10.8.2.**

The lowest values of the elastic critical buckling loads P^1_{EY} and P^I_{EX} shall be taken as K times the critical loads determined by non-linear buckling analysis of the battened member.

where

K = 1 for welded or friction grip bolted battens;
= 0.7 for riveted or black bolted connections

Alternatively, where the arrangement of battens complies with the requiremetns of **10.8.5.1** and battens are equally spaced, P^1_{EY} and P^I_{EX} may be determined from Appendix M of the accompanying Advice Note.

The factor √ϕ should be taken as

$$\sqrt{\frac{P^{\,1}_{EY}}{P_{EY}}} \ \ or \ \ \sqrt{\frac{P^{\,1}_{EX}}{P_{EX}}} \qquad as\ appropriate$$

The adequacy of each main component of the battened member shall be checked assuming it to resist, in addition to the axial force, a bending moment about each of the X-X and Y-Y axes equal to Qs/4 together with the effects of transverse external forces, if any and assuming its effective length to be equal to l_{b1} where Q is as defined in **10.8.5.2** and l_{b1}, is as defined in **10.8.3**.

NOTE: Members with planes of battens in opposite faces in which the centres of the battens are staggered may conservatively be treated as if the battens were not staggered.

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perpendicular planes in contact at a transverse edge and fixed successively to the two legs of both angles, or be of cruciform section and fixed to both legs of both angles.

A pair of battens, or a cruciform batten, should be placed:

(a) at each end of the member;

(b) at not less than two intermediate positions, inclusive of any positions where battens are provided under (c);

(c) at each intermediate point, if any, where the member is provided with restraint against lateral displacement or has another member connected to it.

10.8.6.2 *Loads and moments on battens*. Each batten, and its connections to the main components, should be proportioned to resist simultaneously:

(a) a longitudinal shear force equal to $\frac{Q_{\mathrm{r}}s}{b}$

(b) a bending moment, acting in the plane of the batten, equal to $\frac{Q_{\mathrm{r}}s}{2}$

where

$Q_{\mathrm{r}} = Q$ if the largest slenderness ratio ℓ/r of the member as a whole occurs about the X-X or Y-Y axis

$= Q/\sqrt{2}$ if the largest slenderness ratio ℓ/r of the member as a whole occurs about a diagonal axis V-V

Q, s and b are as defined in **10.8.5.2**;

(c) the effects of any transverse external loads on the member.

10.8.6.3 *Strength of components of the member*. Each main component of a cruciform member should be designed to resist, in addition to the axial force, a bending moment about each of the X-X and Y-Y axes equal to $\frac{Q_{\mathrm{r}}s}{4}$, together with the effects of transverse external forces, if any,

where

s is as defined in **10.8.5.2**

Q_{r} is as defined in **10.8.6.2**.

10.8.7 ***Welding of battens***

10.8.7.1 The aggregate length of weld connecting each longitudinal edge of a batten to a main component of a member should not be less than half the length of the batten. At least one-third of the longitudinal weld should be placed at each end of the edge of the batten. A further length of weld, equal to at least four times the thickness of the batten, should be returned along the end of the batten from each longitudinal edge.

10.8.7.2 Where batten plates are fitted between main components they should be connected to each component either by fillet welds on each side of the plate, at least equal in length to that given in **10.8.7.1**, or by complete penetration butt welds along the whole length of the plate.

10.9 Laced compression members

10.9.1 ***General***. A compression member consisting of two or more main components may have lacing connecting the components, either in one plane, or in two or more parallel planes, or in two perpendicular sets of parallel planes.

The lacing should form a fully triangulated system and should be uniform throughout the length of the member.

A laced compression member should be provided with a batten, in accordance with **10.8.4**, in each plane of lacing, at each end of the member, and at each point where the system of lacing is interrupted or where another member is connected to the laced member. These battens should be designed to resist the forces stated in **10.8.5.2**.

The strength of individual components, including lacings and their connections, should be in accordance with **10.1** to **10.7**, **11.1** to **11.5** and clause **14**, as appropriate.

10.9.2 ***Inclination of lacing bars***. If a single system of lacing bars is used, the bars should be inclined at an angle between 50° and 70° to the axis of the member; if a double system of intersecting bars is used, the bars should be inclined at an angle between 40° and 50°.

10.9.3 ***Spacing of lacing bars***. The spacing of lacing bars should be such that each main component of the member satisfies the following:

$$\frac{\ell_{\mathrm{p1}}}{r_{\mathrm{p1}}} \leqslant 0.7\lambda_{\max}, \text{ and } \frac{\ell_{\mathrm{p2}}}{r_{\mathrm{p2}}} \leqslant 0.7\lambda_{\max},$$

where

ℓ_{p1} is the distance between the centroids of successive end fastenings of lacing bars in one plane

ℓ_{p2} is the distance between the centroids of successive end fastenings of lacing bars in any plane

r_{p1} is the radius of gyration of a main component of the member about an axis parallel to the plane of lacing based on the gross cross-section of the member

r_{p2} is the least radius of gyration of a main component of the member based on the gross cross-section of the member

$\lambda_{\max}$ is as defined in **10.8.3**.

10.9.4 ***Slenderness of lacing bars***. In a single system the effective length of a lacing bar should be taken as the clear length along the bar between innermost fixings to the main components of the member, and, in a double intersecting system, 0.7 of this clear length.

10.9.5 ***Loads on lacing***. Lacing bars and their fixings should be designed to resist, at any point along the length of the member, a transverse shear force Q, as defined in **10.8.5.2** for battened members, together with the effects of any external transverse loads on the member. The shear force Q should be considered as divided equally between all the systems of lacing or plates connecting the components in the appropriate parallel planes.

10.9.6 ***Double lacing***. In a double system of intersecting lacing bars the effects of axial deformation of the member on the lacing bars and their connections should be considered.

Except for battens, in accordance with **10.9.1**, a double system of intersecting lacing bars should not be combined with diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for.

10.9.7 ***Welding of lacing bars to main components***. Where a lacing bar to be connected by welding is lapped on to a main component of a member, the length of lap, measured along the centreline of the lacing bar, should not be less than four times the thickness of the bar, or four times the mean thickness of the flange of the main

10.8.6.2 Loads and moments on battens

Add at end:

For assessment (a) and (b) shall alternatively be modified to read:

(a) a longitudinal shear force equal to $K_b Q_r$,s/b

(b) a bending moment acting in the plane of the batten equal to $K_b Q_r$,s/b

where K_b is as defined in **10.8.5.2**

10.9.1 General

In the fourth paragraph after 'The strength', *insert* 'of a member as a whole and'.

10.9.2 Inclination of lacing bars

Add at end:

For assessment of a laced member having lacing bars not complying with the above limits to inclination the critical buckling loads and strength of the whole member shall be determined as follows:

The critical loads for buckling about the Y-Y or X-X axes respectively may be taken as:

$P^1_{EY} = \phi P_{EY}$, $P^1_{EX} = \phi P_{EX}$ where ϕ may be derived from:

$$\phi = \left\{ 1 + \pi^2 \frac{A_e r^2}{l^2} \left[\frac{1}{A_l \, Cos\, \theta \, Sin^2\, \theta} \right] \right\}^{-1}$$

where

- A_l is the total cross sectional area of lacings within a laced panel in the appropriate plane of bracings;
- θ is angle of inclination of the lacings to the axis of the member;
- l $= l_x$ or l_y as appropriate;
- r = radius of gyration of the member as a whole about the X-X or Y-Y axes as appropriate;
- A_e is the effective area of the whole member determined in accordance with **10.5**;
- P_{EY}, P_{EX} are as defined in **10.8.5.2**.

The strength of the member as a whole shall be determined in accordance with **10.9.1** with the radius of gyration about the appropriate axis taken as $\sqrt{\phi}$ times the actual radius of gyration using the value of ϕ appropriate to the axis considered.

10.9.3 Spacing of lacing bars

Add at end:

For assessment where the spacing of lacing bars does not comply with these requirements, the main components of the member shall comply with the following requirement:

$$\frac{P}{A_e} + \frac{M_x}{Z_x}\left(\frac{1}{1 - \frac{P}{P^1_{EX}}}\right) + \frac{M_y}{Z_y}\left(\frac{1}{1 - \frac{P}{P^1_{EY}}}\right) \le \frac{\sigma_c}{\gamma_m \, \gamma_{f3}}$$

where

- A_e is the effective area of cross section of the laced member (see Clause **10.5.2.1**);
- P is the axial load applied to the laced member;
- Z_x and Z_y are the section moduli of the laced member about the X-X and Y-Y axes respectively related to the centroid of the main component considered;
- P^1_{EX}, P^1_{EY} are as defined in **10.9.2**.
- M_x $= M_{ox} + 1.2\, P\Delta_x$;
- M_y $= M_{oy} + 1.2\, P\Delta_y$;
- M_{ox}, M_{oy} are any applied bending moments about the X-X and Y-Y axes respectively in the plane of the lacing including that due to eccentricity of axial load to the centroid of the laced member;
- Δx, Δy are the maximum departures from straightness of the laced member in the directions normal to the X-X and Y-Y axes respectively measured in the plane of the lacings over a length between points of effective lateral restraint to the laced member in the relevant direction;

perpendicular planes in contact at a transverse edge and fixed successively to the two legs of both angles, or be of cruciform section and fixed to both legs of both angles.

A pair of battens, or a cruciform batten, should be placed:

(a) at each end of the member;
(b) at not less than two intermediate positions, inclusive of any positions where battens are provided under (c);
(c) at each intermediate point, if any, where the member is provided with restraint against lateral displacement or has another member connected to it.

10.8.6.2 *Loads and moments on battens*. Each batten, and its connections to the main components, should be proportioned to resist simultaneously:

(a) a longitudinal shear force equal to $\frac{Q_t s}{b}$

(b) a bending moment, acting in the plane of the batten, equal to $\frac{Q_t s}{2}$

where

$Q_t = Q$ if the largest slenderness ratio ℓ/r of the member as a whole occurs about the X-X or Y-Y axis
$= Q/\sqrt{2}$ if the largest slenderness ratio ℓ/r of the member as a whole occurs about a diagonal axis V-V

Q, s and b are as defined in **10.8.5.2**;

(c) the effects of any transverse external loads on the member.

10.8.6.3 *Strength of components of the member*. Each main component of a cruciform member should be designed to resist, in addition to the axial force, a bending moment about each of the X-X and Y-Y axes equal to $\frac{Q_t s}{4}$, together with the effects of transverse external forces, if any,

where

s is as defined in **10.8.5.2**
Q_t is as defined in **10.8.6.2**.

10.8.7 *Welding of battens*

10.8.7.1 The aggregate length of weld connecting each longitudinal edge of a batten to a main component of a member should not be less than half the length of the batten. At least one-third of the longitudinal weld should be placed at each end of the edge of the batten. A further length of weld, equal to at least four times the thickness of the batten, should be returned along the end of the batten from each longitudinal edge.

10.8.7.2 Where batten plates are fitted between main components they should be connected to each component either by fillet welds on each side of the plate, at least equal in length to that given in **10.8.7.1**, or by complete penetration butt welds along the whole length of the plate.

10.9 Laced compression members

10.9.1 ***General.*** A compression member consisting of two or more main components may have lacing connecting the components, either in one plane, or in two or more parallel planes, or in two perpendicular sets of parallel planes.

The lacing should form a fully triangulated system and should be uniform throughout the length of the member.

A laced compression member should be provided with a batten, in accordance with **10.8.4**, in each plane of lacing, at each end of the member, and at each point where the system of lacing is interrupted or where another member is connected to the laced member. These battens should be designed to resist the forces stated in **10.8.5.2**.

The strength of individual components, including lacings and their connections, should be in accordance with **10.1** to **10.7**, **11.1** to **11.5** and clause **14**, as appropriate.

10.9.2 *Inclination of lacing bars.* If a single system of lacing bars is used, the bars should be inclined at an angle between 50° and 70° to the axis of the member; if a double system of intersecting bars is used, the bars should be inclined at an angle between 40° and 50°.

10.9.3 *Spacing of lacing bars.* The spacing of lacing bars should be such that each main component of the member satisfies the following:

$$\frac{\ell_{p1}}{r_{p1}} \leqslant 0.7\lambda_{max}, \text{ and } \frac{\ell_{p2}}{r_{p2}} \leqslant 0.7\lambda_{max},$$

where

ℓ_{p1} is the distance between the centroids of successive end fastenings of lacing bars in one plane
ℓ_{p2} is the distance between the centroids of successive end fastenings of lacing bars in any plane
r_{p1} is the radius of gyration of a main component of the member about an axis parallel to the plane of lacing based on the gross cross-section of the member
r_{p2} is the least radius of gyration of a main component of the member based on the gross cross-section of the member
λ_{max} is as defined in **10.8.3**.

10.9.4 *Slenderness of lacing bars.* In a single system the effective length of a lacing bar should be taken as the clear length along the bar between innermost fixings to the main components of the member, and, in a double intersecting system, 0.7 of this clear length.

10.9.5 *Loads on lacing.* Lacing bars and their fixings should be designed to resist, at any point along the length of the member, a transverse shear force Q, as defined in **10.8.5.2** for battened members, together with the effects of any external transverse loads on the member. The shear force Q should be considered as divided equally between all the systems of lacing or plates connecting the components in the appropriate parallel planes.

10.9.6 *Double lacing.* In a double system of intersecting lacing bars the effects of axial deformation of the member on the lacing bars and their connections should be considered.

Except for battens, in accordance with **10.9.1**, a double system of intersecting lacing bars should not be combined with diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for.

10.9.7 *Welding of lacing bars to main components.* Where a lacing bar to be connected by welding is lapped on to a main component of a member, the length of lap, measured along the centreline of the lacing bar, should not be less than four times the thickness of the bar, or four times the mean thickness of the flange of the main

σ_c is the ultimate compressive stress for buckling of the main component about its centroidal axis perpendicular to the plane of lacing obtained from σ_c/σ_y in accordance with Figure 37 using l_e equal to the spacing of the lacing bar intersections along the component;

r is the least radius of gyration of the section of the main component;

y is the distance from the axis of least radius of gyration to the extreme fibre of the section of the main component;

σ_y is the nominal yield stress of the material.

component to which it is attached, whichever is less. The bar should be welded along the whole length of lap on both sides of the bar, and the weld should be returned across the end of the bar for an aggregate distance of not less than the width of the bar, or four times its thickness, whichever is less.

Where a welded lacing bar is fitted between main components of a member, it should be attached to each component either by welding all round, or by a full penetration butt weld.

10.10 Compression members connected by perforated plates

10.10.1 ***General.*** A compression member consisting of two or more main components may have continuous perforated plates connecting the components, either in one plane, or in two or more parallel planes, or in two perpendicular sets of parallel planes. The thickness of a perforated plate should not be less than one-fiftieth of the unsupported distance between innermost attachments to the main components.

The overall length of a perforation, measured in the direction of stress, should not be more than twice its width. Each end of a perforation should be rounded.

The clear distance between perforations, and the clear length beyond the perforation at each end of the member, should not be less than three-quarters of the unsupported distance between the innermost attachments to the main components.

The unsupported width of a plate at a perforated section, between the inner edge of the perforation and the nearest attachment to a main component, should be in accordance with **10.3.1**.

10.10.2 ***Strength of member.*** The net section of a perforated plate may be included as part of the effective section of the member when computing the strength of the member in accordance with **10.1** to **10.7**.

10.10.3 ***Loads on perforated plates.*** Perforated plates and their fixings should be designed to resist, at any point along the length of the member, a transverse shear force Q, as defined in **10.8.5.2**, together with the effects of any transverse external loads. The shear force Q should be considered as divided equally between all the parallel perforated and other plates connecting the components of the member.

10.11 Compression members with components back to back

10.11.1 ***General.*** A compression member may consist of two angles, channels or tees, connected together, back to back, either in contact or separated by packs or washers. The components should preferably be in contact: when not in contact, either adequate space between the components should be provided in accordance with **4.5.4**, or the thickness should be increased to meet the provisions of **4.5.5**. In no case should the clear distance between the components exceed 50 mm.

Members with components not in contact should not be used to resist loads or moments applied in a plane perpendicular to the connected faces.

10.11.2 ***Slenderness of components.*** The components should be connected together so that:

$$\frac{l_p}{r_p} \leqslant 0.5\lambda_{max}$$

where

l_p is the distance between the centroids of successive connections

r_p is the least radius of gyration of the unsupported length of a component of the member between successive connections based on the gross cross-section of the member

λ_{max} is as defined in **10.8.3**.

10.11.3 ***Connections between components***

Connections between components should be spaced so as to divide the overall length of the member into at least three approximately equal parts.

Where these connections are made by welding, solid packings should be used to effect the jointing unless the components are sufficiently close together to permit welding along both pairs of edges.

Where the components are separated, connections made by bolts or rivets should pass through solid washers or packs. At least two connectors should be provided side by side transversely at each connected point if the width of connected face is more than 130 mm in the case of angles or tees, or more than 150 mm in the case of channels.

Connections should be designed to resist the shear force Q, given for battens in **10.8.5.2**, and the effects of any transverse external loads, and should be designed in accordance with clause **14**.

11. Design of tension members

11.1 General. This clause covers the design of straight members subjected to axial tension or to combined tension and bending.

11.2 Limit state

11.2.1 ***Ultimate limit state.*** Tension members should be designed to satisfy the provisions of clause **11** for the ultimate limit state.

11.2.2 ***Fatigue.*** The fatigue endurance should be in accordance with the recommendations of Part 10.

11.2.3 ***Serviceability limit state.*** The serviceability limit state need not be considered.

11.3 Effective section

11.3.1 ***General.*** In determining the effective section of a member, consideration should be given to the adequacy of the end fixings to distribute the load effects into all parts of the section.

11.3.2 ***Effective area.*** The effective section A_e should be taken as:

$$A_e = k_1 k_2 A_1 \text{ but } \leqslant A$$

where

k_1 = 1.0 except at a section through a pin hole when it should be taken as 0.65

k_2 = 1.2 where the member is of grade 43 steel, or
= 1.1 where the member is of grade 50 steel or of steel not complying with the requirements of BS 4360, for which $\sigma_y \leqslant 355$ N/mm², or
= 1.0 where the member is of grade 55 steel or of steel not complying with the requirements of BS 4360, for which $\sigma_y > 355$ N/mm²

11.1 Tension Members - General

Add at end:

This section shall cover the assessment of nominally straight members subjected to axial tension or to combined tension and bending. Where members act as compression members under defined assessment loading then they shall be assessed under section 10 unless it can be shown that sufficient redundancy or alterative load path exists in which case such compression may be ignored.

11.3.2 Effective area

Add at end:

For assessment the value of k_2 shall be taken as follows: -

1.2 where the member is B.S. 4360 grade 43 or B.S. 15 steel;

1.1 where the member is B.S. 4360 grade 50 or B.S. 968 steel;

1.0 where the member is B.S. 4360 grade 55 or Thirty Oak steel;

or

$$1.0 + 0.5\left\{\frac{\sigma_{ULT}}{\sigma_y} - 1.2\right\}$$

but not exceeding 1.2 where the member is of steel not complying with BS4360, BS15, BS548 or BS968 where σ_y and σ_{ULT} are the nominal yield stress and ultimate stress derived in accordance with **6.2** and **6.3** respectively.

A_t is the net cross-sectional area of the member or part given in **11.3.3**

A is the gross cross-sectional area of the member or part.

11.3.3 ***Net area of members with bolt or rivet holes.*** The net cross-sectional area A_t of a member or of any of its components should be taken as the lesser of either the gross cross-sectional area A minus the area of all holes (including all plug holes and countersunk heads) in a section perpendicular to the direction of primary stress, or the least net cross-sectional area of any diagonal or zig-zag section through a chain of holes, taken as:

$$A_t = A - \Sigma A_h + \Sigma \frac{s^2 t}{4g} \text{ (see figure 39)}$$

where

ΣA_h is the sum of the cross-sectional area of the holes lying on the zig-zag or diagonal section, including any plug-holes

s is the spacing of consecutive holes measured parallel to the direction of primary stress in the member

g is the spacing of the same holes measured at right angles to the direction of primary stress in the member. In an angle or similar part having holes in more than one plane, g should be measured along the centre of the thickness of the part

t is the thickness of the part. When the thickness varies between consecutive holes, as around the heel of a channel section, the mean thickness should be taken.

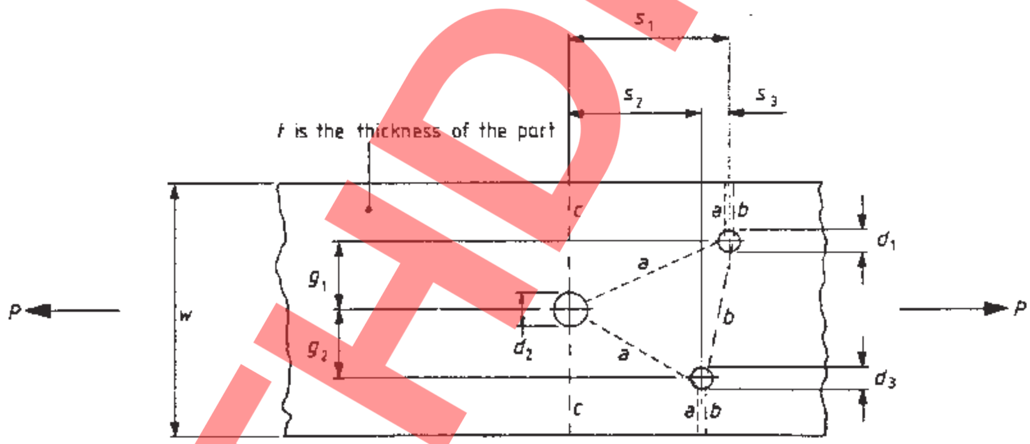
NOTE. Where the critical chain of holes in a component does not coincide with that for the member as a whole, the load resisted by the fasteners joining the components between the two critical chains of holes should be taken into account in determining the strength of the member.

11.3.4 ***Screwed rods.*** The net sectional area of the screwed rod should be taken as either the area at the root of the thread, or the tensile stress area given in BS 3692 or BS 4190 or BS 4395, as appropriate.

11.3.5 ***Pin-connected members.*** In a pin-connected member, the net area of the longitudinal section beyond the pin-hole, parallel to the axis of the member, should not be less than the required net cross-sectional area of the member (see figure 40).

11.4 Thickness at pin-holes. If the edges of the member are unstiffened, the thickness at a pin-hole of a member or part should not be less than:

one-sixteenth × (net width at pin-hole perpendicular to axis of member) (see figure 40).



NOTE. The net area A_t of the part is the least of the areas of sections a, b and c as follows:

a - - - - - - - - $A_t = t\left[w - (d_1 + d_2 + d_3) + 0.25\left(\frac{S_1^2}{g_1} + \frac{S_2^2}{g_2}\right)\right]$, considering all three holes

b — — — $A_t = t\left[w - (d_1 + d_3) + 0.25\left(\frac{S_3^2}{g_1 + g_2}\right)\right]$, considering two holes

c — — — $A_t = t[w - (\text{the largest of } d_1, d_2, \text{ or } d_3)]$, considering any one hole

Figure 39. Net area

11.3.5 Pin connected members

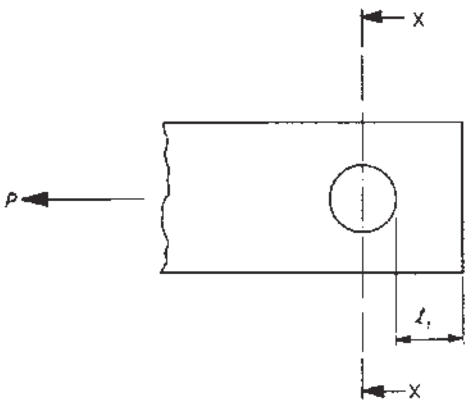
Add at end:

Where this requirement is not met, it shall be checked that tearing will not occur beyond the pin hole.

11.4 Thickness at pin holes

Add at end:

Where this requirement is not complied with, it shall be checked that local buckling will not occur beyond the pin hole.



NOTE. The net area within the length l_t should not be less than the required net cross-sectional area of the member.

The thickness of section X-X should not be less than one-sixteenth of the net width at X-X, unless the edges are stiffened.

Figure 40. Pin-connected member

11.5 Strength

11.5.1 ***Axial tension.*** A member or part subjected to axial tension should be such that the axial load does not exceed the resistance P_D given by:

$$P_D = \frac{\sigma_y A_e}{\gamma_m \gamma_{f3}}$$

where

A_e is the effective cross-sectional area of the member or part given in **11.3.2**

σ_y is the nominal yield stress of the member or part.

11.5.2 ***Combined tension and bending.*** A member subjected to coexistent tension and bending should be such that, at all cross sections:

$$\frac{P}{P_D} + \frac{M_x}{M_{Dxt}} + \frac{M_y}{M_{Dyt}} \leqslant 1.0$$

where

P is the axial tensile force in the member

P_D is as derived in **11.5.1**

M_x and M_y are the coexistent bending moments at the section considered in the member about the X-X and Y-Y axes, respectively (see figure 1)

M_{Dxt} and M_{Dyt} are the corresponding bending resistances with respect to the extreme tensile fibre, determined in accordance with **9.9**, or derived in accordance with **9.10** if the member is stiffened longitudinally.

Additionally, if at any section within the middle-third of the length of the member, the maximum compression stress due to bending exceeds the tensile stress due to axial load, the design should be such that:

$$\frac{M_{x\,max}}{M_{Dxc}} + \frac{M_{y\,max}}{M_{Dyc}} \leqslant 1 + \frac{P}{P_D}$$

where

P and P_D are as defined above

$M_{x\,max}$ and $M_{y\,max}$ are the maximum bending moments anywhere within the middle-third of the length of the member

M_{Dxc} and M_{Dyc} are the corresponding bending resistances with respect to the extreme compression fibres, determined in accordance with **9.9**, or derived in accordance with **9.10** if the member is stiffened longitudinally.

11.5.3 ***Eccentricity of end connections.*** The bending moment resulting from any eccentricity of the end connections of a member or its component should be taken into account in determining the values of the bending moments referred to in **11.5.2**.

In the case of a member consisting of a single angle connected only by one leg, or of a rolled or built-up tee-section connected only by the table of the tee, or a single channel section connected only through the web, these provisions may be considered to be met if the effective area of the unconnected legs is taken as:

$$\left(\frac{3A_1}{3A_1 + A_2}\right)A_2 \text{ for angles and } \left(\frac{5A_1}{5A_1 + A_2}\right)A_2$$

for tees and channels

where

A_1 is the net area of the connected leg of the angle, or of the table of the tee, or of the web of the channel

A_2 is the net area of the unconnected leg of the angle, or of the stalk of the tee, or of both unconnected flanges of the channel

NOTE. The table of a tee-section or web of a channel should be connected without eccentricity about the centroidal axis perpendicular to the table of the tee or the web of the channel

11.6 Battened tension members

11.6.1 ***General.*** A tension member consisting of two or more main components may have battens connecting the components, as described in **10.8.1**.

11.6.2 ***Spacing of battens.*** Battens should generally be spaced uniformly throughout the length of the member.

11.6.3 ***Dimensions of battens.*** The length of each batten, measured between end attachments in a direction parallel to the axis of the member, should not be less than three-quarters of the distance between the centroids of adjacent main components in the case of end battens, or half such distance in all other cases.

The thickness of each batten should not be less than one-sixtieth of the distance between the innermost lines of attachments, except that, where both transverse edges of a batten are effectively stiffened by stiffeners having a slenderness ratio not exceeding 170, the thickness of the batten need not exceed 8 mm.

11.6.4 ***Arrangement of battens in single or parallel planes.*** In any battened member, other than a member of cruciform section, battens should be arranged in accordance with **10.8.5.1**.

11.6.5 ***Arrangement of battens in cruciform members.*** Battens in cruciform members should be arranged in accordance with **10.8.6.1**.

11.6.6 ***Connection of battens.*** A batten attached by bolts or rivets should be connected to each main component by at least two bolts or rivets. Welds used to attach battens to the components should be in accordance with **10.8.7**.

11.6.1 Genera1

Add at end:

Where battens have been incorporated to cater for lateral loading or vibration (or for erection and handling during construction), and the requirements of **11.6.2** to **11.6.7** are not complied with, the battens and their fixings shall be assessed to resist the effects of all-loading to which they are subjected, including wind.

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11.6.7 ***Loads on battens.*** Battens and their fixings should be designed to resist the effects of any external transverse loads on the member.

11.7 Laced tension members

11.7.1 ***General.*** A tension member consisting of two or more main components may have lacing connecting the components, as described in **10.9.1**.

A laced tension member should be provided with a batten, in accordance with **11.6**, in each plane of lacing, at each end of the member, and at each point where the system of lacing is interrupted or where another member is connected to the laced member.

11.7.2 ***Inclination of lacing bars.*** The inclination of lacing bars should be in accordance with **10.9.2**.

11.7.3 ***Loads on lacing bars.*** Lacing bars and their fixings should be designed to resist the effects of any external loads applied transversely to the member. Such loads should be considered to be divided equally between all systems of lacing or plates connecting the main components in the appropriate parallel planes. The slenderness of bars should be determined in accordance with **10.9.4**.

11.7.4 ***Double lacing.*** A double system of intersecting lacing bars should be in accordance with **10.9.6**.

11.7.5 ***Welding of lacing bars to main components.*** Where a lacing bar is connected by welding to a main component of a member, the welding details should be in accordance with **10.9.7**.

11.8 Tension members connected by perforated plates

11.8.1 ***General.*** A tension member consisting of two or more main components may have continuous perforated plates connecting the components, either in one plane, or in two or more parallel planes, or in two perpendicular sets of parallel planes.

The thickness of a perforated plate should not be less than one-sixtieth of the unsupported distance between innermost attachments to the main components. Each end of a perforation should be rounded.

The clear distance between perforations should not be less than half the unsupported distance between innermost attachments to the main components, and the clear length beyond the perforation at each end of the member should not be less than three-quarters of such unsupported distance.

The unsupported width of a plate at a perforated section, between the inner edge of the perforation and the nearest attachment to a main component, should not be more than 20 times the thickness of the plate.

11.8.2 ***Strength of member.*** The net section of a perforated plate may be included as part of the effective section of the member when computing the strength of the member in accordance with **11.1** to **11.5**.

11.8.3 ***Loads on perforated plates.*** Perforated plates and their fixings should be designed to resist the effects of any external transverse loads on the member. Such loads should be considered to be divided equally between all the perforated plates and the other plates connecting the main components in the appropriate parallel planes.

11.9 Tension members with components back to back. Tension members with components back to back should be in accordance with **10.11.1**.

Connections between components should satisfy the provisions of **10.11.3**, except that the shear forces specified for battens need not be considered.

12. Design of trusses

12.1 General. 'Trusses' are defined as triangulated skeletal girders. Design of individual members and connections should be in accordance with **12.2** to **12.7** in conjunction with clauses **10**, **11** and **14** as appropriate.

12.2 Limit states

12.2.1 ***Ultimate limit state.*** All members and components of a truss should satisfy the provisions of clause **12** for the ultimate limit state. In trusses with stiff joints stresses due to axial deformation of members may be ignored.

12.2.2 ***Fatigue.*** Fatigue endurance should be in accordance with the recommendations of Part 10 taking into account coexistent axial and bending stresses in members in accordance with **12.3.1**.

12.2.3 ***Serviceability limit state.*** Tension members need not be checked for the serviceability limit state. All compression members should be checked for the serviceability limit state except:

(a) members of compact section throughout their length, as defined in **10.6.3**; or

(b) members meeting at a joint of a simply supported Warren truss, or modified Warren truss (see figure 41), provided the ratio of length to width, in the plane of the truss, of each of these members is equal to or greater than 12 for the chord members and 24 for web members; the length being taken between the centres of intersection and the width in the plane of the truss

12.3 Analysis

12.3.1 ***General.*** The effects of interaction between the members of the main trusses and the lateral bracing system of a bridge structure should be considered.

12.3.2 ***Global load effects.*** The global load effects on the structure should be calculated in accordance with the elastic theory, based on the assumption that all members are straight, and that either:

(a) all members are pin jointed and each joint lies at the intersection of the centroidal axes of the relevant members and all loads, including the self-weight of members, are applied at the joints; or

(b) the joints are stiff.

When considering the limit state of fatigue, or the limit state of serviceability, either method (b) should be used, or method (a), modified by the inclusion of flexural stresses due to axial deformation, self-weight of the members and the stiffness of joints. If any prestressing of the structure is adopted to counteract these stresses, this should be taken into account for the serviceability limit state, but only 90% of the relieving effects of the prestressing should be considered.

11.7.1 General

Add at end:

Where lacing has been incorporated to cater for lateral loading or vibration (or for erection and handling during construction), and the requirements of **11.7.2** to **11.7.5** are not complied with, the lacing bars and their fixings shall be assessed to resist the effects of all loading to which they are subjected including wind.

11.8.1 General

Add at end:

Where the above requirements are not met, the perforated plate shall be assessed to resist the effects of all loading to which it is subjected including wind.

11.9 Tension members with components back to back

Add at end:

Where the requirements of **10.11.1** and **10.11.3** are not complied with, the members and their fixings shall be assessed to resist the effects of all loading to which they are subjected including wind.

12.1 General

Add at end:

Bending effects shall be ignored in **12.2.2** and **12.2.3** where these are solely due to axial deformation of members where the joints are formed using untensioned bolts or rivets in clearance holes and any secondary bending developed can be relieved by joint movement.

12.3.3 *Local load effects*

12.3.3.1 *Loads not applied at truss joints.* Account should be taken of the following:

(a) resulting stresses when load is applied to a member in the plane of a truss other than at a joint;

(b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross member, the effect of interaction between the cross member so loaded and the truss and adjacent cross members should be taken into account.

12.3.3.2 *Eccentricities at joints.* If, at a joint, the centroidal axes of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account.

12.4 Effective length of compression members

12.4.1 ***General.*** The effective length ℓ_e of a compression member should either be obtained from table 11 or be determined by an elastic critical buckling analysis of the truss. In applying table 11, the end raker of a truss should be considered as a web member.

12.4.2 ***Lateral restraint by deck to compression chord.*** A compression chord, continuously supporting a steel or reinforced concrete deck, may be deemed to be effectively restrained laterally throughout its length if the frictional or other connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length, of 2.5 % of the maximum force in the chord. The effective length ℓ_e of such a compression chord should be taken as zero where friction provides an adequate restraint, or as equal to the spacing of discrete connections where these are provided.

12.5 Unbraced compression chords

12.5.1 ***Effective length.*** Where a compression chord is not provided with a system of lateral bracing, but is restrained laterally by U-frames comprising cross members and truss web members (see figure 41), the effective length ℓ_e of the chord member may be taken as:

$$\ell_e = 2.5 k_3 (E I_c a \delta)^{0.25}, \text{ but not less than } a$$

where

k_3 may be taken as 1.0, but where the compression chord is restrained against bending in plan at its section over the supports of the truss, a lower value of k_3 may be obtained from figure 7(b)

I_c is the maximum second moment of area of the chord about the Y-Y axis shown in figure 41

a is the distance between U-frames

δ is the lateral deflection which would occur in the U-frame, at the level of the centroid of the chord being considered, when a unit force acts laterally to the U-frame only at this point and simultaneously at each corresponding point on the other chord or chords connecting to the same U-frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-frame should be taken as fixed in position at each point of intersection between the cross member and an upright, and as free and unconnected at all other points.

In cases of symmetrical U-frames, where cross members and verticals are each of constant moment of inertia throughout their own length, it may be assumed that:

Table 11. Effective length ℓ_e for compression members in trusses

Member		Effective length ℓ_e: Buckling in plane of truss	Buckling normal to plane of truss when: compression chord is effectively braced by lateral system	Buckling normal to plane of truss when: compression chord is unbraced
Chord		0.85 × distance between intersections with web members	0.85 × distance between intersections with lateral bracing members or rigidly connected cross beams	See 12.5.1
Web member	Single triangulated system	0.70 × distance between intersections with chords	0.85 × distance between intersections with chords	Distance between intersections with chords
Web member	Multiple intersection system with adequate connections at all points of intersection	0.85 × greatest distance between any two successive intersections	0.70 × distance between intersections with chords	0.85 × distance between intersection with chords

12.5.1 Effective length

Delete the existing text and substitute the following

Where a compression chord is not provided with a system of lateral bracing, but is restrained laterally by U-frames comprising cross members and truss web members (see figure 41), the effective length l_e of the chord member shall be taken as:

$l_e = K_3 K_5 (E I_c a \delta)^{0.25}$ but not less than a

where

K_3 shall be taken as 1.0 but where the compression chord is restrained against bending in plan at its section over the supports of the truss, a lower value of K_3 shall be obtained from figure 7(b).

K_5 = 2.5 where the trusses are restrained at their supports against torsion about their longitudinal axis in accordance with **12.5.3.2**, or

= π where the trusses are unrestrained at their supports against torsion about their longitudinal axis.

NOTE: Where the restraint against torsion at supports is less than required to resist forces $2(F_U + Fc)$ under **12.5.3.2** then linear interpolation shall be assumed to determine a value of K_5 between 2.5 and π.

I_c Is the maximum second moment of area of the chord about the Y-Y axis shown in figure 41;

a is the distance between U-frames;

δ is the lateral deflection which would occur in the U-frame, at the level of the centroid of the chord being considered, when a unit force acts laterally to the U-frame only at this point and simultaneously at each corresponding point on the other chord or chords connecting to the same U-frame. The direction of each unit force shall be such as to produce the maximum aggregate value of δ. The U-frame shall be taken as fixed in position at each point of intersection between the cross member and an upright, and as free and unconnected at all other points.

NOTE: In cases of symmetrical U-frames, where cross members and verticals are each of constant moment of inertia throughout their own length, it shall be assumed that

$$\delta = \frac{d_1^3}{3E\left(I_1 + I_3 \dfrac{d_1^3}{d_3^3}\right)} + \frac{usd_2^2}{EI_2} + fd_2^2$$

where

d_1 is the distance from the centroid of the compression chord to the nearer face of the cross member of the U-frame;

d_2 is the distance from the centroid of the compression chord to the centroidal axis of the cross member of the U-frame;

d_3 is the length of the diagonals measured as the distance sloping from the centroid of the compression chord to the top face of the cross member of the U-frame;

I_1 is the second moment of area of the web member forming an arm of the U-frame in its plane of bending;
NOTE: In the case of warren girder without intermediate vertical members then the diagonal members shall be assumed to provide U-frame stiffness, with $I_1 = 0$

I_2 is the second moment of area of the cross member in its plane of bending;

I_3 is the second moment of area of the diagonals which are assumed to act as part of the U-frame acting in their plane of bending;

$$\delta = \frac{d_1^3}{3EI_1} + \frac{usd_2^2}{EI_2} + fd_2^2$$

where

d_1 is the distance from the centroid of the compression chord to the nearer face of the cross member of the U-frame

d_2 is the distance from the centroid of the compression chord to the centroidal axis of the cross member of the U-frame

I_1 is the second moment of area of the web member forming an arm of the U-frame in its plane of bending

I_2 is the second moment of area of the cross member in its plane of bending

u = 0.5 for an outer beam, or
= 0.33 for an inner beam, if there are three or more beams interconnected by U-frames

s is the distance between centres of consecutive main girders connected by the U-frame

f is the flexibility of the joint between the cross member and the verticals of the U-frame, expressed in radians per unit moment; f may be taken as:

0.5×10^{-10} rad/N mm, when the cross member is bolted or riveted through unstiffened end plates or cleats (see figure 42 (a)), or

0.2×10^{-10} rad/N mm, when the cross member is bolted or riveted through stiffened end plates (see figure 42 (b)), or

0.1×10^{-10} rad/N mm, when the cross member is welded right round its cross section or the connection is by bolting or riveting between stiffened end-plates on the cross member and a stiffened part of the vertical or a stiffened section of the chord (see figure 42(c)).

In the case of modified Warren or similar trusses (see figure 41), the end raker should be considered as a chord for the purposes of this clause.

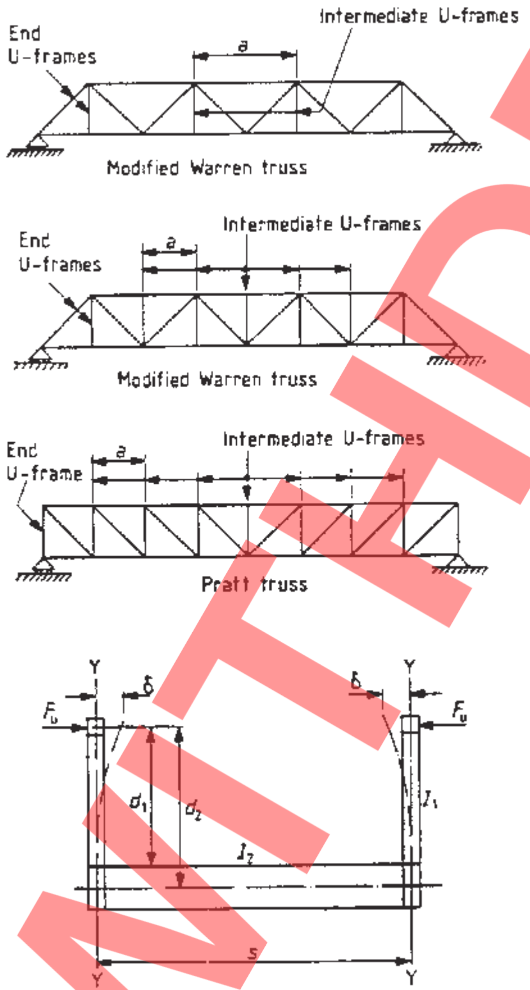

Figure 41. Lateral restraint by U-frames

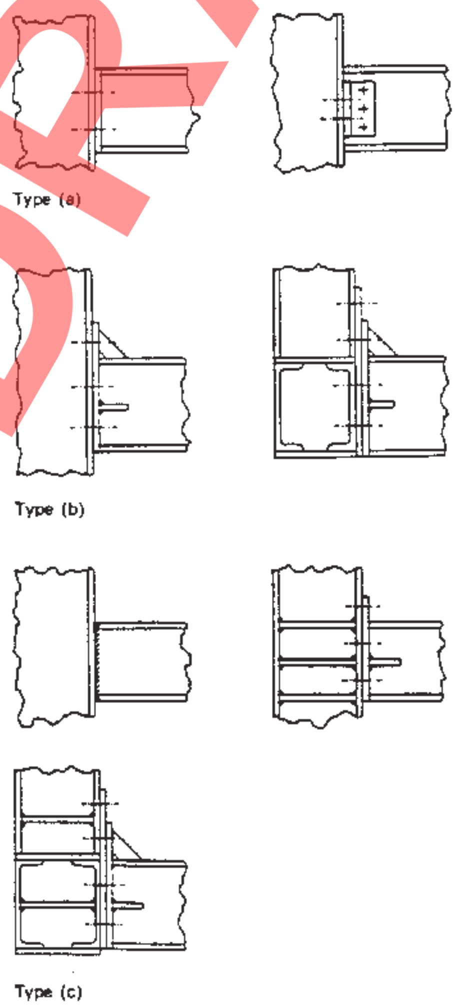

Figure 42. U-frame joints

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u = 0.5 for an outer beam, or
= 0.33 for an inner beam, if there are three or more beams interconnected by U-frames;

s is the distance between centres of consecutive main girders connected by the U-frame;

f is the flexibility of the joint between the cross member and the verticals of the U-frame, expressed in radians per unit moment: f shall be taken as 0.5×10^{-10} rad/N mm, when the cross member is bolted or riveted through unstiffened end plates or cleats (see Figure 42(a)) or

0.2×10^{-10} rad/Nmm. when the cross member is bolted or riveted through stiffened end plates (see Figure 42(b)) or

0.1×10^{-10} rad/N mm, when the cross member is welded right round its cross section or the connection is by bolting or riveting between stiffened end-plates on the cross member and a stiffened part of the vertical or stiffened section of the chord (see Figure 42(c)).

NOTE: Values of f may be determined experimentally or taken from test results available which shall cover the required ultimate capacity of the joint.

In the case of the modified warren or similar trusses (see Figure 41) the end diagonal shall be considered as chord for the purposes of this clause.

Figure 41. Lateral restraint by U-frames

Delete the existing fgure and substitute the following

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12.5.2 *Effect of loading on a cross member.* When the compression chord is held in position over the supports by means of end members in the plane of the truss, the lateral flexure of a compression chord due to loading on a cross member should be considered. In the absence of a rigorous analysis of the interaction of main girders and cross members, the maximum value M_y of the lateral bending moment in a compression chord may be taken as:

$$M_y = \frac{5EI_c\theta d_2}{L\ell_e\left(1 - \frac{P_c}{P_E}\right)} \times \left[1 + \frac{\frac{L}{\ell_e} - 1.25}{2.8 + 3.5\left(\frac{P_c}{P_E}\right)^2}\right]$$

provided that each main girder is in a vertical plane, and that the two chords of a main girder are parallel;

where

θ is the rotation (in rad) of the cross member at its junction with the main girder under consideration, under the loading used when calculating P_c. θ may be calculated neglecting any interaction between the cross member and the main girders. If, because of non-uniform loading, θ varies between cross members, the average value of θ for those cross members within the loaded portion of the span should be used

I_c and d_2 are as defined in **12.5.1**

L is the span of the main girder being considered

ℓ_e is as defined in **12.5.1**

P_c is the maximum force in the compression chord in the span being considered

P_E is taken as follows:

(a) $P_E = \dfrac{\pi^2 EI_c}{\ell_e^2}$

if ℓ_e is less than three times the spacing of U-frames;

(b) $P_E = \dfrac{1.25\pi^2 EI_c}{\ell_e^2}$

if ℓ_e is more than four times the spacing of U-frames;

(c) P_E is obtained by linear interpolation, for intermediate values of ℓ_e.

For HA or RL or any uniformly distributed loading when placed over the whole span, the maximum moment M_y derived above should be assumed to act anywhere within a horizontal distance ℓ_e from each bearing support of the beam. Elsewhere the bending moment should be assumed to be $0.5M_y$.

For all other loading cases it should be assumed that M_y acts anywhere within the span.

12.5.3 *U-frames*

12.5.3.1 *Intermediate U-frames.* Each intermediate U-frame and its connections should be designed to resist all of the following:

(a) Wind and other applied forces.

(b) Horizontal forces F_u acting normal to the compression chord at the level of its centroid, given by:

$$F_u = \left(\frac{P_c}{P_E - P_c}\right)\frac{\ell_e}{667\,\delta}$$

but not greater than:

$$\left(\frac{P_c}{P_E - P_c}\right)\frac{EI_c}{16.7\,a^2}$$

where

P_c, P_E are as defined in **12.5.2**

ℓ_e, δ, I_c, a are as defined in **12.5.1**.

In the case of several interconnected trusses, two such forces F_u should be applied in the same or opposite directions, in such a way as to produce the most severe effect in the part being considered.

(c) Horizontal forces F_c applied to the U-frame at the same points and in the same manner as in (b) for F_u. The forces F_c result from the interaction between the bending of the cross member and the vertical members of the U-frame, and, in the absence of a more rigorous analysis, should be assumed to be given by:

$$F_c = \frac{3EI_1\theta}{d_2^2}$$

where

θ is as defined in **12.5.2**

I_1, d_2 are as defined in **12.5.1**.

12.5.3.2 *End U-frames.* End U-frames of trusses restrained laterally by a system of U-frames should be designed to resist all applied loads, and, in addition lateral forces each equal to $2(F_u + F_c)$, where the magnitudes of F_u and F_c and their manner of application are given in **12.5.3.1**.

In the case of Warren-type trusses, where the extreme node of the compression chord is laterally restrained by U-frames both in the plane of the end diagonals and in the plane of the extreme verticals, the total lateral forces given above may be assumed to be shared by the two sets of U-frames.

12.6 Lateral bracing

12.6.1 *General.* Sufficient bracing should be provided between main trusses to ensure that all external and stabilizing loads and load effects can be transmitted to the supporting structures, and that restraint is provided at all intersection points where such restraint is assumed in determining the effective length of compression members, and also at each point where a compressive force is applied to a web member, owing to change of direction of a chord (whether the chord is in tension or compression).

Bracing members and their connections to compression chords, or to U-frames restraining compression chords, should be designed to resist the forces given in **12.6.2**. U-frames should be in accordance with **12.5.3**.

12.6.2 *Forces on bracing.* Lateral restraint should be provided to compression chords in such a way that at all transverse sections of the bridge the following stabilizing lateral shear force can be resisted:

(a) $\Sigma P_c/80$ when the load combination includes wind and other laterally applied forces;

(b) $\Sigma P_c/40$ when the load combination excludes wind and other laterally applied forces;

where

ΣP_c is the sum of the greatest coexistent axial forces in any two chords at the section under consideration.

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12.5.3.1 Intermediate U-frames

In item (c), delete the expression for 'F_c' and the definitions for 'θ, I, and d_2' and substitute the following:

$$F_c = \frac{\theta d_2}{\delta + \dfrac{a^3}{15EI_c}}$$

where

d_2, δ, a and I_c are as defined in **12.5.1**

θ is as defined in **12.5.2** except that it should be calculated assuming the cross member is simply supported.'

Add at end:

Where in assessment of the adequacy of an intermediate U-frame allowance is to be made for initial departures from straightness of a compression chord F_u shall be calculated in accordance with **9.12.2.2** with l_e in accordance with **12.5.1.**

Add new Clause 12.6.3:

12.6.3 Lateral bracing not providing adequate restraint

In cases where any of the provisions of **12.6.1** or **12.6.2** are not met in assessment, such bracing shall be ignored and assumed to provide no restraint. However, in cases where partial restraint may be available from any lateral bracing provided, this may be utilised providing it can be verified by a rigorous non-linear analysis of the complete system. Alternatively, the values of ΣP_C in **12.6.2** could be restricted such that the requirements of **12.6.2** are complied with.

12.7 Curved members. A tension or compression member curved to a circular arc (see figure 43) may be designed in accordance with this Part, provided that:

(a) the deviation δ from the straight line joining the points of intersection at the ends of the member does not exceed one-twelfth of the length of the straight line;

(b) the cross section is compact as defined in **10.6.3**;

(c) a flange outstand, if any, is such that:

$$\frac{b_o}{t_o} \leqslant \frac{R}{6b_o}$$

where

b_o is the width of the outstand measured from the edge to the nearest line of bolts or rivets connecting it to a supporting part of the member, or to the surface of such supporting part in the case of welded construction, or to the root fillet of a rolled section

t_o is the mean thickness of the outstand, or the aggregate thickness where two or more parts are connected in accordance with **14.5** or **14.6**

R is the radius of curvature;

(d) the unsupported width of flange is such that:

$$\frac{b}{t_f} \leqslant \frac{R}{2b}$$

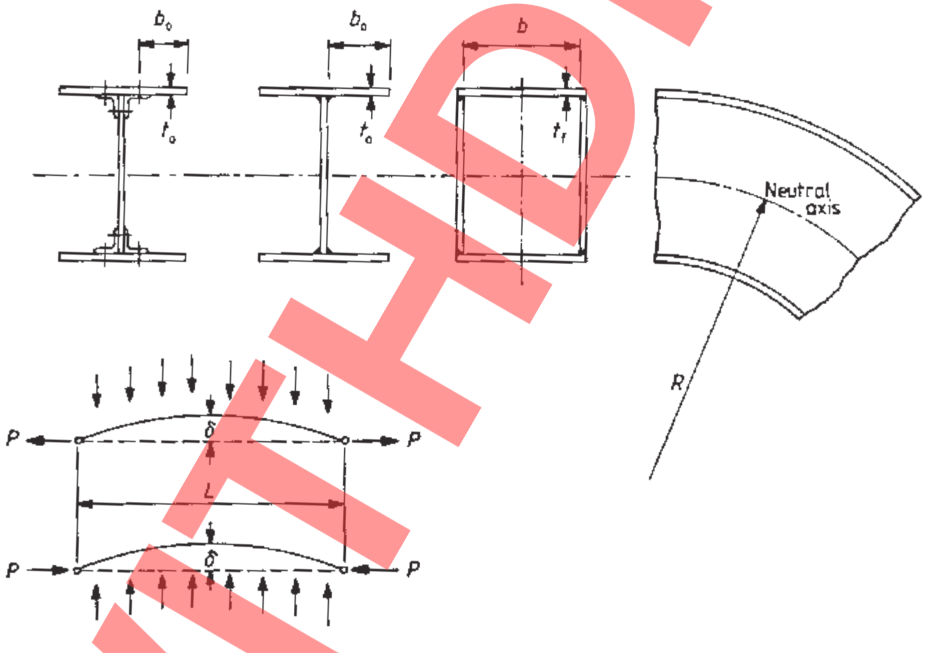
where

b is the unsupported width of flange between lines of bolts or rivets connecting the plate to supporting parts of the member, or between surfaces of such supporting parts in the case of welded construction, or between root fillets of rolled sections

t_f is the mean thickness of the flange over width b, or the aggregate thickness where two or more parts are connected in accordance with **14.5** or **14.6**;

(e) a transverse load of uniform intensity is considered as applied in the plane of the curve throughout the length of the member, acting on the convex side of a tension member, or the concave side of a compression member, and having a value P/R, where P is the axial force in the member.

Bending moments in the member should be calculated from this load on the assumption that the member is pin-ended, and should be superimposed on the bending moments due to rigidity of the joints determined in accordance with **12.3.2**(b).



NOTE. δ is not greater than $L/12$.

Figure 43. Curved members

12.7 Curved members

Add at end:

Where members do not comply with requirements (a) to (d) consideration shall be given to the following:

(1) The forces and stresses according to **9.5.7**;
(2) The effects of the change in neutral axis position due to curvature;
(3) The buckling resistance of the section where it does not satisfy the criteria for a compact section;
(4) Flanges must be adequate to resist the radial component of the flange force. Assuming the axial force in the flange is distributed uniformly across the width, the line load radial force per unit width across the flange per unit length of the flange may be expressed as:

$$\frac{\sigma_f\, t_{fo}}{R_f}$$ in a flange outstand, or

$$\frac{\sigma_f\, t_f}{R_f}$$ in a plate panel between longitudinal stiffeners and/or webs

where σ_f, t_{fo}, t_f and R_f are all as defined in **9.5.7.1.**

12.8 Gusset plates

12.8.1 *Strength.* Gusset plates should be capable of resisting the actions from connected members in such a way that the maximum equivalent stress does not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_y is the nominal yield stress of the gusset material.

Any reasonable assumption as to the distribution of stresses may be made, provided that the assumed stresses are in equilibrium with the forces in the connecting members, and that the connections are in accordance with clause 14.

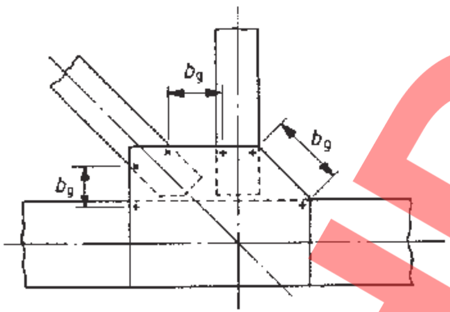
In assessing the fatigue life, the stresses in a gusset plate should be determined by elastic analysis.

12.8.2 *Detailing.* Gusset plates should be so shaped, and connnectors so arranged, as to avoid severe stress concentrations. The lengths b_g of unstiffened unsupported edges shown in figure 44 should be such that:

$$\frac{b_g}{t} \leqslant 50\sqrt{\frac{355}{\sigma_y}}$$

where

t is the thickness of the gusset
σ_y is the nominal yield stress of the gusset material.



NOTE. The unsupported edge distance b_g is measured clear between fixings.

Figure 44. Gusset plates

13. Design of base, cap and end plates

Base plates, cap plates and end plates should be designed at the ultimate limit state to satisfy the following yield criterion:

$$\frac{16M^2}{t^4} + \frac{3V^2}{t^2} \leqslant \left(\frac{\sigma_y}{\gamma_m \gamma_{f3}}\right)^2$$

where

M is the maximum bending moment per unit width of the plate
V is the co-existent shear force per unit width
t is the plate thickness
σ_y is the nominal yield stress of the plate material.

In addition, the calculated deformation of such a plate and the contact pressure distribution assumed should be compatible with the assumptions made in the design of the adjacent structural elements.

14. Design of connections

14.1 General. The term 'connection' applies to all joints between different components of a structural member, joints between separate structural members and splices in members.

The term 'fastener' applies to bolts, rivets and pins.

14.2 Limit states

14.2.1 *Ultimate limit state.* All connections should satisfy the provisions of clause 14 for the ultimate limit state.

14.2.2 *Fatigue.* The fatigue endurance should be in accordance with the recommendations of Part 10.

14.2.3 *Serviceability limit state.* Connections made with HSFG bolts in accordance with BS 4395 : Parts 1 and 2 in normal clearance holes should also satisfy the provisions of clause 14 for the serviceability limit state (see **14.5.4.1**). This limit state should be deemed to be reached when the design shear load on a bolt equals its friction capacity.

14.3 Basis of design

14.3.1 *General.* Connections should be designed on the basis of the strengths of the individual fasteners or welds, to transmit at least the design loads and moments communicated by the members.

14.3.2 *Alignment of members.* The centroidal axes of members meeting at a joint or at a splice should preferably meet at a point. When this is not the case, the moment on the connection due to any eccentricity should be taken into account.

14.3.3 *Distribution of loads between fasteners or welds*

14.3.3.1 *Elastic analysis.* The distribution of forces between individual fasteners in a bolted or riveted connection and between welds in a welded connection may be determined on the assumption that:

(a) all the fasteners and all the welds share the design axial load in proportion to their respective strengths;
(b) the force on a fastener or a length of weld due to a moment on the connection is proportional to its distance from the centroid of the connection.

14.3.3.2 *Plastic analysis.* Except in the case of connections made with HSFG bolts, for which the friction capacity of the fasteners is taken as the design strength in accordance with **14.5.4.2**, any reasonable distribution of the forces on the fasteners and stresses in the welds may be assumed provided that:

(a) they are in equilibrium with the applied load effects;
(b) the implied deformations are within the capacity of the fasteners or welds and of the connected parts;
(c) each element in the connection is capable of resisting the forces or stresses assumed in the analysis.

14.3.4 *Distribution of load to the connected members.* As far as possible, members should be so connected that the load in the connected member is appropriately distributed over its whole effective section. Where any part of a member cannot be connected so as to meet this provision, the manner in which the load effects are distributed should be considered. For this purpose, it may be assumed that the load is dispersed from a fastener into a connected part within an angle of ±45° from the direction of the force.

Groups of fasteners should be as compact as possible.

12.8.2 Detailing

Add at end:

In the case of severe changes in geometric shape such as the presence of sharp re-entrant cuts then stress concentration factors shall be applied.

Where b_g/t exceeds the above limit then local buckling of the gusset plates shall be checked, either by means of a detailed analysis or by means of reducing the yield stress, σ_y, given in **12.8.1** to a value given by

$$0.9 \times 10^6 \left(\frac{t}{b_g} \right)^2$$

14.1 General

Add at end:

For assessment the term 'fastener' also applies to the components of members such as screwed tie rods and turnbuckles.

14.2.3 Serviceability limit state

Add at end:

Where, in assessment, such connections are calculated to slip under serviceability factored loading in accordance with **14.5.4.1.2** and no distress is apparent at the joints (see Note 3 in **4.2.2**) they shall be checked under ultimate factored loading in accordance with **14.5.4.1.1**(b) and the fatigue endurance of the joint shall be checked assuming the fasteners to be black bolts. If there is evidence of loose rivets in rivetted connections then the fatigue endurance of the joint shall also be checked assuming the fasteners to be black bolts. Fatigue endurance shall be assessed in accordance with **14.2.2.**

Add new Clause 14.3.3.3:

14.3.3.3 Assessment

For assessment elastic analysis shall be used in accordance with **14.3.3.1** for H.S.F.G bolts checked for the serviceability limit state when adopted under **14.2.3**, and when considering the fatigue endurance of welds as required under **14.2.2**. In other ases, plastic analysis shall be used in accordance with **14.3.3.2.**

It may normally be assumed that for the assessment of connections in beams, all the bending is resisted by the flanges along with any associated flange angles, and :hat shear only is resisted by the web, provided that this is compatible with the basis of the assessment of the member.

14.3.4 Distabution of load to the connected members

Add at end:

For assessment where any part of a member is connected so that the load is not distributed over its effective section, then it shall be assumed that the load is dispersed from a fastener onto a connected part within an angle of ± 45° from the direction of the force, unless a detailed analysis is carried out which can substantiate an improved distribution.

14.3.5 ***Connection of restraints to parts in compression.*** A connection between a part in compression and any intermediate restraints should be designed to resist:

(a) a force equal to 2.5 % of the axial force in the member acting in a direction opposite to that of the restraint; and

(b) the effects of any other external load/moment on the member.

14.3.6 ***Prying force.*** Where bolts or rivets are required to carry an applied tensile load P_t, they should be proportioned to resist an additional force H due to prying action where this can occur (see figure 45). The force H should be assigned a value not smaller than $P_t/10$, such that the bending moments at sections 1 and 2 do not exceed:

$$\frac{1}{\gamma_m \gamma_{f3}} \times \text{(plastic moment capacity of the respective section based on the net area of the section).}$$

In determining the moment capacity at section 2, the length of the section may be determined on the assumption of a dispersion from the fasteners at angles not exceeding $\pm 60°$ from the normal to section 2, but not beyond half of the distance to the adjacent fasteners in the line. The same gross length may be assumed to apply when determining the moment capacity at section 1.

14.4 Splices

14.4.1 ***Cover material***

14.4.1.1 *General.* Where cover material is used to transmit load through a splice the following conditions should be satisfied whenever practicable:

(a) both surfaces of the spliced parts should be provided with cover material;

(b) cover material should be so disposed, with respect to the cross section of the member, as to communicate the proportional load in the respective parts of the section.

Where the provisions of either (a) or (b) are not met, the effect of any eccentricity of cover material with respect to the centroid of the spliced section, or any part of such section, should be considered when determining the strength both of cover material and of the member or part.

14.4.1.2 *Welded stiffener splices.* The edges of cover material in a welded stiffener splice should be welded to the plate which is being stiffened.

14.4.2 ***Compression members***

14.4.2.1 *Loads to be transmitted.* A splice located at or near an effectively braced joint shall be capable of transmitting at least the design load effects in the member.

All other splices shall be capable of transmitting at least the load effects and the stresses at the spliced section due to initial imperfections. To achieve this, the splice should be designed to transmit a force equal to $\frac{P}{P_D} \times P_{Dk}$, in addition to any coincident moments and shears;

where

P is the load in the member or part

P_D is the resistance of the member or part

P_{Dk} is the resistance of the member or part determined as if it were a stocky member (see 10.6.3).

14.4.2.2 *Design stresses.* The maximum stress σ_s in a spliced part and that in the cover material should not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_s is the axial stress or, where shear is present, the equivalent stress, based on the effective area of the cross section determined in accordance with 10.5

σ_y is the nominal yield stress of the spliced part or the cover material, as appropriate.

14.4.2.3 *Machined abutting ends of parts in compression.* A splice which has machined abutting ends in contact over the whole area of the section may be assumed to carry 75 % of any compressive load directly through the abutting ends. If the abutting area is increased by means of machined end plates, the whole compressive load may be assumed to be transmitted through the abutting faces. The alignment of the abutting ends shall be maintained by cover plates or other means. The cover material and its fastenings should be proportioned to carry a force at the butting ends, acting in any direction perpendicular to the axis of the member equal to 2.5 % of the compressive force in the member.

14.4.3 ***Tension members***

14.4.3.1 *Loads to be transmitted.* A splice in a member or part subjected to tension shall be designed to transmit at least the load in the member or part.

14.4.3.2 *Design stresses.* The maximum stress σ_s in the spliced part and that in the cover material should not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_s is the axial stress or, where shear is present, the equivalent stress, based on the effective section determined in accordance with 11.3 or 0.8 times the effective section for outer plies in connections made with HSFG bolts acting in friction

σ_y is the nominal yield stress of the splice part or the cover material, as appropriate.

14.4.4 ***Members in bending***

14.4.4.1 *General.* A splice in a member or part subjected to bending and axial load effects should satisfy the requirements of 14.4.4.2 to 14.4.4.4 and 14.4.2 or 14.4.3 as appropriate.

14.4.4.2 *Compression flanges.* Compression flanges should be treated as compression members and spliced in accordance with 14.4.2. In determining the load to be transmitted at a splice that is not effectively braced, the following definitions should be adopted:

P is the force in the compression flange at the splice position

P_D is the flange compression calculated from the bending resistance of the beam at the position of the maximum bending moment

14.3.5 Connection of restraints to parts in compression

Add at end:

In cases where the connection cannot resist the forces in (a) and (b) above, the intermediate restraint shall be ignored, or the system may be checked making due allowance for the maximum restraint that can be provided, see **9.6.**

14.3.6 Prying force

Add at end:

Where more than one line of bolts or rivets is present, then in the absence of effective stiffening to reinforce the connection, only the inner line of fasteners adjacent to the web shall be assumed as effective in resisting the tensile load.

The value of prying force assigned to the force H shall only be taken as $P_t/10$ providing a greater value is not produced by either H_1 or H_2, where:

$$H_1 = P_t \left\{ \frac{\tfrac{1}{2} - \left(Lt^4 / 30ab^2 A_e \right)}{\frac{a}{b}\left(\frac{a}{3b} + 1 \right) + \left(Lt^4 / 6ab^2 A_e \right)} \right\}$$

$$H_2 = \left[\frac{c}{2a} - \frac{1}{8} \right] \left[P_t - \left(F_v L t^4 / 18ac^2 A_e \right) \right]$$

In cases where both H_1 and H_2 are less than $P_t/10$, the higher value of H_1 or H_2 shall be used. For notation see Fig 45A in the Advice Note.

14.4.1.1 General

Add at end:

The following assumptions shall be made for assessment:

(a) Where both surfaces of the spliced parts are provided with covers then axial stresses shall only be assumed in design.

(b) Where only one surface is provided with covers, then bending effects are to be considered at the serviceability limit state, but may be ignored at the ultimate limit state. For the calculation of bending effects it may be assumed that the line of action of the axial force in the splice is located along the interface between the parent material and the cover. The effects of eccentricity shall be ignored when bending is effectively prevented by:

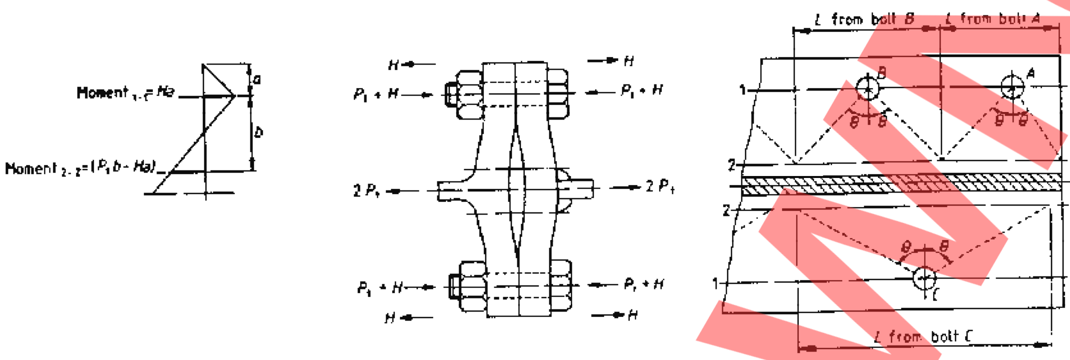
(i) the presence of surrounding or adjacent concrete or other solid infill, or

(ii) the presence of an element which prevents bending of either the parent material or the cover. This element shall be within a distance of 12t from the furthest fastener where t is the thickness of the parent material to which the cover plate is attached.

Add new Clause 14.4.5:

14.4.5 Obsolete splicing methods

In older bridges, limitations on thickness of plate which could be rolled meant that several plates were required to build up the required section thickness. When assessing splices in such section, consideration shall be given to the load path through the joint to ensure no single component is overloaded, see figure 14.4A and B.



NOTE. L is the limitation on the length of sections 1-1 and 2-2.
θ is the angle of dispersal of P_t and should not exceed 60°

Figure 45. Prying forces

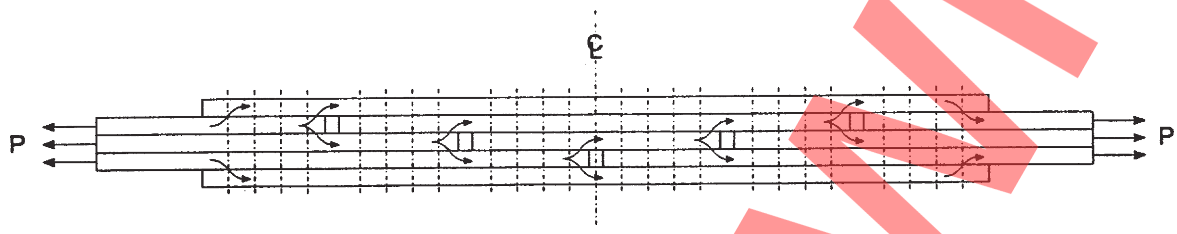
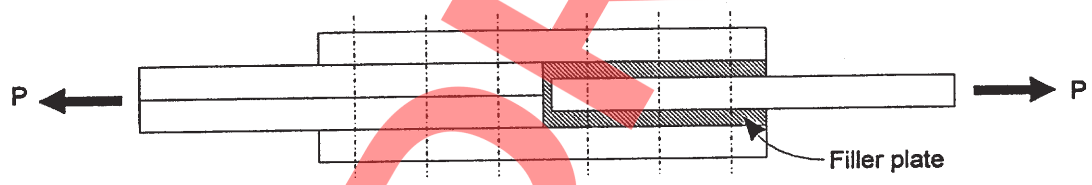
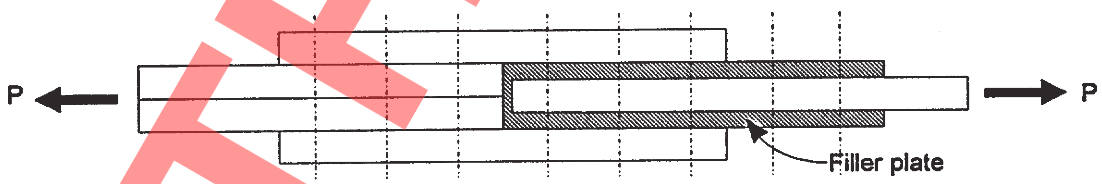


Figure 14.4A Force flow in typical triple plate shinglejoint



(a) Loose fillers



(b) Tight fillers

Figure 14.4B Types of filler plates.
(a) Loose fillers, (b) Tight fillers

14.5 Connections made with bolts, rivets or pins

14.5.1 ***Spacing of bolts or rivets***

14.5.1.1 *Minimum pitch.* The distance between centres of bolts or rivets should not be less than 2.5 times the diameter of the shank of the bolt or rivet.

14.5.1.2 *Maximum pitch*

14.5.1.2.1 *In any direction.* Except as noted in **14.5.1.2.2**, the distance between centres of two adjacent bolts or rivets should not exceed $32t$ or 300 mm, whichever is the lesser, where t is the thickness of the thinner of the outer parts joined (see figure 46).

14.5.1.2.2 *In the direction of stress.* Except as noted in **14.5.1.3**, the distance between centres of two consecutive bolts or rivets in a line lying in the direction of stress should not be greater than:

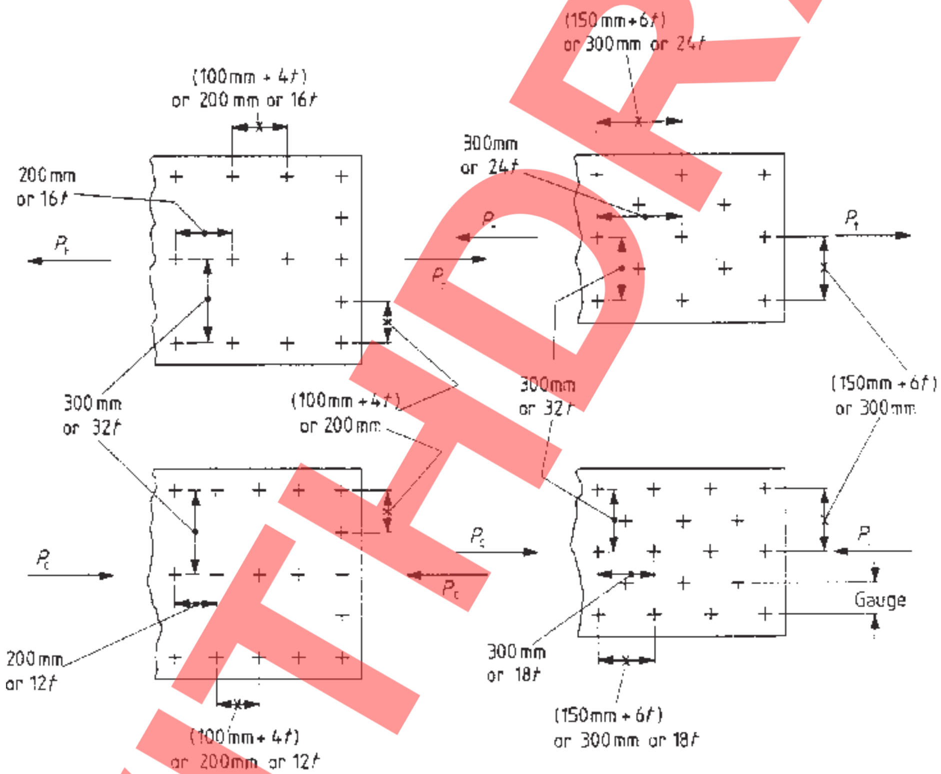
(a) $16t$ or 200 mm, whichever is the lesser, if the parts joined are in tension or shear;

(b) $12t$ or 200 mm, whichever is the lesser, if the parts joined are in compression. Where compressive forces are transferred through abutting faces, this distance should not exceed 4.5 times the diameter of the bolts or rivets for a distance from the abutting faces equal to 1.5 times the width of the member.

14.5.1.2.3 *Adjacent to an edge.* Except as noted in **14.5.1.3**, the distance between centres of two consecutive bolts or rivets in a line adjacent to, and parallel to an edge of, an outside connected part should not be greater than (100 mm + $4t$) or 200 mm, whichever is the lesser.

14.5.1.3 *Staggered spacing.* Where bolts or rivets are staggered at equal intervals, and the gauge is not greater than 75 mm, the maximum distance between centres of bolts or rivets, as given in **14.5.1.2.2** and **14.5.1.2.3**, may be increased by 50 %.

14.5.1.4 *Spacing in stiffener attachment.* The distance between centres of two consecutive bolts or rivets connecting a stiffener to a plate or other part subjected to compression or shear should not be greater than $b/4$, or the limit obtained from **14.5.1.2.2**, whichever is the lesser; where b is the maximum distance between adjacent stiffeners or between a stiffener and other support to the plate.



(a) Normal spacing

(b) Staggered spacing

NOTE 1. Staggered spacing values are for 75 mm gauge (maximum).

NOTE 2. Spacing of fasteners should not exceed the least relevant value. ● are appropriate away from edge. × are appropriate at edge.

Figure 46. Maximum pitch of bolts and rivets

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Add new Clause 14.5.1.5:

14.5.1.5 Assessment of non-complying arrangements

Where any of the limits in **14.5.1.1, 14.5.1.2, 145.1.3** or **145.1.4** are not complied with, allowance shall be made for a reduced strength of the fasteners or plate in assessment. Guidance is given in the accompanying Advice Note.

14.5.2 ***Edge and end distance.*** The distance from the centre of the fastener hole to the edge of a part should not be less than:

(a) $1.2d$ for bolts (other than HSFG bolts acting in friction) or rivets, or such larger distance as may be required to meet the provisions of **14.5.3.6**;
(b) $1.5d$ for HSFG bolts acting in friction;

where

d is the diameter of the hole.

A line of bolts or rivets should be placed at a distance of not more than (40 mm + $4t$) from any edge, where t is the thickness of the thinner outside part.

14.5.3 ***Strength of fasteners other than HSFG bolts acting in friction***

14.5.3.1 *General.* The ultimate strengths given for bolts and rivets in shear and bearing apply only to bolts and rivets in holes not larger than the sizes given in **4.5** of Part 6 of BS 5400 : 1980.

Black bolts should not be used in permanent main structural connections of highway and railway bridges.

For fasteners other than HSFG bolts, the ultimate limit state of a connection should be deemed to be reached when the design load on any fastener equals its ultimate capacity, determined in accordance with **14.5.3.2** to **14.5.3.11**.

14.5.3.2 *Bolts subjected to axial tension.* In a bolt subjected to applied axial tension, the tensile stress:

$$\sigma = \frac{P_t + H}{A_{et}}$$

should not exceed:

$$\frac{\sigma_t}{\gamma_m \gamma_{f3}}$$

where

P_t is the externally applied tensile load
H is the prying force determined in accordance with **14.3.6**
A_{et} is the tensile stress area of the bolt given in BS 3692, BS 4190 or BS 4395, as appropriate
σ_t is the lesser of
0.7 × minimum ultimate tensile stress,
and either the yield stress or the stress at permanent set of 0.2 %, as appropriate.

14.5.3.3 *Rivets subjected to axial tension.* The use of rivets in tension should be avoided wherever possible. When unavoidable, the tensile stress:

$$\sigma = \frac{P_t + H}{A_{et}}$$

should not exceed:

$$\frac{\sigma_t}{\gamma_m \gamma_{f3}}$$

where

P_t and H are as defined in **14.5.3.2**
A_{et} is the area of the rivet hole
σ_t $= 0.8\sigma_y$ for normal rivets, or
$= 0.5\sigma_y$ for countersunk rivets
σ_y is the yield stress of the rivet material.

14.5.3.4 *Fasteners subjected to shear only.* In a fastener subjected to shear, the average shear stress:

$$\tau = \frac{V}{nA_{eq}}$$

should not exceed:

$$\frac{\sigma_q}{\gamma_m \gamma_{f3} \sqrt{2}}$$

where

V is the applied load on the fastener
n is the number of shear planes resisting the applied shear
A_{eq} for a bolt is the sectional area of the unthreaded shank, provided the shear plane or planes pass through the unthreaded part, or
$= A_{et}$ if any shear plane passes through the threaded part,
A_{eq} for a rivet is the area of the hole
A_{eq} for a pin is the cross-sectional area of the pin
A_{et} is as defined in **14.5.3.2**
σ_q $= \sigma_y$ for all fasteners except black bolts and hand driven rivets, or
$= 0.85\sigma_y$ for black bolts and hand driven rivets
σ_y is the yield stress of the fastener material.

14.5.3.5 *Fasteners subjected to tension and shear.* Fasteners subjected to coexistent tensile and shear forces should be in accordance with **14.5.3.4** and **14.5.3.2** or **14.5.3.3**, as appropriate, and the tensile stress σ and shear stress τ in combination should be such that:

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

where

σ, σ_t, τ and σ_q are as defined in **14.5.3.2**, **14.5.3.3** and **14.5.3.4**, as appropriate.

14.5.3.6 *Bolts and rivets in bearing.* The bearing pressure σ_b between a fastener and each of the connected parts equal to V/A_{eb}, should not exceed:

$$\frac{k_1 k_2 k_3 k_4 \sigma_y}{\gamma_m \gamma_{f3}}$$

where

V is the load transmitted to each connected part by the fastener
A_{eb} for a bolt is the product of the shank diameter of the bolt and the thickness of each connected part loaded in the same direction, irrespective of the location of the thread
A_{eb} for a rivet is the product of the diameter of the hole and the thickness of each connected part loaded in the same direction
A_{eb} for countersunk bolts or rivets is as stipulated above with half the depth of the countersink deducted from the thickness of the part joined
k_1 = 0.85 for black bolts and hand driven rivets, or
= 1.0 for all other fasteners
k_2 = 2.5 except that when the force transmitted by the connector is towards the edge of the part connected (see figure 47), the following values of k_2 should be taken:
= 2.5 when the edge distance ⩾ $3d$, or
= 1.7 when the edge distance = $1.5d$ (minimum for HSFG bolts), or
= 1.2 when the edge distance = $1.2d$

14.5.2 Edge and end distance

Add at end:

Where any of the above limits are not complied with, the strength of the fastener or plate shall be reduced for assessment purposes. Guidance is given in the accompanying Advice Note.

14.5.3.1 General

Add at end:

Where any of the general or specific requirements of this or any of the following sub-clauses are not met in assessment, due allowance shall be made on the strength of the fasteners. Guidance is given in the accompanying Advice Note. Where black bolts have been used in permanent main structural connections, their assessment shall include a fatigue check, ie generally as Class G detail. Bolts shall be assumed to be black bolts and rivets shall be assumed to be hand driven, unless there is evidence to the contrary.

d is the diameter of the hole, and the edge distance is measured from the centre of the hole.

NOTE. For intermediate values of edge distance, k_2 may be obtained by linear interpolation.

k_3 = 1.2 if the part being checked is enclosed on both faces with the fastener acting in double shear, or
= 0.95 in all other cases

k_4 = 1.0 except when the fasteners are HSFG bolts acting in friction, or
= 1.5 when the fasteners are HSFG bolts acting in friction

σ_y is the yield stress of the fastener material or of the connected part, whichever is the lesser.

For fasteners adjacent to an edge where k_2 is less than 2.5, the reduced capacity applies only to fasteners adjacent to the edge. Subject to the provisions of **14.5.5**, the total bearing capacity of the fasteners in a connection shall be the sum of the full bearing capacities of fasteners away from the edge and the reduced strength of those adjacent to the edge.

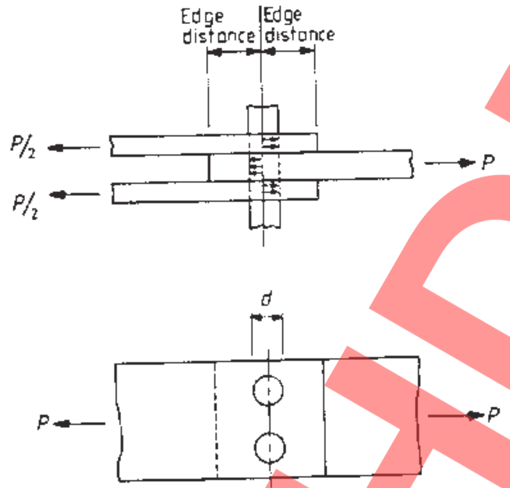

Figure 47. Fastener force towards edge of part

14.5.3.7 *Pins in bearing.* The bearing area of a pin is the product of the pin diameter and the thicknesses of the parts in bearing loaded in the same direction. Consideration should be given to bending stresses in pins. For this purpose, the effective span should be taken as the distance between centres of the appropriate bearing areas. In addition, if the pin passes through bearing plates having a thickness greater than half the diameter of the pin, consideration should be given to the variation of bearing pressure across the thickness of the plate, and the effective span.

The following stresses should not be exceeded.

(a) bending stress in the pin:

$$\frac{1.5\sigma_y}{\gamma_m\gamma_{f3}}$$

where σ_y is the yield stress of the pin material;

(b) bearing pressure where relative rotation occurs between the pin and the connected element:

$$\frac{0.75\sigma_y}{\gamma_m\gamma_{f3}}$$

(c) bearing pressure where such rotation does not occur:

$$\frac{1.5\sigma_y}{\gamma_m\gamma_{f3}}.$$

where σ_y is the yield stress of the pin or of the connected part, whichever is the lesser.

14.5.3.8 *Long grip rivets.* The grip of a rivet should not be greater than eight times the diameter of the hole.

Where the grip length of a rivet exceeds six times the diameter of the hole, the number of rivets calculated to satisfy the provisions of **14.5.3.3** to **14.5.3.6** should be increased by 1% for each additional 2 mm of grip.

14.5.3.9 *Securing nuts.* Wherever there is a risk of nuts becoming loose they should be secured. Nuts of friction grip bolts need not be further secured after tightening.

14.5.3.10 *Bolts and rivets through packings.* The number of bolts or rivets transmitting load in bearing with packings thicker than 6 mm should be increased above the number calculated in accordance with **14.5.3.4** and **14.5.3.6** by 1.25% for each additional millimetre thickness of packing.

For double shear connections with packings on both sides of the splice the number of additional rivets or bolts may be determined from the thickness of the thicker packing.

The additional rivets or bolts may be placed in an extension of the packing.

14.5.3.11 *Pin plates.* Pin plates may be attached to the ends of a pin-connected member in order to ensure that the provisions of **14.5.3.7** are met.

Pin plates should be of sufficient thickness to make up the required bearing or cross-sectional area and should be so arranged as to reduce the eccentricity to a minimum. Their length measured from the centre of the pin to the end (on the reaction side) should be at least equal to their width at the pin, and at least one plate on each side should be as wide as the dimensions will allow. Pin plates should be connected with enough rivets, bolts or welds to transmit the bearing pressure on them and should be so arranged as to distribute it uniformly over the full section of the member (see figure 48).

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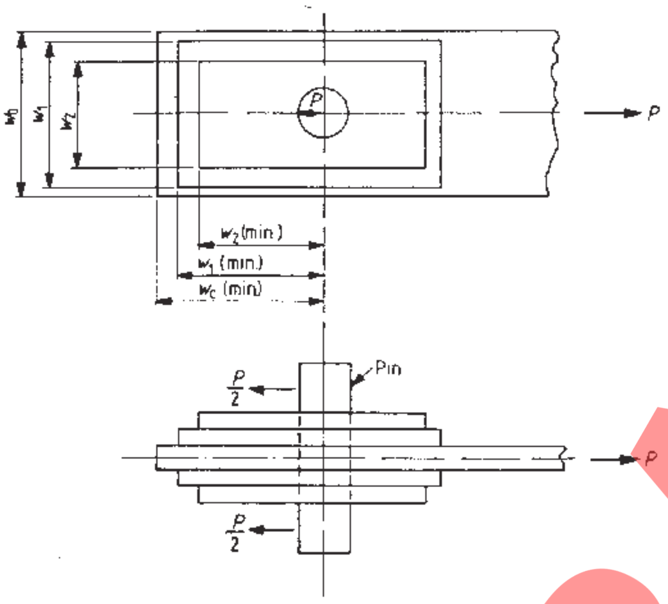


Figure 48. Pin plates

14.5.4 *Strength of HSFG bolts acting in friction*

NOTE. The recommendations given in 14.5.4 apply only to bolts tightened in accordance with the requirements of BS 4604: Parts 1, 2 and 3.

14.5.4.1 *General*

14.5.4.1.1 *Ultimate limit state.* For HSFG bolts in accordance with BS 4395 : Parts 1 and 2 in normal clearance holes, as specified in BS 4604, the design ultimate capacity is the greater of:

(a) the friction capacity determined in accordance with **14.5.4.2** and;

(b) the lesser of either the shear capacity determined in accordance with **14.5.3.4** or the bearing capacity determined in accordance with **14.5.3.6**.

For HSFG bolts in accordance with BS 4395 : Parts 1 and 2 in oversize or slotted holes, and for HSFG bolts in accordance with BS 4395 : Part 3, the ultimate capacity is the friction capacity determined in accordance with **14.5.4.2**.

14.5.4.1.2 *Serviceability limit state.* In a connection made with HSFG bolts in accordance with BS 4395 : Parts 1 and 2 in normal clearance holes, as specified in BS 4604, the serviceability limit state is reached when slip occurs between the parts joined, which should be deemed to occur when the shear load applied to any bolt equals its friction capacity, determined in accordance with **14.5.4.2**.

14.5.4.2 *Friction capacity.* The friction capacity P_D of a HSFG bolt should be taken as:

$$P_D = k_h \frac{F_v \mu N}{\gamma_m \gamma_{f3}}$$

where

F_v is the prestress load, as defined in **14.5.4.3**

μ is the slip factor, having a value in accordance with **14.5.4.4**

N is the number of friction interfaces

k_h = 1.0 where the holes in all the plies are of normal size, as specified in BS 4604, otherwise k_h is as stated in **14.5.4.5**.

14.5.4.3 *Prestress.* The prestress load F_v of a HSFG bolt should be taken as:

$$F_v = F_o - F_t$$

where

F_o is the initial load, i.e. the proof load given in BS 4395, except that for bolts in accordance with Part 2 of that standard it should be 0.85 × the proof load

F_t is the externally applied tensile load if any.

14.5.4.4 *Slip factor.* Unless determined by test, the slip factor μ at friction surfaces should be taken as:

(a) μ = 0.45 for weathered surfaces clear of all mill scale and loose rust;

(b) μ = 0.50 for surfaces blasted with shot or grit and with loose rust removed;

(c) μ = 0.50 for surfaces sprayed with aluminium;

(d) μ = 0.40 for surfaces sprayed with zinc;

(e) μ = 0.35 for surfaces treated with zinc silicate paint;

(f) μ = 0.25 for surfaces treated with etch primer.

The slip factors given in (a) to (f) should be reduced by 10 % where higher grade bolts in accordance with BS 4395 : Part 2 are used.

If the friction surfaces do not conform to any of the above, the characteristic value of the slip factor should be established by test in accordance with BS 4604 or obtained from other reliable sources to the satisfaction of the Engineer.

14.5.4.5 *Over-sized and slotted holes.* Where there are three or more plies, over-sized or slotted holes may be used in the inner plies in accordance with table 12, subject to a reduction of the friction capacity given in **14.5.4.2** by a factor k_h.

where

k_h = 0.85 for over-sized and short slotted holes, or
= 0.70 for long slotted holes.

14.5.4.1 General

Add at end:

Friction grips bolts of types, arrangements or tightening not in accordance with this or any of the following sub clauses shall be assessed by reference to **14.2** and published data relating to the bolt type or by tests on selected bolts in the structure. Guidance is given in the accompanying Advice Note.

Table 12. Over-sized and slotted holes

Diameter of bolt	Maximum size of hole		
	Diameter of over-sized hole	Short-slotted hole	Long-slotted hole
mm	mm	mm	mm
16	21	18 × 22	18 × 40
20	25	22 × 26	22 × 50
22	27	24 × 28	24 × 55
24	30	26 × 32	26 × 60
27	33	30 × 35	30 × 70
30	38	33 × 40	33 × 80
36	44	39 × 46	39 × 90

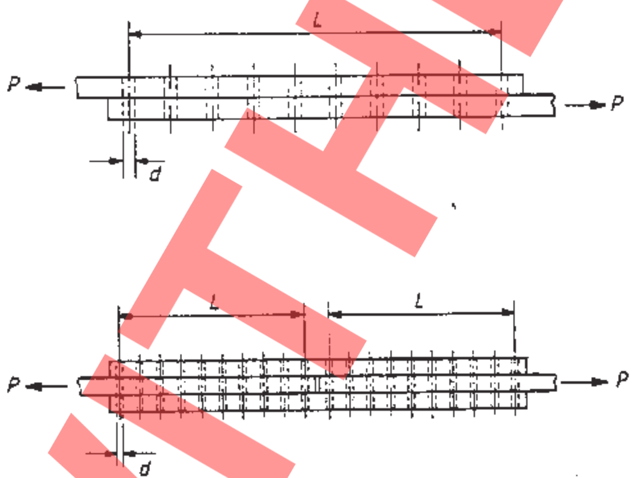
14.5.5 ***Long connections.*** Where the distance L between centres of end fasteners, measured in the direction of the load transmitted, in splices or end connections of tension and compression members (see figure 49) is more than $15d$, the strength of all the fasteners determined in accordance with **14.5.3** or **14.5.4** should be reduced by a multiplying factor:

$$k_5 = 1 - \left(\frac{L - 15d}{200d}\right)$$

but should not be smaller than 0.75; where d is the diameter of the fasteners.

14.6 Welded connections

14.6.1 ***General.*** Welds should be detailed in accordance with the appendices of BS 5135, unless otherwise stipulated in this Part. The design strengths given in **14.6.2.3** and **14.6.3.11** are valid only when the yield stress of the weld metal is at least equal to that of the parent metal. Welds made in accordance with Part 6 may be deemed to satisfy this provision.



NOTE. Long connections are those for which L is greater than $15d$.

Figure 49. Long connections

14.6.2 ***Butt welds***

14.6.2.1 *Intermittent butt welds.* Intermittent butt welds shall not be used.

14.6.2.2 *Partial penetration butt welds.* Partial penetration butt welds should not be used to transmit tensile forces, nor to transmit a bending moment about the longitudinal axis of the weld.

Unless determined by procedure trials, the throat thickness of a partial penetration butt weld should be taken as:

(a) the depth of weld preparation where this is of the J or U type;

(b) the depth of the weld preparation minus 3 mm where the preparation is of the V or bevel type.

Where determined by procedure trials, the throat thickness should not be taken as more than the penetration consistently achieved, ignoring weld reinforcement.

14.6.2.3 *Strength of butt welds.* The strength of a full penetration butt weld should be taken as equal to the strength of the weaker of the parts joined.

The strength of a partial penetration butt weld, together with its reinforcing fillet weld, if any, should be calculated as for a full penetration fillet weld.

14.6.3 ***Fillet welds***

14.6.3.1 *Intermittent fillet welds.* The clear unconnected gap between the ends of the welds, whether in line or staggered, should not be more than 200 mm, and also should not be more than:

(a) 12 times the thickness of the thinner part when the part is in compression;

(b) 16 times the thickness of the thinner part when the part is in tension;

(c) one-quarter of the distance between stiffeners when used to connect stiffeners to a plate or other part subjected to compression or shear.

14.6.1 General

Add at end:

In assessment of bridges known to have been welded in accordance with Part 6 or BS5135: 1974 (or 1981), the strength of the welds shall be determined as given by **14.6.2.3** and **14.6.3.11**. In assessment of bridges not known to have been welded in accordance with Part 6 nor with BS5135: 1984 (or 1981) the strengths of the welds shall be derived in accordance with (a) to (d) as follows.

(a) For butt welds in compression and butt welds in tension or shear demonstrated to comply with BS5135: table 18 quality A, the strengths may be taken as defined in **14.6.2.3.**

(b) For butt welds in tension or shear free from surface cracks but not known to comply with BS5135: table 18 quality A the strengths shall be taken as 85% of those derived from **14.6.2.3.**

(c) For fillet welds in bridges constructed to BS153: Part 1: 1958 or 1972 and free from surface cracks the strengths shall be taken as 90% of those derived from **14.6.3.11** in the absence of demonstration of their compliance with BS5135: Table 19 quality A or equal to those strengths when such compliance has been demonstrated.

(d) For other fillet welds free from visible surface cracks the strengths shall be calculated in accordance with **14.6.3.11**, but replacing $\sigma_w=[1/2\ (\sigma_y + 455)]$ by $0.4(400 + \sigma_{ymin})$ in the absence of demonstration of their compliance with BS5135: Table 19 quality A, or by $0.5(400 + \sigma_{ymin})$ when such compliance has been demonstrated, where σ_{ymin} is the yield stress of the weaker of the parts connected by the welds.

Where any of the general or specific requirements of this or any of the following sub-clauses are not met, due allowance shall be made in the assessment of the strength of welds. Guidance is given in the accompanying Advice Note.

14.6.2.1 Intermittent butt welds

Delete the existing clause and substitute the following:

In the assessment of intermittent butt welds any contribution to strength of the weld at each end of any intermittent length, a length equal to three times the throat thickness shall be ignored.

14.6.2.2 Partial penetration butt welds

Add at end:

The strength of partial penetration butt welds shall be calculated as for fillet welds. For single sided joints where transverse bending causes tension across the root then the yield stress of the weld metal shall be taken as 50% of the weaker of the parts joined in assessing the resistance to transverse bending. However, partial penetration butt welds in non-fatigue prone connections could be assessed by reference to BS5950. Eccentric welds need to be specially considered.

Unless known to have been tested through procedure trials at the time of construction or demonstrated by testing, the throat thickness of a partial penetration butt weld shall be taken as 90% of the nominal.

In a line of intermittent welds there should be a weld at each end of the part connected.

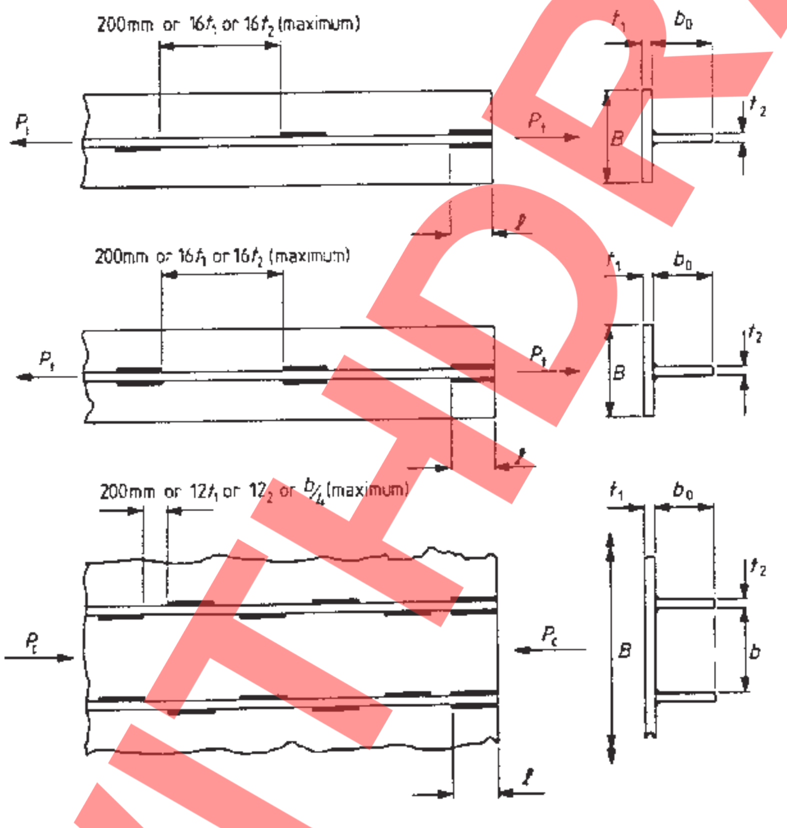
In built-up members in which plates are connected by intermittent welds, continuous side fillet welds should be provided at the ends of each side of the plate for a length at least equal to three-quarters of the width of the narrower plate concerned (see figure 50).

14.6.3.2 *End returns.* A fillet weld should be returned continuously around the corner at the end or the side of a part, for a length beyond the corner of not less than twice the leg length of the weld.

14.6.3.3 *End connections by side fillets.* If the end of a part is connected by side fillet welds only, both sides of the part should be welded and, where possible, the length of weld on each side should be not less than the distance between the welds *b* on the two sides (see figure 51(a)), nor less than four times the thickness of the thinnest part connected. Where the distance between the welds exceeds 16 times the thickness of the thinnest part connected, intermediate plug or slot welds should be provided to prevent separation.

14.6.3.4 *End connections by transverse welds.* The overlap between the connected parts should not be less than four times the thickness of the thinnest part and the parts should be connected by two transverse lines of welds (see figure 51(b)). Where the distance between the welds exceeds 16 times the thickness of the thinnest part connected, intermediate plug or slot welds should be provided to prevent separation.

14.6.3.5 *Welds with packings.* Where two parts connected by welding are separated by packing having a thickness less than the leg length of weld necessary to transmit the force, the required leg length should be increased by the thickness of the packing (see figure 52(a)). The packing should be trimmed flush with the edge of the part which is to be welded. Where two parts connected by welding are separated by packing having a thickness equal to, or greater than, the leg length of weld necessary to transmit the force, each of the parts should be connected to the packing by a weld capable of transmitting the design force (see figure 52(b)).



Note. ℓ should not be shorter than the least of $0.75B$ or $0.75b_0$ or $0.75b$.

Figure 50. Intermittent fillet welds

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14.6.3.6 *Welds in holes and slots.* Fillet welds in holes or slots may be used to transmit shear in lap joints, or to prevent the buckling or separation of lapped parts, or to join the components of built-up members. For the purposes of strength calculations the effective area of such welds should be determined in accordance with **14.6.3.10**.

14.6.3.7 *Longitudinal welds in members subject to bending.* The distribution of the longitudinal shear per unit length of a weld connecting parts of members subjected to bending (e.g. flange and web of beams) should be determined in accordance with linear elastic theory.

14.6.3.8 *Effective length of fillet welds.* When applicable, the effective length of welds should be taken as specified in BS 5135.

The effective length of a continuous weld round the perimeter of a hole or slot should be taken as the length of the centroidal axis of the throat of the weld.

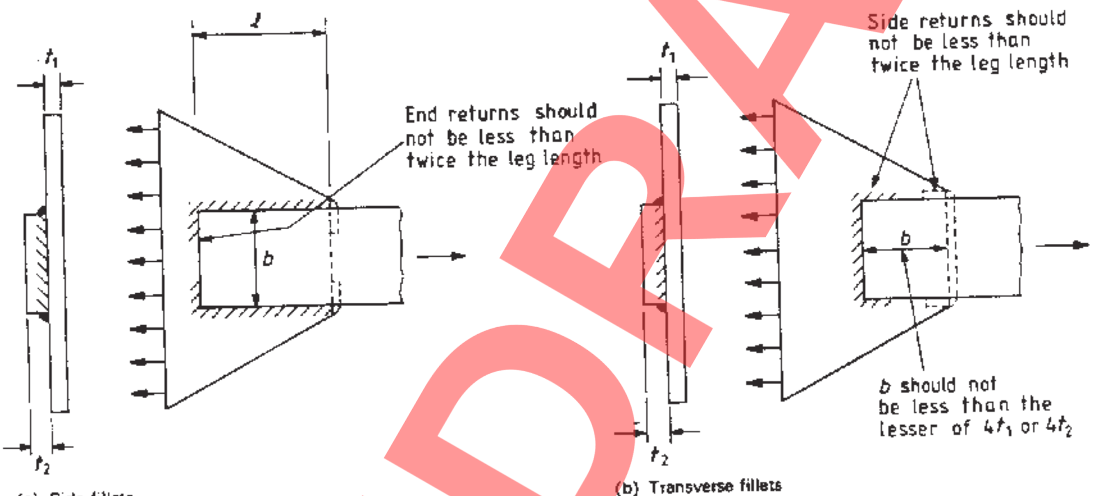
The effective length of welds connecting webs of deep beams (including diaphragms) to other parts of the structure, and of the longitudinal welds in the end connections and splices of axially loaded members, should be taken as $\eta \times L$,

where

η $= 1.10 - (0.05\xi - 0.04)L$, but not more than 1.0

L is the length of the weld in metres or 8 m which ever is lesser

ξ is the ratio of the maximum to the average longitudinal shear stress in the weld. ξ may be taken as 2 unless determined by a linear elastic analysis.



NOTE 1. l should be greater than b when practicable, and should not be less than the lesser of $4t_1$ or $4t_2$.

NOTE 2. Intermediate plug or slot welds are required when b is greater than the lesser of $16t_1$ or $16t_2$.

Figure 51. End connections by fillet welds

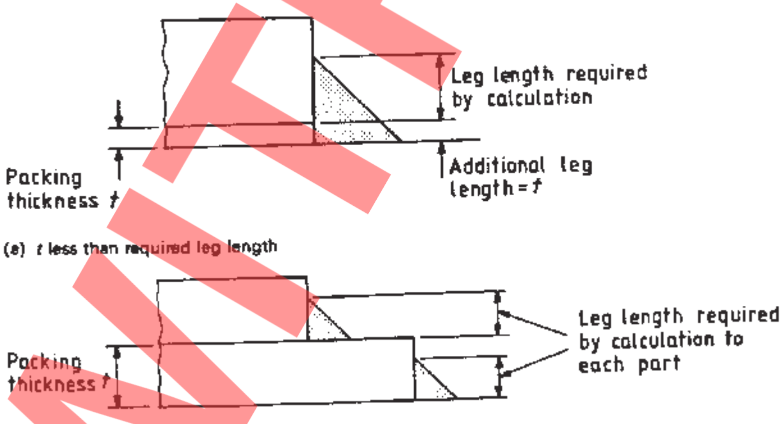


Figure 52. Welds with packings

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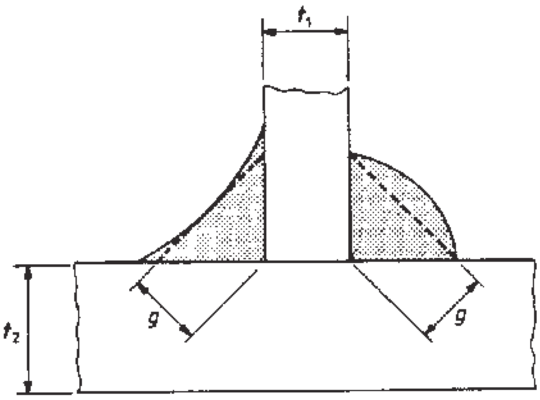
14.6.3.9 *Effective throat of a fillet weld.* The throat of a fillet weld g is the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side (see figure 53).

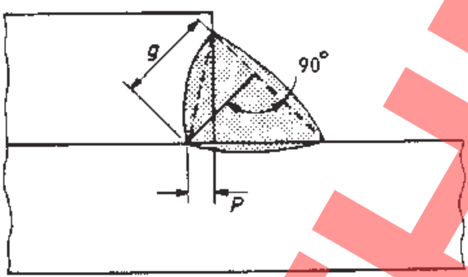
For this purpose the weld should be taken to include any specified penetration, provided that it is shown by procedure trials to the satisfaction of the Engineer that the required penetration can be consistently achieved. In a fillet weld made by the submerged arc process the penetration p may be assumed, without trials, to be 2 mm or $0.2g$ whichever is the lesser (see figure 54).



NOTE 1. g is the effective throat of the weld.

NOTE 2. If either t_1 or t_2 is greater than 4 mm, g has to be at least 3 mm.

Figure 53. Effective throat of fillet weld



NOTE. p is the penetration of the weld
g is the throat of the weld.

Figure 54. Penetration of fillet weld

14.6.3.10 *Effective area of a fillet weld.* The effective area of a fillet weld is its throat dimension multiplied by its effective length, except that for fillet welds in holes or slots the effective area shall not be taken as greater than the area of the hole or slot.

14.6.3.11 *Capacity of a fillet weld*

14.6.3.11.1 *Weld subject to longitudinal shear i.e. shear in the direction of its length (see figure 55(a))*

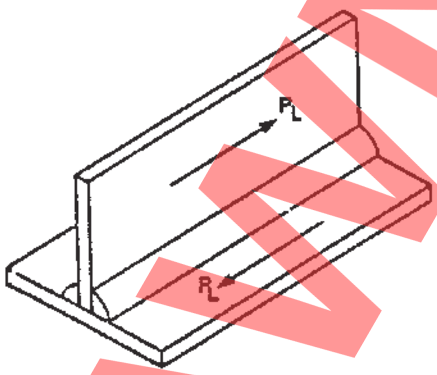


Figure 55(a) Weld subjected to longitudinal shear

The stress in a weld, calculated as the longitudinal shear force per unit length P_L divided by the effective throat g, shall not exceed

$$\frac{\sigma_w}{\gamma_{f3}\,\gamma_m\,\sqrt{3}}$$

where σ_w is the yield stress of the deposited weld metal and may be taken as

$$\tfrac{1}{2}(\sigma_y + 455) \quad \text{N/mm}^2$$

σ_y is the smaller nominal yield stress of the two parts joined.

14.6.3.11.2 *Weld subject to transverse force i.e. force at right angles to its length (see figure 55(b))*

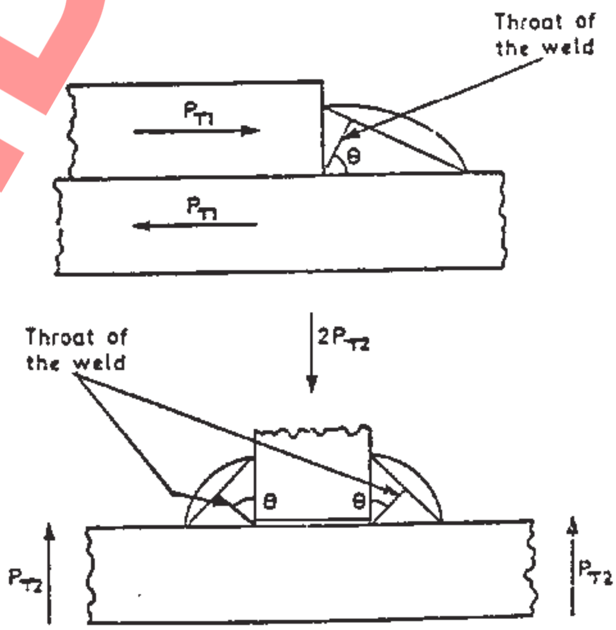


Figure 55(b) Weld subjected to transverse force

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The stress in a weld, calculated as the transverse force per unit length P_{T1} (or P_{T2}) divided by the effective throat g, shall not exceed

$$\frac{K\sigma_w}{\gamma_{f3}\ \gamma_m\ \sqrt{3}}$$

Where σ_w is as defined in 14.6.3.11.1

K depends on the angle θ between the direction of the applied force and the throat and is given by

$$K = \left[\frac{3}{1 + 2\cos^2\theta}\right]^{\frac{1}{2}}$$ but not greater than 1.4

For equal leg fillets between components at right angles θ = 45° and K = 1.225.

14.6.3.11.3. *Weld subject to forces in both transverse and longitudinal directions*

The following condition should be satisfied:-

$$\frac{1}{g}\left[P_L^2 + \frac{P_T^2}{K^2}\right]^{\frac{1}{2}} \leq \frac{\sigma_w}{\gamma_{f3}\ \gamma_m\ \sqrt{3}}$$

where

P_L is the longitudinal shear force per unit length of the weld

P_T is the resultant of transverse forces per unit length of the weld (See Fig 55(c))

g is the effective throat of the weld

$$K = \left[\frac{3}{1 + 2\cos^2\theta}\right]^{\frac{1}{2}}$$ but not greater than 1.4

θ is the angle between the resultant transverse force and the throat

σ_w is as defined in 14.6.3.11.1

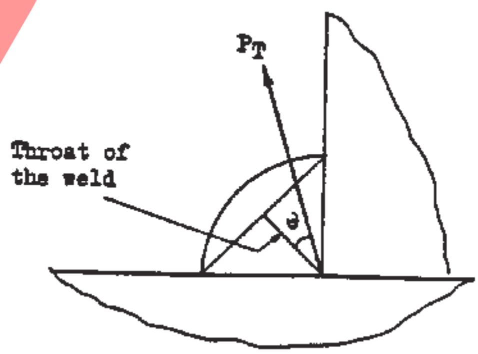

Figure 55(c). Resultant transverse force at weld

14.6.4 ***Plug welds.*** Plug welds should not normally be used for transmitting applied load. Plug welds in circular or slotted holes may be used to prevent the buckling or separation of lapped parts, or to join the components of built-up members. The entire area of the hole or slot should be filled with weld metal having a thickness:

(a) equal to the thickness of the holed or slotted part, where this is 16 mm or less;

(b) in other cases, not less than any of the following:

(1) 16 mm;

(2) 0.45 times the diameter of the hole or the width of the slot;

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(3) one-tenth of the length of the slot;

but need not be greater than the thickness of the holed or slotted part.

The diameter of a hole, or the width of a slot, should not be less than the thickness of the holed or slotted part plus 8 mm.

The distance between centres of holes, or between the centrelines of slots, should not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of slots measured in the direction of their length should not be less than double the length of a slot.

The ends of a slot should be semi-circular, except where the slot terminates at the edges of the part when it can be square.

14.6.5 ***Load transfer by parts in contact.*** Where a good fit is ensured between a flat surface and an edge of a section abutting it, the forces applied to either part transmitted to the other in direct bearing may be taken as follows:

(a) the whole of such forces if the surfaces are machined;

(b) 75 % of such force if the surfaces are sawn or flame cut by machine.

14.7 Hybrid connections

14.7.1 ***Allowable combinations.*** The following combinations of fasteners and welds in a connection may be taken as sharing the loads, transmitted by the connection, proportionally to their respective strength at the ultimate limit state:

(a) rivets, close tolerance bolts and turned barrel bolts when acting in shear or bearing;

(b) welds and HSFG bolts acting in friction, provided that the ultimate capacity for the bolts is in accordance with **14.5.4.2** and that the procedure of making the joint is such that there is no distortion of the faying surfaces. However, the ultimate strength of the connection should not be taken as greater than 90 % of the combined strengths.

14.7.2 ***Other combinations.*** With all other combinations of fasteners and welds in a connection, one type of the fasteners or welds should be assumed to transmit the loads, unless the deformation capacities of the different fasteners or welds have been proved to the satisfaction of the Engineer to be compatible and sufficient to share the loads.

14.8 Lug angles. Lug angles connecting angle members and their fastenings to the gusset or other supporting part, should be designed, in accordance with clauses **10** or **11** as appropriate, to transmit a force 20 % greater than the force in the outstand of the angle connected. The fastenings connecting the lug angle to the outstand of the angle member should be designed to transmit a force 40 % greater than the force in the outstand of the angle member.

Lug angles connecting a channel or similar member, should be disposed symmetrically about the axis of the member, and, together with their fastenings to the gusset or other supporting part, should be designed to transmit a force 10 % greater than the force in the component of the member not directly connected. The fastenings connecting the lug angles to the member should be designed to transmit a force 20 % greater than such excess force.

In no case should less than two bolts or rivets be used to attach a lug angle to a gusset or other supporting part.

The connection of the lug angle to the gusset or other supporting part should terminate at the end of the member connected. The connection of the lug angle to the member should run from the end of the member to a point beyond the direct connection of the member to the gusset or other supporting part.

14.9 Other attachments. The dimensions of any other attachments such as brackets, stools and cleats should be such that:

(a) the maximum equivalent stress does not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_y is the yield stress of the material of the attachment;

(b) their deformation under load is compatible with the distribution of forces assumed in the design of the connection;

(c) buckling does not occur in any component or in a free edge.

Add new Clause 15.

15 Outmoded forms of construction

15.1 General

Some outmoded forms of construction cannot be directly assessed even by the assessment clauses above. This section gives guidance and methods of assessment for certain specific outmoded forms of construction. Any outmoded forms not covered within this section shall be assessed by the relevant section of the standard where possible. Where necessary additional studies, special analyses and tests may be beneficial to the type and form of outmoded construction encountered, to supplement the assessment checks carried out.

15.2 Buckle plates

15.2.1 General

Buckle plates consisting of vertically curved steel plates supporting non-structural filling beneath roadways or footways and spanning between supporting steel members shall be assessed by **15.2.2** or **15.2.3** as appropriate.

15.2.2 Spans of 1.2 m. or less

Where the clear span measured between edges of supporting members is 1.2 m. or less and complies with the following:

(a) rise between $\frac{1}{12}$ th and $\frac{1}{18}$ th of the clear span, and
(b) plate thickness is at least 6 mm,

the strength shall be assessed assuming arch or catenary action where the horizontal thrust may be taken as:

$$\frac{wL^2}{8_r} \text{ per unit width}$$

where

w = pressure on surface of plate due to dead loads and distributed wheel load. The wheel pressure calculated is resumed to occupy the full area of the plate, (see the Advice Note).

L = spans of buckle plate between supporting members

r = rise of buckle plate.

Arched (ie concave downward) buckle plates may be checked as straight compression members in accordance with 10.6 with η taken as a (λ - 15) when calculating the value of σ_c with an appropriate effective length of not less than 0.25L. Suspended (ie concave upward) buckle plates may be checked as tension members under axial load in accordance with **11.5.1**. The fixings and the supporting members shall be capable of resisting the horizontal thrust. Concentrated wheel loads over the plate may be dispersed at 1:1 through solid filling and at 1 (horizontal) to 2 (vertical) through loose filling. Alternatively, the buckle plate may be considered as an encastre flat plate in which case the effects of horizontal thrust may be ignored.

15.2.3 Spans of more than 1.2 m.

When the clear span exceeds 1.2 m. or does otherwise not comply with **15.2.2**, then the strength shall be assessed assuming that the plate behaves as a flat member without benefit from arch action unless testing or other information is made available to demonstrate the strength under traffic loading.

15.3 Joggled stiffeners

15.3.1 Joggled stiffeners acting as transverse web stiffeners other than at supports

Joggled or knee type stiffeners may be considered as transverse web stiffeners. Where an axial force resulting from application of **9.13.3.1** (c), (d), (e) and (f) can be applied to joggled or knee type stiffeners other than within the straight portion between joggles, then the additional bending stress introduced by the shape of the stiffener shall be included within the joggle height when checking yielding of the stiffener under **9.13.5.2**.

(a) For joggled stiffeners the bending stress may be calculated assuming that a bending moment is applied to each stiffener leg equivalent to its axial load multiplied by an eccentricity equal to one half of the joggle offset.

The joggle height over which the bending stress can be included shall be taken as at the level of the joggle and

(3) one-tenth of the length of the slot;

but need not be greater than the thickness of the holed or slotted part.

The diameter of a hole, or the width of a slot, should not be less than the thickness of the holed or slotted part plus 8 mm.

The distance between centres of holes, or between the centrelines of slots, should not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of slots measured in the direction of their length should not be less than double the length of a slot.

The ends of a slot should be semi-circular, except where the slot terminates at the edges of the part when it can be square.

14.6.5 ***Load transfer by parts in contact.*** Where a good fit is ensured between a flat surface and an edge of a section abutting it, the forces applied to either part transmitted to the other in direct bearing may be taken as follows:

(a) the whole of such forces if the surfaces are machined;

(b) 75 % of such force if the surfaces are sawn or flame cut by machine.

14.7 Hybrid connections

14.7.1 ***Allowable combinations.*** The following combinations of fasteners and welds in a connection may be taken as sharing the loads, transmitted by the connection, proportionally to their respective strength at the ultimate limit state:

(a) rivets, close tolerance bolts and turned barrel bolts when acting in shear or bearing;

(b) welds and HSFG bolts acting in friction, provided that the ultimate capacity for the bolts is in accordance with **14.5.4.2** and that the procedure of making the joint is such that there is no distortion of the faying surfaces. However, the ultimate strength of the connection should not be taken as greater than 90% of the combined strengths.

14.7.2 ***Other combinations.*** With all other combinations of fasteners and welds in a connection, one type of the fasteners or welds should be assumed to transmit the loads, unless the deformation capacities of the different fasteners or welds have been proved to the satisfaction of the Engineer to be compatible and sufficient to share the loads.

14.8 Lug angles. Lug angles connecting angle members and their fastenings to the gusset or other supporting part, should be designed, in accordance with clauses **10** or **11** as appropriate, to transmit a force 20 % greater than the force in the outstand of the angle connected. The fastenings connecting the lug angle to the outstand of the angle member should be designed to transmit a force 40 % greater than the force in the outstand of the angle member.

Lug angles connecting a channel or similar member, should be disposed symmetrically about the axis of the member, and, together with their fastenings to the gusset or other supporting part, should be designed to transmit a force 10 % greater than the force in the component of the member not directly connected. The fastenings connecting the lug angles to the member should be designed to transmit a force 20 % greater than such excess force.

In no case should less than two bolts or rivets be used to attach a lug angle to a gusset or other supporting part.

The connection of the lug angle to the gusset or other supporting part should terminate at the end of the member connected. The connection of the lug angle to the member should run from the end of the member to a point beyond the direct connection of the member to the gusset or other supporting part.

14.9 Other attachments. The dimensions of any other attachments such as brackets, stools and cleats should be such that:

(a) the maximum equivalent stress does not exceed:

$$\frac{\sigma_y}{\gamma_m \gamma_{f3}}$$

where

σ_y is the yield stress of the material of the attachment;

(b) their deformation under load is compatible with the distribution of forces assumed in the design of the connection;

(c) buckling does not occur in any component or in a free edge.

extending to the first fastener either side which connects the stiffener to the web.

(b) For knee stiffeners the bending stress may be calculated assuming that a bending moment is applied to each stiffener leg equivalent to its axial load multiplied by an eccentricity equal to one half of the horizontal distance from the centroid of the stiffener to the point of intersection of its flange with the beam flange.

The height over which the bending stress should be included shall be taken as from the flange in contact with the stiffener to the first fastener where the stiffener is connected to the web.

15.3.2 Joggled stiffeners acting as load bearing support stiffeners

Gusseted joggled, or gusseted knee stiffeners may also be considered as load bearing support stiffeners. Joggled or knee type stiffeners as considered are potentially unsuitable, but where they occur then additional bending stress may be introduced due to the shape of the stiffener and shall be assessed when applying **9.14.4.1**. The additional bending stress may be calculated in accordance with **15.3.1.**

15.3.3 Assessment of joggled stiffeners

9.13 and **9.14** have only been addressed above as far as specific criteria for the assessment of joggled stiffeners is concerned. The criteria for assessment of such stiffeners shall use the relevant sub-clauses of **9.13** or **9.14** as appropriate to derive effective sections and loading and assess the strength. In addition, consideration of limitations on shape shall be addressed in accordance with **9.3**. Further guidance on the typical form of joggled stiffeners is given in the Advice Note.

Add new Clause 16.

16 Bearings and bearing areas

16.1 General

Bearings shall be assessed to B.S. 5400 Part 9 where appropriate. Steel bearings of types outside the scope of B.S. 5400 Part 9 shall be assessed using B.S. 5400 Part 3, where Part 9 is not applicable. Due consideration shall be given in allowance for load effects resulting from movement restraint where freedom of the bearing is restricted or impaired.

16.2 Beams without bearings

Where beams do not have discrete bearings and bear directly on concrete, brickwork, or masonry substructures with or without a bearing plate or other distributive layer then the local distribution of load to the substructure shall be assessed taking due account of any rotation or movement. Patch loading to the web of the beam shall be assessed in accordance with **9.9.6** where appropriate.

16.3 Pressure distribution under bearing areas

Where the end of a beam bears directly on a substructure without bearings a linear pressure distribution may be assumed varying from a maximum at the inner face of the contact area down to zero at the far face or free end of the girder. The assumed length of the contact area shall not exceed the length of girder in contact, or the depth of the beam if less. For distribution transversely or in other directions as appropriate, a dispersal angle of 2 horizontal to 1 vertical shall be assumed through any flange angles, flange plate(s) and bearing plate (s) present onto the surface of concrete, brickwork, masonry or other material of the substructure. The effective span of the beam shall be assumed to extend from the centroid of the contact area determined. See further guidance in the Advice Note.

16.4 Maximum pressures under bearing areas

For concrete substructures the compressive stress in the contact area shall not exceed those given by B.S. 5400 Part 4. Where inspection shows no evidence of local spalling, cracking or similar distress beneath bearing areas then the limiting strength in compression for unreinforced concrete may be increased by a maximum of 50%.

For masonry bedstones, stresses shall not exceed 0.4 f_{cu} where f_{cu} is the characteristic compressive strength of the masonry. This value may be increased to 0.6 f_{cu} where inspection shows no evidence of local spalling, cracking or other distress. Bearing loads shall be assumed to disperse at an angle of 1: 1 down to supporting coursed masonry or brickwork.

(3) one-tenth of the length of the slot;

but need not be greater than the thickness of the holed or slotted part.

The diameter of a hole, or the width of a slot, should not be less than the thickness of the holed or slotted part plus 8 mm.

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For concrete substructures the compressive stress in the contact area shall not exceed those given by B.S. 5400 Part 4. Where inspection shows no evidence of local spalling, cracking or similar distress beneath bearing areas then the limiting strength in compression for unreinforced concrete may be increased by a maximum of 50%.

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Appendix A
Evaluation of effective breadth ratios

A.1 General. The methods given in this appendix may be used to determine the effective breadth ratios for conditions not specifically covered by **8.2** or where a special study is warranted. Examples are where point loads of significant magnitude act on a bridge deck, either in isolation or in combination with other loads, and where there are single spans with cantilevered projections continuous over the supports.

The notation used in this appendix is the same as that used in **8.2** except that L may be taken as the distance between adjacent points at which the bending moment is zero.

A.2 Equivalent simply supported spans. In structures other than simply supported beams, the effective breadth ratio ψ may be obtained by treating each portion of a continuous beam between adjacent points of zero moment as an equivalent simply supported span, and by using table 4 or table 13, as appropriate.

The positions of the points of zero moment should be those corresponding to the particular loading under consideration. In the special case of a portion of a span between a fixed end and an adjacent point of zero moment, the equivalent span should be obtained by considering a fictitious symmetrical span extending beyond the fixed end with the loading and reactions applied symmetrically about the fixed end.

A.3 Point loads at mid-span. For point loads and other concentrated loads at mid-span, or at the free end of a cantilever, the effective breadth ratios ψ may be obtained from tables 13, 14, 15 or 16. These tables should only be used for point loads and reactions of significant magnitude and should not be used for standard highway wheel or axle loads.

A.4 Point loads not at mid-span. For point loads on a simply supported beam at positions other than mid-span, the effective breadth ratio ψ under the point of application of the load may be determined from:

$$\psi = 0.33\,(2\psi_x + \psi_{(L-x)})$$

where

ψ_x is the value of ψ from table 13 for a point load at mid-span with $L = 2x$

$\psi_{(L-x)}$ is the value of ψ from table 13 for a point load at mid-span with $L = 2(L-x)$

x is the shorter distance from the end of the span to the point of application of the load.

In the special case of a simply supported beam with $b/L < 0.1$, the effective breadth ratio ψ, under a point load anywhere in the span, may be taken as the effective breadth ratio ψ from table 13 for a point load at mid-span.

The effective breadth ratio at all points in the span or equivalent simply supported span, at a distance of more than $L/4$ from the point load, may be assumed to be the value of ψ given in table 13 at quarter-span. Within a distance $L/4$ of the point load, the effective breadth ratio may be assumed to vary linearly between the value at the load and the value at $L/4$ from the point load.

Where the distance between the point load and the support is less than $L/4$, the effective breadth ratio throughout that distance may be taken as the value under the load point.

A.5 Combination of loads. Under combinations of distributed and/or point loads the values of ψ may be derived from:

$$\psi = \frac{\Sigma M}{\dfrac{M_1}{\psi_1} + \dfrac{M_2}{\psi_2} \dots + \dfrac{M_n}{\psi_n}}$$

where

$M_1 \dots M_n$ are the bending moments at the cross section considered due to each component of load

ΣM is the total bending moment at the same section due to load components 1 to n

$\psi_1 \dots \psi_n$ are the effective breadth ratios for the same section appropriate to each load component, using tables 4 to 7 for distributed loads and tables 13 to 16 for concentrated loads.

NOTE. In calculating the value of ψ due account should be taken of the algebraic sign of the bending moments.

A.6 Transverse distribution of stress. The longitudinal stress σ_1 at any point in the flange at a distance x from the centreline of the web may be calculated from:

$$\sigma_1 = \sigma_{max}[\chi^4 + k(1 - \chi^4)]$$

where

$\chi = \left(\dfrac{b-x}{b}\right)$

k $= 0.25\,(5\psi - 1)$ for portions between web centrelines, or

$= 0.25\,[5(1 - 0.15\,\dfrac{b}{L})\psi - 1]$ for portions projecting beyond an outer web

σ_{max} is the maximum stress in the flange due to longitudinal bending of the section, calculated by elastic analysis using the effective flange breadth determined in accordance with **8.2**

ψ, b and L are as defined in **8.2**.

x is as given in figure 56.

NOTE. If the calculated value of σ_1 is negative it should be taken as zero.

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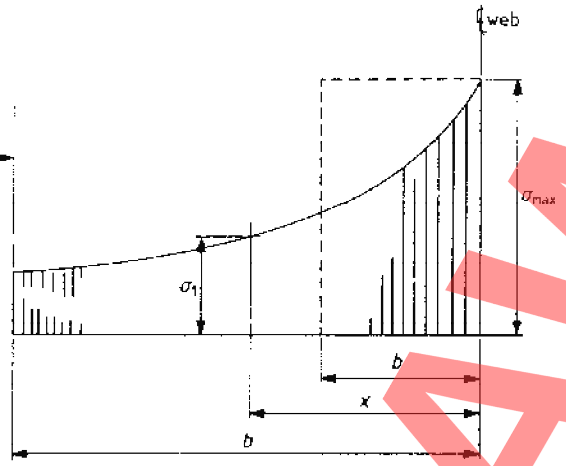

Figure 56. Distribution of longitudinal stress in the flange of a beam

Table 13. Effective breadth ratio ψ for simply supported beams for point load at mid-span

$\frac{b}{L}$	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.80	0.75	1.00	1.00	1.00	1.00
0.10	0.67	0.59	1.00	0.99	1.00	1.00
0.20	0.49	0.40	0.98	0.84	1.00	0.93
0.30	0.38	0.30	0.80	0.61	0.87	0.69
0.40	0.30	0.23	0.63	0.44	0.70	0.51
0.50	0.24	0.17	0.48	0.32	0.54	0.37
0.75	0.15	0.10	0.25	0.19	0.31	0.22
1.00	0.12	0.08	0.19	0.14	0.20	0.15

Table 14. Effective breadth ratio ψ for interior spans of continuous beams for point load at mid-span

$\frac{b}{l}$	Mid-span		Quarter-span		Support	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.67	0.59	1.00	1.00	0.67	0.59
0.10	0.49	0.40	1.00	0.93	0.49	0.40
0.20	0.30	0.23	0.70	0.51	0.30	0.23
0.30	0.19	0.14	0.42	0.29	0.19	0.14
0.40	0.14	0.10	0.28	0.20	0.14	0.10
0.50	0.12	0.08	0.20	0.15	0.12	0.08
0.75	0.09	0.06	0.08	0.07	0.09	0.06
1.00	0.08	0.04	0.02	0.05	0.08	0.04

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Table 15. Effective breadth ratio ψ for propped cantilever beams for point load at mid-span

$\frac{b}{L}$	Fixed-end		Quarter-span near fixed end*		Mid-span	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.68	0.61	1.00	1.00	0.69	0.62
0.10	0.51	0.42	1.00	0.98	0.53	0.44
0.20	0.33	0.25	0.77	0.58	0.34	0.27
0.30	0.21	0.15	0.48	0.32	0.23	0.18
0.40	0.15	0.11	0.32	0.23	0.19	0.14
0.50	0.12	0.08	0.23	0.17	0.15	0.10
0.75	0.10	0.06	0.10	0.09	0.10	0.07
1.00	0.08	0.05	0.05	0.04	0.07	0.05

* May also be used for propped end.

Table 16. Effective breadth ratio ψ for cantilever beams for point load at free-end

$\frac{b}{L}$	Fixed-end		Mid-span		Free-end	
	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$	$\alpha = 0$	$\alpha = 1$
0.00	1.00	1.00	1.00	1.00	1.00	1.00
0.05	0.89	0.86	1.00	1.00	1.00	1.00
0.10	0.80	0.75	1.00	1.00	1.00	1.00
0.20	0.67	0.59	1.00	0.99	1.00	1.00
0.30	0.56	0.47	1.00	0.94	1.00	1.00
0.40	0.49	0.40	0.98	0.84	1.00	0.93
0.50	0.43	0.35	0.88	0.71	0.97	0.81
0.75	0.32	0.25	0.66	0.47	0.74	0.55
1.00	0.24	0.17	0.48	0.33	0.54	0.37

Appendix B
Distortion and warping stresses in box girders

B.1 General. When a highway bridge is subject to live loading, as specified in Part 2, and comprises one or more single-cell box girders, the simplified procedure given in **B.2** to **B.4** may be used to calculate transverse and longitudinal stresses due to restraint of warping. This procedure may also be used for multi-cell box girders, provided that interior webs are ignored for this purpose.

B.2 Restraint of torsional warping. When an increment of torque T is applied at a section of a box girder (other than at a free end) the resulting maximum longitudinal stress at this section due to restraint of torsional warping σ_{TW} may be calculated as follows.

(a) *Stresses at the junction between the bottom flange and web.* At the junction between the bottom flange and a web, at the section where an increment of torque T is applied:

$$\sigma_{TWB} = \frac{DT}{J}$$

where

D is the depth of the box at its centreline, measured between centres of flange plates, or, in composite construction, between the effective centroid of the composite top flange and the centre of the bottom flange plate

J is the torsional constant $4A_o^2/\Sigma(B/t)$

A_o is the area enclosed by the median line of the perimeter material of the section

B and t are the width and thickness, respectively, of each wall of the section forming the closed perimeter.

NOTE. In the case of a wall made from material other than steel, t should be taken as the actual thickness multiplied by the ratio of the shear modulus of the material used to the shear modulus of steel. Where the shear modulus varies with the load history, the long term value should be used.

(b) *Stresses at the junction between the top flange and web.* At the junction between the top flange and a web, at the section where an increment of torque T is applied:

$$\sigma_{TWT} = \left(\frac{B_B}{B_T}\right)^2 \frac{DT}{J\left(1 + \frac{2B_c}{B_T}\right)^3}$$

where

B_c is the width of flange projection beyond the centre of the web

B_B, B_T are the widths of the bottom and top flanges, respectively, measured between centres of webs

D and J are as defined in (a).

NOTE. When there are two or more box girders in a single structure, σ_{TWT} may be taken as zero.

B.2 Restraint of torsional warping

In the definition for J, delete "B" and replace by "W".

In the definition for B, delete "B" and replace by "W".

(c) *Distribution of stresses.* At a distance x from the section where an increment of torque is applied:

$$\sigma_{TWx} = \sigma_{TW} e^{-(2x/B_B)}$$

The distribution across the section of the longitudinal stress due to restraint of torsional warping may be assumed to be as shown in figure 57.

B.3 Restraint of distortional warping

B.3.1 ***General.*** When torque is applied to a box girder other than at a cross frame or a diaphragm, the resulting longitudinal stress due to restraint of distortional warping σ_{DW} may be calculated in accordance with **B.3.2**, provided that the cross frames or diaphragms are in accordance with **B.3.4**.

B.3.2 Corner stresses. The distortional warping stress σ_{DW} may be calculated as follows.

(a) At a junction between a flange and a web, under a uniformly distributed applied torque T_{UD} per unit length of span:

$$\sigma_{DW} = \frac{T_{UD}\bar{y}L_D{}^2}{4.5 B_T I_x} \quad \text{when } \beta L_D < 1.6$$

$$\sigma_{DW} = 0.6 \frac{T_{UD}\bar{y}L_D{}^2}{(\beta L_D)^2 B_T I_x} \quad \text{when } \beta L_D \geqslant 1.6$$

where

$\bar{y}$ is the distance from the horizontal neutral axis to the flange/web junction

I_x is the second moment of area of the girder, inclusive of its effective flanges, about the horizontal neutral axis

L_D is the spacing of cross frames or diaphragms in accordance with **B.3.4**

B_T is as defined in **B.2**(b)

$$\beta L_D = \left(\frac{K L_D{}^4}{E I_x}\right)^{0.25}$$

$$K = \frac{24 D_{YT} R_D}{B_T{}^3}$$

D_{YT} is the transverse flexural rigidity (EI) of the top flange, including transverse stiffeners if any, per unit length of span

R_D may be obtained from figure 58(a) for a rectangular box, and from figures 58(b) to (d) for a trapezoidal box with webs inclined at 30° from the vertical. Values of R_D for trapezoidal boxes in which the webs are inclined at less than 30° from the vertical may be obtained by linear interpolation

In figure 58,

$$\phi_T = \frac{d}{B_T}$$

D_{YB} is the transverse flexural rigidity (EI) of the bottom flange, including transverse stiffeners, if any, per unit length of span

d is the clear depth of web measured in the plane of the web, or, if corner stiffening is provided, the distance between centres of connections of such stiffening to the web

D_{YC} is the transverse flexural rigidity (EI) of the web, including its transverse stiffeners, if any, per unit length of span.

(b) At a junction between a flange and a web, due to a concentrated applied torque T where an axle or knife-edge load is applied mid-way between diaphragms:

$$\sigma_{DW} = \frac{T\bar{y}L_D}{B_T I_x} \quad \text{when } \beta L_D \leqslant 1.0$$

$$\sigma_{DW} = \frac{T\bar{y}L_D}{(\beta L_D) I_x B_T} \quad \text{when } \beta L_D > 1.0$$

(c) Under HA loading, the effects of the uniformly distributed and the knife-edge load should be separately calculated as described in (a) and (b), and the sum of the resulting stresses taken.

(d) Under a series of concentrated torques due to axle loads:

$$\Sigma \sigma_{DW} = \sigma_{DW1} \Sigma (C_{\beta x})$$

where

σ_{DW1} is the value of σ_{DW} obtained under (b) above for a unit axle load

$C_{\beta x}$ $= P_n(\cos \beta x - \sin \beta x) e^{-\beta x}$

P_n is the load on an axle at distance x from the mid-point between diaphragms

$$\beta x = \left(\frac{K x^4}{E I_x}\right)^{0.25}$$

K is as defined in (a).

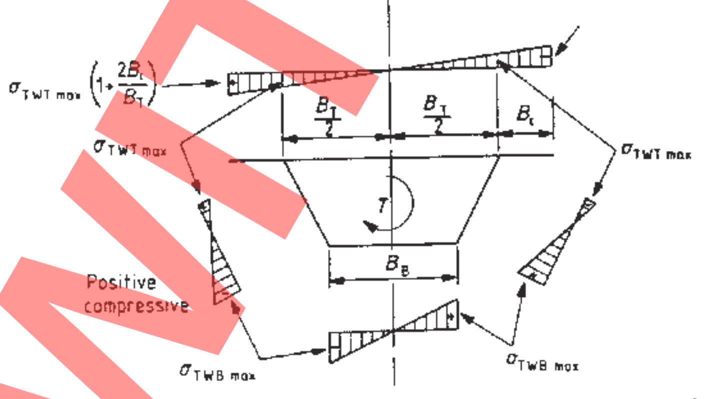

Figure 57. Longitudinal stresses due to restraint of torsional warping

B.3.2 Corner stresses

Add the following at the end of the definition for R_D:

"or from

$$R_D = \frac{1 + \dfrac{B_T}{B_B}}{\dfrac{D_{YT}}{D_{YC}} \dfrac{d}{B_T} \left(\dfrac{2B_B}{B_T + B_B} \right) + \left(\dfrac{B_B}{B_T + B_B} \right) - V_D \left[\left(2 + \dfrac{B_B}{B_T} \right) \dfrac{D_{YT}}{D_{YC}} \dfrac{d}{B_T} + 1 \right]}$$

where V_D is as defined in **B.42**"

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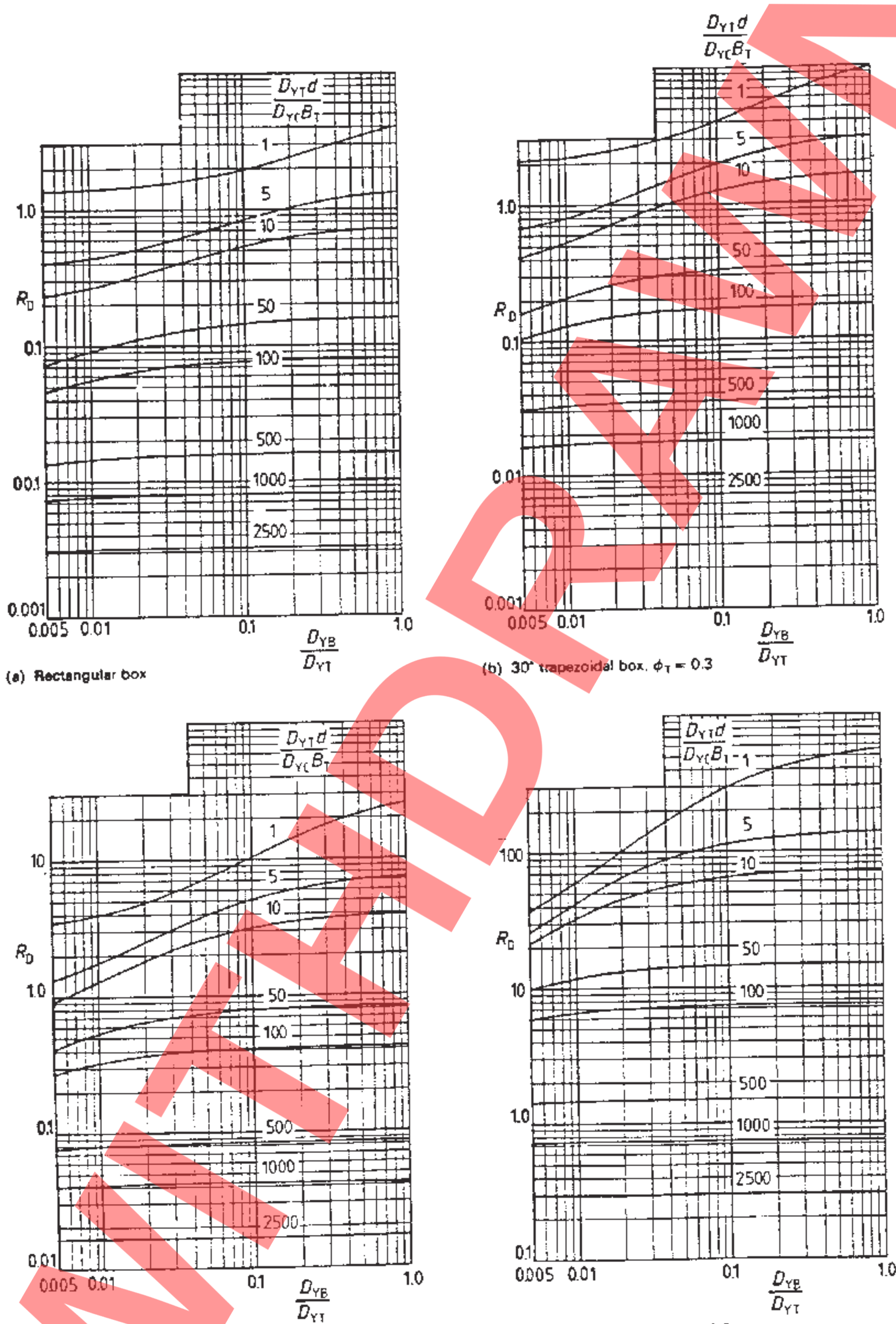

(a) Rectangular box

(b) 30° trapezoidal box, $\phi_T = 0.3$

(c) 30° trapezoidal box, $\phi_T = 0.5$

(d) 30° trapezoidal box, $\phi_T = 0.8$

NOTE. For basis of curves see G.17.

Figure 58. Distortional warping stress parameters

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B.3.3 *Distribution of distortional warping stress*

Stress due to restraint of distortional warping should be assumed to be distributed over the cross section as shown in figure 59.

In figure 59, B_c is the width of the flange projection beyond the web, or, where there are two or more boxes in one structure, half the clear width of flange between boxes.

B.3.4 *Effective diaphragm*

B.3.4.1 *General.* To be effective for the purposes of this clause, a cross frame or a diaphragm should be such as to satisfy the conditions given in **B.3.4.2** and **B.3.4.3**, where the load effects mentioned should be considered as acting in combination with all other simultaneously acting loading effects.

B.3.4.2 *Strength*

(a) A plate diaphragm should be capable of resisting a shear stress τ_D, due to the applied torque T, given by:

$$\tau_D = \frac{T}{D(B_T + B_B)t_d}$$

where

t_d is the thickness of the diaphragm plate

B_T, B_B, D are as defined in **B.2**.

(b) A cross frame consisting of a pair of cross braces connecting both pairs of opposite corners of the box, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, given by:

$$F_B = \frac{T\sqrt{1 + \left(\dfrac{B_T + B_B}{2D}\right)^2}}{2\left(1 + \dfrac{B_T}{B_B}\right)B_T}$$

where

T is as defined in (a)

B_T, B_B and D are as defined in **B.2**.

(c) A V-braced cross frame with the V centred in the top or bottom flange, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, as given in (b).

B.3.4.3 *Stiffness.* Where the effect of distortional warping is to be considered, a cross frame or a diaphragm should have a dimensionless stiffness S not less than the value obtained from table 17.

where

$$S = \frac{Gt_d L_p^2 \delta_b^2 K}{2A_p L_D} \quad \text{for a plated diaphragm, or}$$

$$S = \frac{EA_b \delta_b^2 K}{L_b L_D} \quad \text{for a cross braced cross frame, or}$$

$$S = \frac{EA_b \delta_b^2 K}{4L_b L_D} \quad \text{for a vee braced cross frame, or}$$

$$S = 1.0 \quad \text{for a ring cross frame}$$

$$L_p = \sqrt{D^2 + \left(\frac{B_T + B_B}{2}\right)^2} \quad \text{is the length of the diagonal}$$

L_b is the length of the brace

A_p is the surface area of the plated diaphragm $= D(B_T + B_B)/2$

A_b is the area of cross section of the brace

$$\delta_b = \frac{\dfrac{2}{K}\left(1 + \dfrac{B_T}{B_B}\right)}{\sqrt{1 + \left(\dfrac{B_T + B_B}{2D}\right)^2}}$$

L_D and K are as defined in **B.3.2**

B_T, B_B and D are as defined in **B.2.**

Table 17. Diaphragm stiffness S (see B.3.4.3)

βL_D	Single torque		Uniformly distributed torque	
	For σ_{DW}	For σ_{DB}	For σ_{DW}	For σ_{DB}
3.0	-	1	-	1
2.0	-	5	-	20
1.0	10	50	100	500
0.8	50	100	200	1 000
0.5	500	1 000	200	10 000
0.3	2 000	10 000	200	20 000

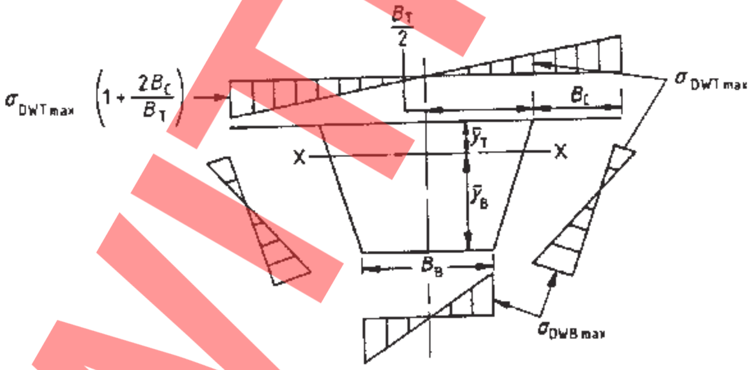

Figure 59. Longitudinal stresses due to distortional warping

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B.3.4.2 Strength

Delete the whole clause and substitute the following:

“(a) A plate diaphragm should be capable of resisting a shear stress τ_D given by:

$$\tau_D = \frac{T}{2BDt_d}$$

where

t_d	is the thickness of the diaphragm plate
B	is as defined in **9.17.2.7**, ie the average of the widths at the top and bottom flanges
D	is as defined in **B.2**.
T	is the torque due to loads applied to the deck at or adjacent to a diaphragm against which the diaphragm provides distortional restraint. Any such torque due to loads applied between diaphragms should be apportioned to adjacent diaphragms by elastic analysis.

(b) A cross frame consisting of a pair of cross braces connecting both pairs of opposite corners of the box, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, given by:

$$F_B = \frac{TL_p\ B_B}{4BD\ B_T}$$

where

T, B and D are as defined in (a)

B_B and B_T are as defined in **B.2**

L_p is as defined in **B.3.4.3**.

(c) A V-braced cross frame with the V centred in the top or bottom flange, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, as given by:

$$F_B = \frac{TL_b\ B_B}{2BD\ B_T}$$

where

T, B and D are as defined in (a)

L_b is as defined in **B.3.4.3**.”

B_B and B_T are as defined in **B.2.**

3.4.3 Stiffness

Delete the whole clause and substitute the following:

"Where the effect of distortional warping is to be considered, a cross frame or a diaphragm should have dimensionless stiffness S not less than the value tained from table 17.

where

$$S = \frac{Gt_d L_p^2 \delta_b^2 K}{2A_p L_D}$$ for plated diaphragm,

or

$$S = \frac{EA_b \delta_b^2 K}{L_D L_P}$$ for a cross braced cross frame

or

$$S = \frac{EA_b L_p^2 \delta_b^2 K}{4L_D\ L_b^3}$$ for a vee braced cross frame irrespective of whether the centre of the V is at the top or bottom flange, alternatively

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B.3.3 *Distribution of distortional warping stress*
Stress due to restraint of distortional warping should be assumed to be distributed over the cross section as shown in figure 59.

In figure 59, B_c is the width of the flange projection beyond the web, or, where there are two or more boxes in one structure, half the clear width of flange between boxes.

B.3.4 *Effective diaphragm*

B.3.4.1 *General.* To be effective for the purposes of this clause, a cross frame or a diaphragm should be such as to satisfy the conditions given in **B.3.4.2** and **B.3.4.3**, where the load effects mentioned should be considered as acting in combination with all other simultaneously acting loading effects.

B.3.4.2 *Strength*

(a) A plate diaphragm should be capable of resisting a shear stress τ_D, due to the applied torque T, given by:

$$\tau_D = \frac{T}{D(B_T + B_B)t_d}$$

where

t_d is the thickness of the diaphragm plate
B_T, B_B, D are as defined in **B.2**.

(b) A cross frame consisting of a pair of cross braces connecting both pairs of opposite corners of the box, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, given by:

$$F_B = \frac{T\sqrt{1 + \left(\frac{B_T + B_B}{2D}\right)^2}}{2\left(1 + \frac{B_T}{B_B}\right)B_T}$$

where

T is as defined in (a)
B_T, B_B and D are as defined in **B.2**.

(c) A V-braced cross frame with the V centred in the top or bottom flange, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, as given in (b).

B.3.4.3 *Stiffness.* Where the effect of distortional warping is to be considered, a cross frame or a diaphragm should have a dimensionless stiffness S not less than the value obtained from table 17.

where

$$S = \frac{Gt_d L_p{}^2 \delta_b{}^2 K}{2A_p L_D} \quad \text{for a plated diaphragm, or}$$

$$S = \frac{EA_b \delta_b{}^2 K}{L_b L_D} \quad \text{for a cross braced cross frame, or}$$

$$S = \frac{EA_b \delta_b{}^2 K}{4L_b L_D} \quad \text{for a vee braced cross frame, or}$$

$$S = 1.0 \quad \text{for a ring cross frame}$$

$$L_p = \sqrt{D^2 + \left(\frac{B_T + B_B}{2}\right)^2} \quad \text{is the length of the diagonal}$$

L_b is the length of the brace
A_p is the surface area of the plated diaphragm $= D(B_T + B_B)/2$
A_b is the area of cross section of the brace

$$\delta_b = \frac{\frac{2}{K}\left(1 + \frac{B_T}{B_B}\right)}{\sqrt{1 + \left(\frac{B_T + B_B}{2D}\right)^2}}$$

L_D and K are as defined in **B.3.2**
B_T, B_B and D are as defined in **B.2**.

Table 17. Diaphragm stiffness S (see B.3.4.3)

βL_D	Single torque		Uniformly distributed torque	
	For σ_{DW}	For σ_{DB}	For σ_{DW}	For σ_{DB}
3.0	-	1	-	1
2.0	-	5	-	20
1.0	10	50	100	500
0.8	50	100	200	1 000
0.5	500	1 000	200	10 000
0.3	2 000	10 000	200	20 000

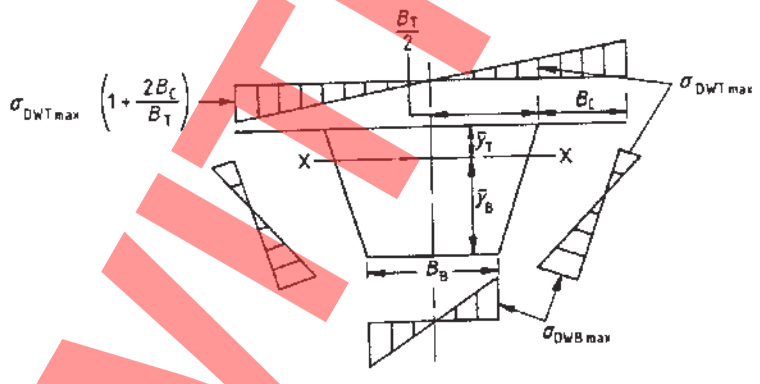

Figure 59. Longitudinal stresses due to distortional warping

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$$S = \frac{K_R}{KL_D}$$ for an unbraced ring cross frame with constant-section framing members

$$S = \frac{P_p \delta_b^2 K}{\Delta_p L_D}$$ for any type of cross frame including a ring cross frame

Δ_p is the change in length of the diagonal L_p calculated to occur under the system of diagonal forces P_p as shown in the figure below. This method of deriving stiffness may be used for any type of frame including those given above.

L_p = √ ($D^2 + B^2$) is the length of the diagonal

L_b is the length of the brace

A_p = BD is the surface area of the plated diaphragm

A_b is the area of cross section of brace

$$\delta_b = \frac{4BD}{KB_B L_p}$$ is a unit length flexibility

B = $(B_T + B_B)/2$ is the average width of the box girder

L_D and K are as defined in **B.3.2**

K_R is the value of K derived by taking D_{YT}, D_{YB} and D_{YC} as the flexural rigidities of the effective framing members attached to the top and bottom flanges and webs respectively and d equal to the distance between the centroids of the effective sections of the top and bottom framing members

B_T B_D and D are as defined in **B.2**."

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B.3.3 *Distribution of distortional warping stress*
Stress due to restraint of distortional warping should be assumed to be distributed over the cross section as shown in figure 59.

In figure 59, B_c is the width of the flange projection beyond the web, or, where there are two or more boxes in one structure, half the clear width of flange between boxes.

B.3.4 *Effective diaphragm*

B.3.4.1 *General.* To be effective for the purposes of this clause, a cross frame or a diaphragm should be such as to satisfy the conditions given in **B.3.4.2** and **B.3.4.3**, where the load effects mentioned should be considered as acting in combination with all other simultaneously acting loading effects.

B.3.4.2 *Strength*

(a) A plate diaphragm should be capable of resisting a shear stress τ_D, due to the applied torque T, given by:

$$\tau_D = \frac{T}{D(B_T + B_B)t_d}$$

where

t_d is the thickness of the diaphragm plate
B_T, B_B, D are as defined in **B.2**.

(b) A cross frame consisting of a pair of cross braces connecting both pairs of opposite corners of the box, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, given by:

$$F_B = \frac{T\sqrt{1 + \left(\frac{B_T + B_B}{2D}\right)^2}}{2\left(1 + \frac{B_T}{B_B}\right)B_T}$$

where

T is as defined in (a)
B_T, B_B and D are as defined in **B.2**.

(c) A V-braced cross frame with the V centred in the top or bottom flange, in which both braces are considered to be simultaneously effective, should be capable of carrying a force F_B in each brace, as given in (b).

B.3.4.3 *Stiffness.* Where the effect of distortional warping is to be considered, a cross frame or a diaphragm should have a dimensionless stiffness S not less than the value obtained from table 17.

where

$$S = \frac{G t_d L_p^2 \delta_b^2 K}{2 A_p L_D} \quad \text{for a plated diaphragm, or}$$

$$S = \frac{E A_b \delta_b^2 K}{L_b L_D} \quad \text{for a cross braced cross frame, or}$$

$$S = \frac{E A_b \delta_b^2 K}{4 L_b L_D} \quad \text{for a vee braced cross frame, or}$$

$$S = 1.0 \quad \text{for a ring cross frame}$$

$$L_p = \sqrt{D^2 + \left(\frac{B_T + B_B}{2}\right)^2} \quad \text{is the length of the diagonal}$$

L_b is the length of the brace
A_p is the surface area of the plated diaphragm $= D(B_T + B_B)/2$
A_b is the area of cross section of the brace

$$\delta_b = \frac{\frac{2}{K}\left(1 + \frac{B_T}{B_B}\right)}{\sqrt{1 + \left(\frac{B_T + B_B}{2D}\right)^2}}$$

L_D and K are as defined in **B.3.2**
B_T, B_B and D are as defined in **B.2**.

Table 17. Diaphragm stiffness S (see B.3.4.3)

	Single torque		Uniformly distributed torque	
βL_D	For σ_{DW}	For σ_{DB}	For σ_{DW}	For σ_{DB}
3.0	-	1	-	1
2.0	-	5	-	20
1.0	10	50	100	500
0.8	50	100	200	1 000
0.5	500	1 000	200	10 000
0.3	2 000	10 000	200	20 000

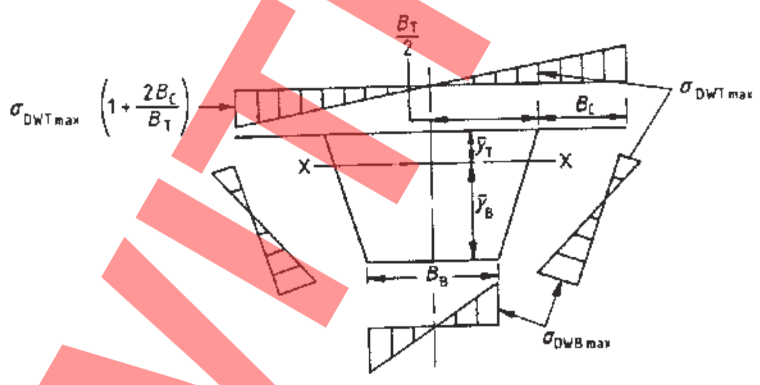

Figure 59. Longitudinal stresses due to distortional warping

Table 17 Diaphragm stiffness S

Delete the existing table and substitute the following table:

βL_D	Single torque		Uniformly distributed torque	
	For σ_{DW}	For σ_{DB}	For σ_{DW}	For σ_{DB}
2.65	0	0	0	0
2.0	0	5	0	20
1.6	0	10	0	70
1.0	10	50	100	500
0.8	50	100	200	1 000
0.5	500	1 000	200	10 000
0.3	2 000	10 000	200	20 0000

Note: For intermediate values of βL_D values of S may be obtained by logarithmic interpolation.

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B.4 Transverse distortional bending stresses

B.4.1 ***General.*** When loads are applied to a box girder other than at a cross frame or a diaphragm, the resulting transverse bending stresses in the walls of a box should be calculated on the basis of linear elastic theory. The simplified procedure given in **B.4.2** may be used provided that the cross frames or diaphragms are in accordance with **B.3.4**.

B.4.2 ***Corner stresses.*** The transverse bending stress σ_{DB} may be calculated as follows:

(a) At a junction between a transverse stiffener on a web and a transverse stiffener or cross beam on a flange, under a uniformly distributed torque T_{UDL}:

$$\sigma_{DB} = \frac{T_{UDL}F_D}{B_T Z} \quad \text{when } \beta L_D > 2.65$$

$$\sigma_{DB} = \frac{T_{UDL}F_D}{2B_T Z}\left(\frac{\beta L_D}{2.2}\right)^{3.7} \quad \text{when } \beta L_D \leqslant 2.65$$

where

Z is the elastic section modulus, per unit of span length, of the flange or web inclusive of transverse stiffeners or cross beams

σ_{DB} is the maximum distortional bending stress in the part to which Z refers

$$F_D = \frac{B_T}{2}\left(\frac{B_B}{B_T + B_B} - V_D\right) \text{ at a top flange junction, or}$$

$$= B_B \frac{V_D}{2} \text{ at a bottom flange junction}$$

βL_D is as defined in **B.3.2**

V_D is to be obtained from figure 60,

ϕ_T, D_{YB}, D_{YC}, D_{YT}, B_B and B_T are as defined in **B.3.2**.

(b) At a junction between a transverse stiffener on a web and a transverse stiffener or cross beam on a flange, due to a concentrated torque T where an axle or knife-edge load is applied mid-way between diaphragms:

$$\sigma_{DB} = \frac{TF_D}{2B_T L_D Z}\beta L_D \text{ when } \beta L_D > 2.$$

$$\sigma_{DB} = \frac{TF_D}{15.5B_T L_D Z}(\beta L_D)^{3.9} \text{ when } \beta L_D \leqslant 2.$$

where all symbols are as defined in (a).

(c) Under HA loading, the effects of the uniformly distributed and the knife-edge load should be separately calculated as described in (a) and (b), and the sum of the resulting stresses used.

(d) Under a series of concentrated torques due to axle loads:

$$\Sigma\sigma_{DB} = \sigma_{DB1}\Sigma(P_{\beta x})$$

where

σ_{DB1} is the value of σ_{DB} obtained from (b) for a unit axle load

$P_{\beta x} = P_n(\cos \beta x + \sin \beta x)e^{-\beta x}$

P_n and βx are as defined in **B.3.2** (d).

B.4.3 ***Distribution of distortional bending stress*** Transverse distortional bending moments should be assumed to be distributed over the cross section, as shown in figure 61, and the resulting stresses calculated using the appropriate value of Z at each section.

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B.4.2 Corner stresses

Add the following at the end of the definition for V_D:

"or from

$$V_D = \frac{\left[\frac{D_{YT}}{D_{YC}}\frac{d}{B_T}\left(2+\frac{B_B}{B_T}\right)+1\right]}{\left(\frac{B_T}{B_B}+1\right)\left[1+2\left[\frac{D_{YT}}{D_{YC}}\frac{d}{B_T}\left(1+\frac{B_B}{B_T}+\left(\frac{B_B}{B_T}\right)^2\right)\right]+\frac{D_{YT}}{D_{YB}}\left(\frac{B_B}{B_T}\right)^3\right]}$$

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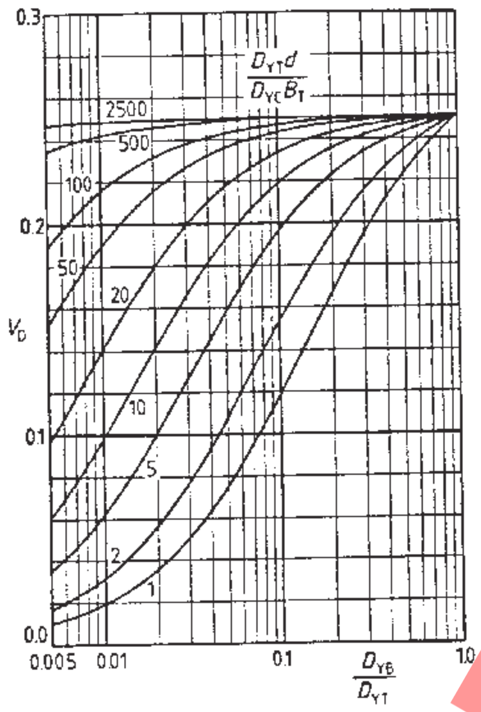
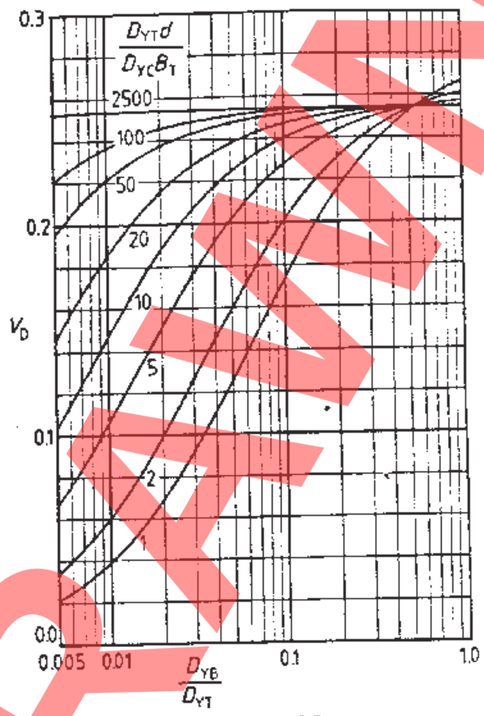
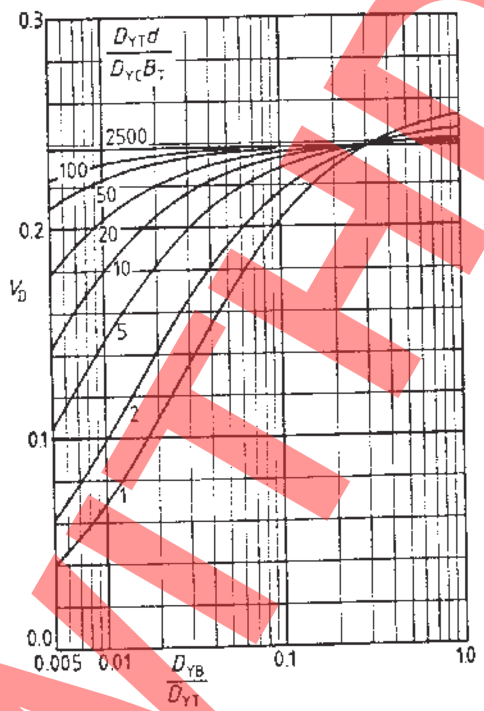


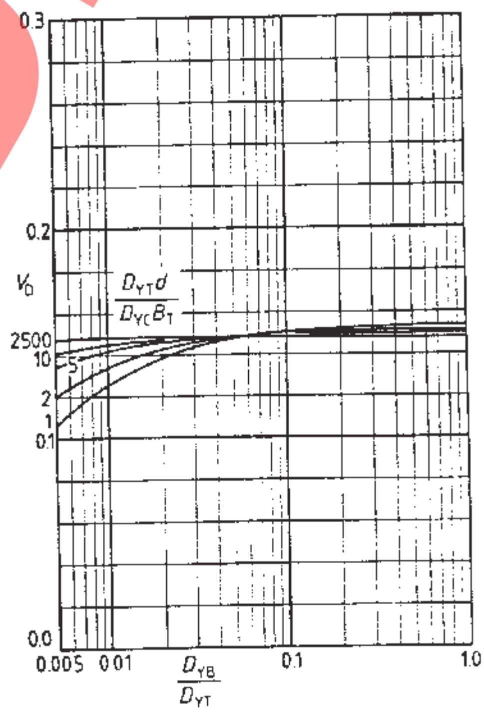
(a) Rectangular box



(b) 30° trapezoidal box, $\phi_T = 0.3$



(c) 30° trapezoidal box, $\phi_T = 0.5$



(d) 30° trapezoidal box, $\phi_T = 0.8$

NOTE. For basis of curves, see G.17.

Figure 60. Distortional bending stress parameters

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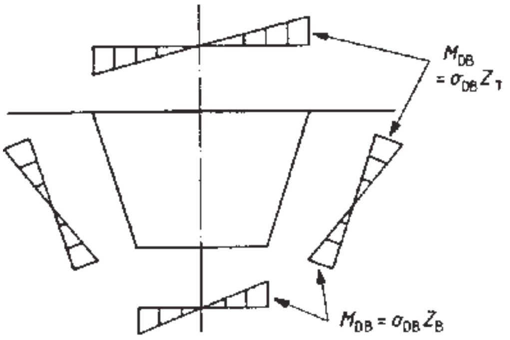

Figure 61. Transverse distortional moments

Appendix C
Slenderness limitations for open stiffeners

As an alternative to the provisions of 9.3.4.1 the limiting proportions of open stiffeners may be determined as follows:

(a) For all open stiffeners:

(1) $\frac{d_s}{t_s}$ should not exceed $1.7\sqrt{\frac{E}{\sigma_{ys} + \sigma_a}}$

(2) additionally, if $m\left[F_1 + F_2\left(\frac{d_s}{t_s}\right)^2\right]$ is less than $\frac{2.25\sigma_{ys}}{E}$, then $\frac{d_s}{t_s}$ should not exceed:

$$\left[\frac{4F_3\sigma_{ys}}{\left\{\frac{2.25\sigma_{ys}}{E} - m\left[F_1 + F_2\left(\frac{d_s}{t_s}\right)^2\right]\right\}(3\sigma_{ys} + \sigma_a)}\right]^{0.5}$$

(b) In addition, for angle and tee stiffeners:

$\frac{b_{so}}{t_{so}}\sqrt{\frac{\sigma_{ys}}{355}}$ should not exceed 10

where

$$m = \frac{1}{50}\left[(\alpha_1^2 + 40\alpha_2 - 4\alpha_3)^{0.5} - \alpha_1\right] \text{ but not less than } \left(\frac{t_s}{\ell_s}\right)^2$$

$$\alpha_1 = \frac{btt_s^2}{I_{ps}}$$

$$\alpha_2 = \frac{bt}{A_s}\left(\frac{t}{b}\right)^2 \frac{t_s^4}{d_s^2 r_{ys}^2}$$

$$\alpha_3 = \frac{bt}{A_s}\left(\frac{J_s}{I_{ps}}\right) \frac{t_s^4}{d_s^2 r_{ys}^2}$$

A_s is the cross-sectional area of stiffener
I_{ps} is the polar moment of inertia of stiffener about its point of attachment with the plate
r_{ys} is the radius of gyration of the stiffener about its centroidal axis normal to the plate
J_s is the St. Venant torsion constant of the stiffener
F_1, F_2, F_3 are coefficients given in figure 62 for the ratios t_{so}/t_s and b_s/d_s
σ_a, b, t, b_s, t_s, b_{so}, t_{so}, ℓ_s and d_s are as defined in 9.3.4 and shown on figure 1
σ_{ys} is as defined in 9.3.1.

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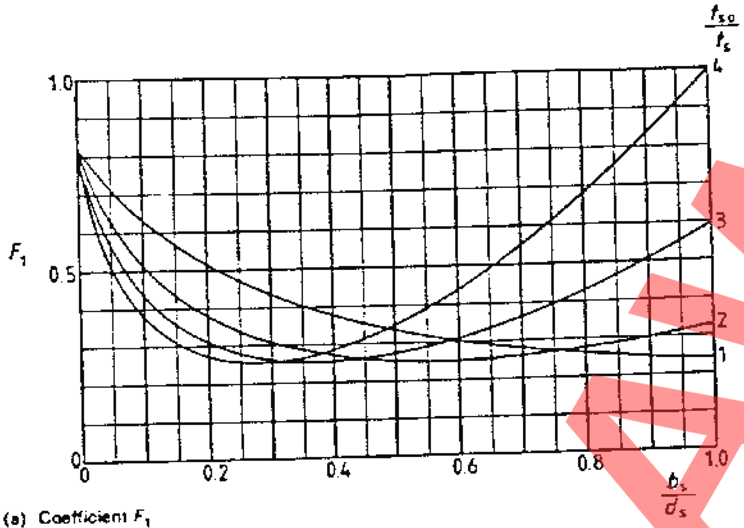
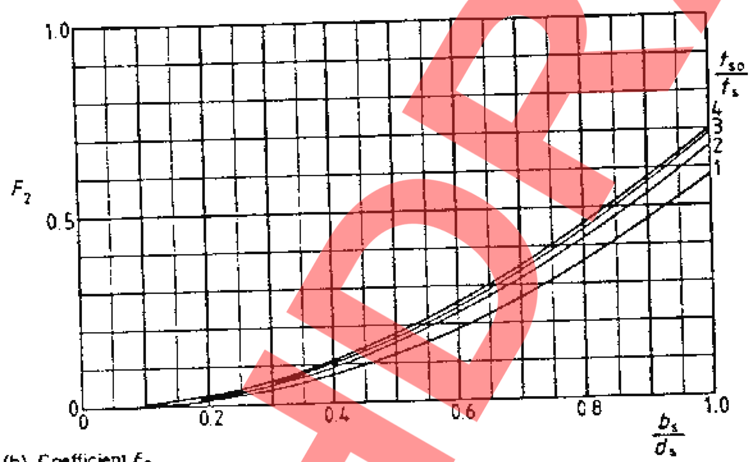
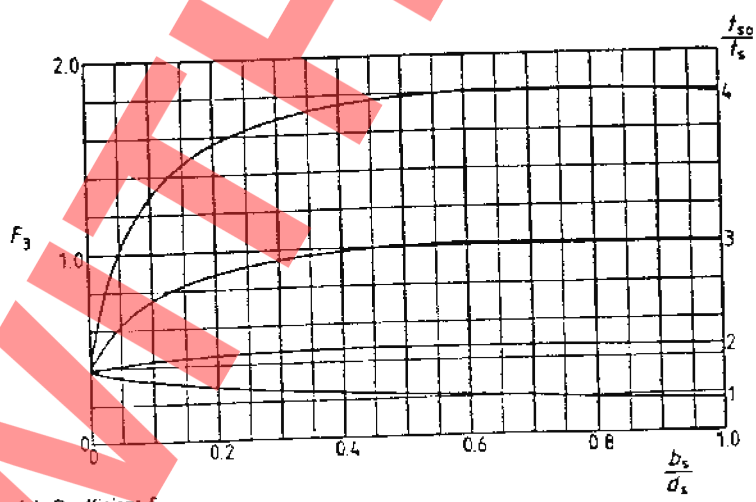


(a) Coefficient F_1



(b) Coefficient F_2



(c) Coefficient F_3

NOTE. For a basis of curves, see G.18.

Figure 62. Coefficients for torsional buckling

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Appendix D
Patch loading on webs: buckling considerations

D.1 Beams without longitudinal stiffeners on web or flange. If the dimensions of the flange plate shown in figure 1 are such that:

$$\frac{t_f}{t_w} \geqslant 1.2\sqrt{\frac{w+2j}{B_f}\,\frac{\sigma_{yw}'}{\sigma_{yf}'}}$$

then the limiting value of patch load P on each web in its plane is given by:

$$P = \left[\frac{B_f t_f^2 \sigma_{yf}}{j} + K_c t_w \sigma_{yw}'(2j+w)\right]\frac{1}{\gamma_m \gamma_{f3}}$$

If the dimensions of the flange plate do not satisfy the above criterion, then:

$$P = \frac{K_c t_w \sigma_{yw}' w}{\gamma_m \gamma_{f3}}$$

where

t_f is the flange plate thickness
t_w is the web plate thickness
w is the width of the patch load (see **9.5.6** and figure 6)

$j \quad = \sqrt{\dfrac{B_f t_f^2 \sigma_{yf}'}{2K_c t_w \sigma_{yw}}}$ but not more than:

(a) $(a-w)/2$ for symmetrically placed patch loading between transverse stiffeners; or
(b) the larger of the two unloaded lengths between the patch load and an adjacent transverse stiffener, for unsymmetrically placed patch loading

B_f is the width of the flange plate
K_c is the coefficient for web strength for transverse compression, to be obtained from curves 1 or 2 of figure 22(a) corresponding to slenderness parameter given by $\lambda = \dfrac{b_e}{t_w}\sqrt{\dfrac{\sigma_{yw}'}{355}}$, the curve for restrained panels may be used for interior panels of transversely stiffened webs, otherwise the curve for unrestrained panels should be used

$b_e \quad = 1.9\sqrt{\dfrac{wd}{K_w}}$

d is the depth of the web in its plane

$K_w \quad = \left(3.4 + 2.2\dfrac{d}{a}\right)\left(0.4 + \dfrac{w}{2a}\right)$

a is the spacing between transverse stiffeners to be taken as infinity for girders without intermediate transverse stiffeners

$\sigma_{yf}' \quad = \sigma_{yf}\left[1 - \left(\dfrac{\sigma_f}{\sigma_{yf}}\right)^2\right]$

$\sigma_{yw}' \quad = \sigma_{yw}\left[1 - \left(\dfrac{\sigma_f}{\sigma_{yw}}\right)^2\right]$

σ_{yf}, σ_{yw} are the nominal yield stresses of material of flange and web, respectively
σ_f is the longitudinal stress in the flange due to bending moment and/or axial force on beam. For a compact section the bending stress may be calculated on the basis of its plastic modulus.

D.2 Beams with longitudinal stiffeners on web or flange. The buckling criteria of **9.11.4** should be satisfied for all web panels within the dispersal zone using the following parameters:

(a) σ_2 should be taken as the transverse stress on the edges nearest to the patch load, obtained as shown in figure 20.
(b) K_c should be obtained from curves 1 or 2 of figure 22(a) for the plate slenderness parameter λ, given by

$$\lambda = \frac{b_e}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

where

$b_e \quad = 1.9\sqrt{\dfrac{w_2 b}{K_w}}$

b is the depth of the panel in its plane

$K_w \quad = \left(3.4 + 2.2\dfrac{b}{a}\right)\left(0.4 + \dfrac{w_2}{2a}\right)$

w_2 is the length of the edge nearest the patch load within the dispersal zone but not greater than a, see figure 20

a, t_w, σ_{yw} are as defined in **D.1**.

Appendix E
Transverse moments in compression flanges: U-frame restraints

The maximum value M_y of the moment in the plane of the flange required in **9.12.2.3**(b) or **9.12.3.2**(c) to be applied to the compression flange of the beam may be taken as follows:

$$M_y = \frac{5EI_c\theta d_2}{L\ell_e\left(1 - \dfrac{\sigma_{fc}}{\sigma_{ci}'}\right)}\left[1 + \frac{\left(\dfrac{L}{\ell_e}\right) - 1.25}{2.8 + 3.5\left(\dfrac{\sigma_{fc}}{\sigma_{ci}'}\right)^2}\right]$$

where

I_c and d_2 are as defined in **9.6.5** or **9.6.6**, as appropriate
θ is as defined in **9.12.2.3**(a) or **9.12.3.2**(b), as appropriate
ℓ_e is the effective length of the beam, derived in **9.6.5** or **9.6.6**, as appropriate
σ_{fc} is the maximum compressive stress in the flange
σ_{ci}' is taken as follows:

(a) if ℓ_e is less than three times the spacing of U-frames, then $\sigma_{ci}' = \sigma_{ci}$, as defined in **9.1.2.2**;
(b) if ℓ_e is more than four times the spacing of U-frames, or if ℓ_e has been calculated in accordance with **9.6.6.2**, then $\sigma_{ci}' = 1.25\sigma_{ci}$; or
(c) for intermediate values of ℓ_e, σ_{ci}' is obtained by linear interpolation

L is as defined in **9.6.3**.

For uniformly distributed loading, HA loading and RL loading, placed over the whole span, the maximum moment M_y derived above should be assumed to act anywhere within a horizontal distance ℓ_e from each bearing support of the beam.

Elsewhere the bending moment may be assumed to be $M_y/2$.

For all other loading cases it should be assumed that M_y acts anywhere within the span.

Appendix D

Patch loading on webs: buckling consideration

Delete the contents of the existing appendix and substitute as follows:

D.1 Beams without longitudinal stiffeners on web.

The limiting value of patch load P on each web in its plane should be taken as the lesser of

(a) web buckling criterion

$$0.5 t_w^2 \left(E\sigma_{yw} \frac{t_f}{t_w}\right)^{0.5} \left[1 + \frac{3w}{d}\left(\frac{t_w}{t_f}\right)^{1.5}\right] \left[1 - \left(\frac{\sigma_f}{\sigma_{yw}}\right)^2\right]^{0.5} \frac{1}{\gamma_m \gamma_{f3}}$$

(b) web yielding criterion

$$\left[2t_f\left(\sigma_{yf}\sigma_{yw}B_f t_w\right)^{0.5} + \sigma_{yw}t_w w\right] \left[1 - \left(\frac{\sigma_f}{\sigma_{yw}}\right)^2\right]^{0.5} \frac{1}{\gamma_m \gamma_{f3}}$$

where

- t_f is the flange plate thickness
- t_w is the web plate thickness
- w is the width of the patch load (see **9.5.6** and figure 6) but to be taken not greater than 0.2d
- B_f is the width of the flange plate
- d is the depth of the web in its plane
- σ_{yf} and σ_{yw} are the nominal yield stresses of the material of flange and web, respectively
- σ_f is the longitudinal stress in the flange due to bending moment and/or axial force on the beam. For a compact section the bending stress may be calculated on the basis of its plastic modulus
- γ_m is taken as 1.05 for the ultimate limit state.

D.2 Beams with longitudinal stiffeners on web.

The limiting value of patch load P on each web in its plane should be taken as the lesser of the limiting values given in **D.1**(a) multiplied by a factor K or **D.1**(b)

where

- K = 1.28 - 0.7 (b/d) but not less than 1.0 or greater than 1.21
- b is the clear distance between the flange and the longitudinal web stiffener nearest to the flange, which complies with **9.11.5.**

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Appendix D
Patch loading on webs: buckling considerations

D.1 Beams without longitudinal stiffeners on web or flange. If the dimensions of the flange plate shown in figure 1 are such that:

$$\frac{t_f}{t_w} \geqslant 1.2\sqrt{\frac{w+2j}{B_f}\frac{\sigma_{yw}'}{\sigma_{yf}'}}$$

then the limiting value of patch load P on each web in its plane is given by:

$$P = \left[\frac{B_f t_f^2 \sigma_{yf}}{j} + K_c t_w \sigma_{yw}'(2j+w)\right]\frac{1}{\gamma_m \gamma_{f3}}$$

If the dimensions of the flange plate do not satisfy the above criterion, then:

$$P = \frac{K_c t_w \sigma_{yw}' w}{\gamma_m \gamma_{f3}}$$

where

t_f is the flange plate thickness

t_w is the web plate thickness

w is the width of the patch load (see **9.5.6** and figure 6)

j $= \sqrt{\dfrac{B_f t_f^2 \sigma_{yf}'}{2K_c t_w \sigma_{yw}'}}$ but not more than:

(a) $(a-w)/2$ for symmetrically placed patch loading between transverse stiffeners; or

(b) the larger of the two unloaded lengths between the patch load and an adjacent transverse stiffener, for unsymmetrically placed patch loading

B_f is the width of the flange plate

K_c is the coefficient for web strength for transverse compression, to be obtained from curves 1 or 2 of figure 22(a) corresponding to slenderness parameter given by $\lambda = \dfrac{b_e}{t_w}\sqrt{\dfrac{\sigma_{yw}'}{355}}$; the curve for restrained panels may be used for interior panels of transversely stiffened webs, otherwise the curve for unrestrained panels should be used

b_e $= 1.9\sqrt{\dfrac{wd}{K_w}}$

d is the depth of the web in its plane

K_w $= \left(3.4 + 2.2\dfrac{d}{a}\right)\left(0.4 + \dfrac{w}{2a}\right)$

a is the spacing between transverse stiffeners to be taken as infinity for girders without intermediate transverse stiffeners

σ_{yf}' $= \sigma_{yf}\left[1 - \left(\dfrac{\sigma_f}{\sigma_{yf}}\right)^2\right]$

σ_{yw}' $= \sigma_{yw}\left[1 - \left(\dfrac{\sigma_f}{\sigma_{yw}}\right)^2\right]$

σ_{yf}, σ_{yw} are the nominal yield stresses of material of flange and web, respectively

σ_f is the longitudinal stress in the flange due to bending moment and/or axial force on beam. For a compact section the bending stress may be calculated on the basis of its plastic modulus.

D.2 Beams with longitudinal stiffeners on web or flange. The buckling criteria of **9.11.4** should be satisfied for all web panels within the dispersal zone using the following parameters:

(a) σ_2 should be taken as the transverse stress on the edges nearest to the patch load, obtained as shown in figure 20.

(b) K_c should be obtained from curves 1 or 2 of figure 22(a) for the plate slenderness parameter λ, given by

$$\lambda = \frac{b_e}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

where

b_e $= 1.9\sqrt{\dfrac{w_2 b}{K_w}}$

b is the depth of the panel in its plane

K_w $= \left(3.4 + 2.2\dfrac{b}{a}\right)\left(0.4 + \dfrac{w_2}{2a}\right)$

w_2 is the length of the edge nearest the patch load within the dispersal zone but not greater than a, see figure 20

a, t_w, σ_{yw} are as defined in **D.1**.

Appendix E
Transverse moments in compression flanges: U-frame restraints

The maximum value M_y of the moment in the plane of the flange required in **9.12.2.3**(b) or **9.12.3.2**(c) to be applied to the compression flange of the beam may be taken as follows:

$$M_y = \frac{5EI_c\theta d_2}{L\ell_e\left(1 - \dfrac{\sigma_{fc}}{\sigma_{ci}'}\right)}\left[1 + \frac{\left(\dfrac{L}{\ell_e}\right) - 1.25}{2.8 + 3.5\left(\dfrac{\sigma_{fc}}{\sigma_{ci}'}\right)^2}\right]$$

where

I_c and d_2 are as defined in **9.6.5** or **9.6.6**, as appropriate

θ is as defined in **9.12.2.3**(a) or **9.12.3.2**(b), as appropriate

ℓ_e is the effective length of the beam, derived in **9.6.5** or **9.6.6**, as appropriate

σ_{fc} is the maximum compressive stress in the flange

σ_{ci}' is taken as follows:

(a) if ℓ_e is less than three times the spacing of U-frames, then $\sigma_{ci}' = \sigma_{ci}$, as defined in **9.1.2.2**;

(b) if ℓ_e is more than four times the spacing of U-frames, or if ℓ_e has been calculated in accordance with **9.6.6.2**, then $\sigma_{ci}' = 1.25\sigma_{ci}$; or

(c) for intermediate values of ℓ_e, σ_{ci}' is obtained by linear interpolation

L is as defined in **9.6.3**.

For uniformly distributed loading, HA loading and RL loading, placed over the whole span, the maximum moment M_y derived above should be assumed to act anywhere within a horizontal distance ℓ_e from each bearing support of the beam.

Elsewhere the bending moment may be assumed to be $M_y/2$.

For all other loading cases it should be assumed that M_y acts anywhere within the span.

Appendix E Transverse moments in compression flanges: U-frame restraints

In the definition for σ_{ci}, item (a), replace "**9.1.2.2**" with "**9.12.2.2**"

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The moment M_y thus obtained should be combined with other effects giving M_y' for checking compliance with the following:

(a) for flanges in beams without longitudinal stiffeners (see **9.9**):

$$\frac{M}{M_D} + \frac{M_y'}{M_{Dy}'} \leqslant 1$$

where

M and M_D are the applied moment and the moment capacity, respectively, for bending parallel to the web of the main beam

M_{Dy}' is the transverse moment capacity of the compression flange with λ_{LT} taken as zero.

(b) for flanges in longitudinally stiffened beams (see **9.10**):

$$\frac{\sigma_f}{\sigma_{fc}} + \frac{\sigma_b}{\sigma_{fco}} \leqslant \frac{1}{\gamma_m \gamma_{f3}}$$

where

σ_f is the longitudinal stress in that part of the flange plate under consideration due to the applied bending moment in the plane of the web, based on the plastic section modulus when the design is for a compact section (see **9.3.7**)

σ_{fc} is the limiting compressive stress derived in accordance with **9.8**

σ_b is the longitudinal stress in that part of the flange plate under consideration when M_y' is applied to the compression flange alone

σ_{fco} is the limiting compressive stress for the compression flange calculated with λ_{LT} taken as zero.

Appendix F
Buckling coefficients for transverse members in compression flanges

For cases not in accordance with the limitations given in (a), (b) or (c) of **9.15.3.2**, values of the buckling coefficient K may be determined from:

$$K = 24\,\frac{\left[a_1 + \frac{kf_c}{f}\Omega\lambda_B a_2\right]\left[a_1{}^2 + \frac{2k}{3}\lambda_B \xi a_2{}^2\right]}{\left[a_1{}^2 + \frac{2k}{3}\frac{f_c}{f}\lambda_B(3\Omega - 2)a_2{}^2\right]^2}$$

where

$a_1 = 3k\Omega\frac{f_c}{f}\lambda_B{}^2 + 1$

$a_2 = 16\left[\frac{\Omega f_c I_{be}}{f I_{bc}}\right]\lambda_B{}^4 + 8\left[(1+k)\frac{\Omega f_c}{f}\right]\lambda_B{}^3 + 3\lambda_B$

k = 2 if there are cantilever brackets on both sides of the segment, or
= 1 if on one side only

f_c is the longitudinal force per unit width in the cantilever portion of the flange

f is the longitudinal force per unit width in the portion of the flange between main beam webs

$\lambda_B = \frac{B_c}{B}$

B and B_c are as shown on figure 29

$\Omega = 1 + \frac{E_E A_E}{B_c E_{fc} A_{fc}}$

$\xi = \left[1 + \frac{3E_E I_E}{B_c E_{fc} I_{fc}}\right]\frac{E_{fc} I_{fc}}{E_f I_f}$

E_f is Young's modulus for the portions of flange between main beam webs

E_{fc} is Young's modulus for the cantilever portion of the flange

E_E is Young's modulus for the edge member

A_{fc} is the area of the flange under consideration per unit width of the cantilever portions of the flange

A_E is the area of the cross section of the edge member

I_{be} is the average second moment of area of the portion of transverse member between centreline of main beam webs

I_{bc} is the average second moment of area of the cantilever portions of transverse member

I_E is the second moment of area of the edge member about its centroidal axis

I_f is the second moment of area of the flange under consideration per unit width of the portions of flange between main beam webs

I_{fc} is the second moment of area of the flange under consideration per unit width of cantilever portions of the flange.

For a compression flange with closed longitudinal stiffeners: I_f in this appendix and in **9.15.3**, may be increased by multiplying by a factor:

$$1 + \frac{G_f J_f a^2}{E_f I_f B^2}$$

and I_{fc} in this appendix may be increased by multiplying by a factor:

$$1 + 0.3\,\frac{G_{fc} J_{fc}}{E_{fc} I_{fc}}\left(\frac{a^2}{B_c{}^2}\right)$$

where

a is as defined in **9.15.3.2**

J_f is the torsional constant of longitudinal stiffeners per unit width of the flange between main beam webs

J_{fc} is the torsional constant of longitudinal stiffeners per unit width of the cantilever overhang

G_f and G_{fc} are the shear modulus of the longitudinal stiffeners between main beam webs and in cantilever overhang respectively.

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Appendix G
Equations used for production of curves in figures

G.1 General. As an alternative to obtaining values from the graphs given in the figures of the main document, the following equations may be used for calculation of the required values; these equations should only be used within the bounds of the figures themselves.

G.2 Figure 2. Limiting slenderness for flat stiffeners

$$\frac{1}{q^2} = \frac{1}{10p^2} + \frac{5.21r^2p^3}{S} + 0.625r, \text{ when } q < 31.0$$

$$p = 10, \text{ when } q \geqslant 31.0$$

where

$$p = \frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}}$$

$$q = \frac{b}{t}\sqrt{\frac{\sigma_y}{355}}$$

$$r = 0.00474 - \frac{0.468}{p^2}$$

$$S = \frac{bt}{t_s^2}\sqrt{\frac{\sigma_{ys}}{355}}$$

σ_y and σ_{ys} are as defined in 9.3.1
h_s, t_s, b and t are as defined in 9.3.4.1.2

G.3 Figure 3. Limiting slenderness for angle stiffeners

$$\frac{h_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 6.2 + \frac{31.6}{\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} - 10}$$

when

$$\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} < 50$$

where

σ_{ys} is as defined in 9.3.1
h_s, t_s, b_s and ℓ_s are as defined in 9.3.4.1.4

G.4 Figure 4. Limiting slenderness for tee stiffeners

(a) *figure 4(a) related to ℓ_s/b_s*

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 80 - 5\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} \text{ when } \frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} \leqslant 12$$

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 5 + \frac{30}{\frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} - 10}$$

$$\text{when } 12 < \frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} < 25$$

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 7 \text{ when } \frac{\ell_s}{b_s}\sqrt{\frac{\sigma_{ys}}{355}} \geqslant 25$$

(b) *figure 4(b) related to b/t*

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 7 + 4\left(\frac{t}{t_s}\right)^{0.8}\left(32 - \frac{b}{t}\sqrt{\frac{\sigma_y}{355}}\right)^{0.6}$$

$$\text{when } \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} < 32, \text{ and } \frac{d_s}{b_s} \leqslant 4$$

$$\frac{d_s}{t_s}\sqrt{\frac{\sigma_{ys}}{355}} = 7 \text{ when } \frac{b}{t}\sqrt{\frac{\sigma_y}{355}} \geqslant 32$$

where

σ_y and σ_{ys} are as defined in 9.3.1
d_s, t_s, b_s, ℓ_s, b and t are as defined in 9.3.4.1.5.

G.5 Figure 5. Coefficient K_c for plate panels under direct compression

(a) *Curve 1 (restrained)*

$K_c = 1$ when $\lambda \leqslant 24$

$K_c = \left(\frac{24}{\lambda}\right)^{0.50}$ when $24 < \lambda \leqslant 43$

$K_c = \left(\frac{28}{\lambda}\right)^{0.68}$ when $43 < \lambda \leqslant 59$

$K_c = \left(\frac{30}{\lambda}\right)^{0.75}$ when $59 < \lambda \leqslant 90$

$K_c = \left(\frac{36}{\lambda}\right)^{0.90}$ when $90 < \lambda \leqslant 130$

$K_c = 0.38 - \frac{\lambda}{2000}$ when $130 < \lambda \leqslant 200$

$K_c = 0.33 - \frac{\lambda}{4000}$ when $200 < \lambda \leqslant 300$

(b) *Curve 2 (unrestrained)*

$K_c = 1$ when $\lambda \leqslant 24$

$K_c = \left(\frac{24}{\lambda}\right)^{0.75}$ when $24 < \lambda \leqslant 47$

$K_c = \left(\frac{26}{\lambda}\right)^{0.85}$ when $47 < \lambda \leqslant 130$

$K_c = 0.274 - \frac{\lambda}{7000}$ when $130 < \lambda \leqslant 300$

(c) *Curve 3 (restrained and unrestrained)*

$K_c = 1$ when $\lambda \leqslant 4.33$

$$K_c = 0.5\left[\left\{1 + \left(1 + \eta\right)\frac{475}{\lambda^2}\right\} - \sqrt{\left\{1 + \left(1 + \eta\right)\frac{475}{\lambda^2}\right\}^2 - \frac{1900}{\lambda^2}}\right]$$

when $4.33 < \lambda \leqslant 300$

where

$\eta = 0.0156(\lambda - 4.33)$
λ is as defined in 9.4.2.4 and on figure 5.

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G.6 Figure 7. Influence on effective length of compression flange restraint

(a) *Figure 7(a) Effect of rotational end restraint*

$$k_1 = 0.5 + \frac{0.5}{1 + 0.425\left(\frac{k_o L}{EI_c}\right)}$$

where

L is the span of the beam or truss, or the length between the ends of a compression member effectively held in position

k_o is as defined in figure 7(a)

I_c is as defined in **9.6.5**, **10.4.1** or **12.5.1**, as appropriate.

(b) *Figure 7(b) Effect of bending restraint*

$$\frac{1}{k_3} = \sqrt{1 + \left(\frac{l_o}{L}\right)^2\left(\frac{1}{k_1{}^2} - 1\right)}$$

where

k_1 is as derived in (a) above

l_o is the value of l_e obtained from **9.6.5**, **9.6.6** or **12.5.1** with $k_3 = 1.0$

L is as defined in (a) above

G.7 Figure 10. Basic limiting stress σ_{li}

$$\frac{\sigma_{li}}{\sigma_{yc}} = 0.5\left[\left\{1 + (1+\eta)\frac{5700}{\beta^2}\right\} - \sqrt{\left\{1 + (1+\eta)\frac{5700}{\beta^2}\right\}^2 - \frac{22800}{\beta^2}}\right] \text{ when } \beta > 45$$

$$\frac{\sigma_{li}}{\sigma_{yc}} = 1 \text{ when } \beta \leqslant 45$$

where

$$\eta = 0.005(\beta - 45)$$

$$\beta = \lambda_{LT}\sqrt{\frac{\sigma_{yc}}{355}}$$

λ_{LT} and σ_{yc} are as defined in **9.8.1**.

G.8 Figures 11 to 17. Limiting shear strength τ_l. The following iterative procedure is required.

(a) Calculate:

$$\beta = \frac{\lambda}{\sqrt{5.34 + \frac{4}{\phi^2}}} \quad \text{when } \phi \geq 1$$

$$\beta = \frac{\lambda}{\sqrt{\frac{5.34}{\phi^2} + 4}} \quad \text{when } \phi < 1$$

(b) Calculate:

$$\frac{\tau_c}{\tau_y} = 1 \quad \text{when } \beta \leqslant 24.55$$

$$\frac{\tau_c}{\tau_y} = 1.54 - 0.022\beta \text{ when } 24.55 < \beta < 33.62$$

$$= \frac{904}{\beta^2} \quad \text{when } 33.62 \leqslant \beta$$

(c) Assume a value of θ such that:

$$0.33 \cot^{-1}\phi \leqslant \theta \leqslant 1.33 \cot^{-1}\phi \text{ and also } \leqslant \frac{\pi}{4}$$

(d) Calculate:

$$\frac{\sigma_t}{\tau_y} = \sqrt{\left[3 + (2.25\sin^2 2\theta - 3)\left(\frac{\tau_c}{\tau_y}\right)^2\right]} - 1.5\frac{\tau_c}{\tau_y}\sin 2\theta$$

(e) Calculate:

$$\frac{\tau_u}{\tau_y} = f\left[\frac{\tau_c}{\tau_y} + 5.264\sqrt{m_{fw}\frac{\sigma_t}{\tau_y}\sin\theta} + \frac{\sigma_t}{\tau_y}(\cot\theta - \phi)\sin^2\theta\right]$$

$$\text{when } m_{fw} \leq \frac{\phi^2}{4\sqrt{3}}\frac{\sigma_t}{\tau_y}\sin^2\theta$$

$$\frac{\tau_u}{\tau_y} = f\left[\frac{4\sqrt{3}\,m_{fw}}{\phi} + \frac{\sigma_t}{2\tau_y}\sin 2\theta + \frac{\tau_c}{\tau_y}\right]$$

$$\text{when } m_{fw} > \frac{\phi^2}{4\sqrt{3}}\frac{\sigma_t}{\tau_y}\sin^2\theta$$

where

$$f = 1 \quad \text{when } \lambda \leqslant 56$$

$$f = \frac{1.15}{1.15 + 0.002(\lambda - 56)} \quad \text{when } 56 < \lambda < 156$$

$$f = \frac{1.15}{1.35} \quad \text{when } 156 \leqslant \lambda$$

ϕ, λ, d_{we}, t_w, τ_y and m_{fw} are as defined in **9.9.2.2**.

(f) Repeat (c) to (e) with different values of θ until the maximum value $(\tau_u/\tau_y)_{max}$ is obtained,

where

τ_l/τ_y is equal to the lesser of $(\tau_u/\tau_y)_{max}$ and 1.0

G.9 Figure 18. Parameters for the design of longitudinal flange stiffeners

$$k_l = 0.5\left[\left\{1 + (1+\eta)\frac{5700}{\lambda^2}\right\} - \sqrt{\left\{1 + (1+\eta)\frac{5700}{\lambda^2}\right\}^2 - \frac{22800}{\lambda^2}}\right]$$

$$k_l = \frac{1}{1+\eta} \text{ when } \lambda = 0$$

$$k_s = 0.4\,k_l'\,(\eta' + 1.7546 \times 10^{-4}\,\lambda^2)$$

where

$k_l' = k_l$ with $\eta = \eta'$

$\eta' = 0.0083(\lambda - 15)$ when $\lambda > 15$

$\eta' = 0$ when $\lambda \leqslant 15$

η and λ are as defined in **9.10.2.3**.

G.10 Figure 21. Minimum value of m_{fw} for outer panel restraint

$$m_{fw} = 0.01875\,\phi^{1.6}\left(\frac{\lambda - 66 - \frac{28}{\phi^2}}{134 - \frac{28}{\phi^2}}\right)^{0.3}$$

where

ϕ, λ and m_{fw} are as defined in **9.11.4.2.2**.

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G.11 Figure 22. Buckling coefficients K_1, K_2, K_q and K_b

(a) *Figure 22(a) K_1 and K_2*. The equations to be used are as given for K_c in **G.5**(a) to (c), but where λ is as defined in **9.11.4.3.2** and **9.11.4.3.5** and in figure 22(a).

(b) *Figure 22(b) K_q*

(1) Restrained panels:

for $\phi \leqslant 0.5$: $K_q = 1$ when $\lambda \leqslant 80$

$K_q = \left(\frac{80}{\lambda}\right)^{0.15}$ when $80 < \lambda \leqslant 300$

for $\phi = 1.0$: $K_q = 1$ when $\lambda \leqslant 48$

$K_q = \left(\frac{48}{\lambda}\right)^{0.11}$ when $48 < \lambda \leqslant 300$

for $\phi > 2.0$: $K_q = 1$ when $\lambda \leqslant 40$

$K_q = \left(\frac{40}{\lambda}\right)^{0.15}$ when $40 < \lambda \leqslant 300$

NOTE. For intermediate values of ϕ, K_q may be obtained by linear interpolation between two adjacent values of ϕ. For this purpose values may be obtained beyond $K_q = 1.0$ from the equation given in **9.11.4.3.3**.

(2) Unrestrained panels:

for $\phi \leqslant 0.5$:

$K_q = 1$ when $\lambda \leqslant 80$

$K_q = 1 - 0.385\left(\frac{\lambda - 80}{120}\right)^{0.743}$ when $80 < \lambda \leqslant 224$

$K_q = 1 - 0.660\left(\frac{\lambda - 80}{320}\right)^{0.505}$ when $224 < \lambda \leqslant 300$

for $\phi = 1.0$:

$K_q = 1$ when $\lambda \leqslant 48$

$K_q = \left(\frac{48}{\lambda}\right)^{0.5}$ when $48 < \lambda \leqslant 300$

for $\phi = 2.0$:

$K_q = 1$ when $\lambda \leqslant 40$

$K_q = 1 - 0.385\left(\frac{\lambda - 40}{60}\right)^{0.743}$ when $40 < \lambda \leqslant 112$

$K_q = 1 - 0.660\left(\frac{\lambda - 40}{160}\right)^{0.505}$ when $112 < \lambda \leqslant 200$

$K_q = 0.34 - 0.07\left(\frac{\lambda - 200}{100}\right)^{0.8}$ when $200 < \lambda \leqslant 300$

for $\phi \geqslant 3.0$:

$K_q = 1$ when $\lambda \leqslant 38$

$K_q = 1 - 0.555\left(\frac{\lambda - 38}{82}\right)^{0.823}$ when $38 < \lambda \leqslant 125$

$K_q = 0.445 - 0.205\left(\frac{\lambda - 120}{80}\right)^{0.725}$ when $125 < \lambda \leqslant 221$

$K_q = 0.24 - 0.075\left(\frac{\lambda - 200}{100}\right)^{0.44}$ when $221 < \lambda \leqslant 300$

NOTE. For interpolation see the note in (1).

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where

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

$$\phi = \frac{a}{b}$$

σ_{yw} and t_w are as defined in **9.11.4.3.2**.
a and b are as defined in figure 22.

(c) *Figure 22(c)* K_b

(1) Restrained panels:

$K_b = 1.3 - 0.0019\lambda$ when $\lambda \leqslant 300$

(2) Unrestrained panels:

$K_b = 1.3 - 0.0027\lambda$ when $\lambda \leqslant 300$

where

$$\lambda = \frac{b}{t_w}\sqrt{\frac{\sigma_{yw}}{355}}$$

σ_{yw} and t_w are as defined in **9.11.4.3.2**.

G.12. Figure 23. Parameters for the design of web stiffeners

$$\frac{\sigma_{ts}}{\sigma_{ys}} = 0.5\left[\left\{1 + (1+\eta)\frac{5700}{\lambda^2}\right\} - \sqrt{\left\{1 + (1+\eta)\frac{5700}{\lambda^2}\right\}^2 - \frac{22800}{\lambda^2}}\,\right]$$

$$k_s = 0.4\frac{\sigma_{ts}}{\sigma_{ys}}(\eta + 1.7546 \times 10^{-4}\lambda^2)$$

where

$\eta = 0.0083(\lambda - 15)$ when $\lambda > 15$

$\eta = 0$ when $\lambda \leqslant 15$

λ is as defined in **9.11.5.2**, **9.13.3.3**, **9.13.5.3**, **9.17.6.3.4**, **9.17.6.7** and **9.17.7.3.2** as appropriate, and in figure 23.

G.13 Figure 24. Coefficient α_{LT} for torsional restraint at supports

$\alpha_{LT} = 0.006$ when $\frac{\ell_e}{r_y} \leqslant 50$

$\alpha_{LT} = \frac{0.3}{(\ell_e/r_y)}$ when $50 < \frac{\ell_e}{r_y} \leqslant 100$

$\alpha_{LT} = \frac{30}{(\ell_e/r_y)^2}$ when $100 < \frac{\ell_e}{r_y} \leqslant 300$

where

ℓ_e and r_y are as defined in **9.12.4.1** and in figure 24

G.14 Figure 29. Buckling coefficient K for transverse members

(a) *Figure 29(a). Cantilevers on one side only*

$$K = \frac{24\left[(3b^2+1) + b^2\left(\frac{16b^3}{a} + 16b^2 + 3\right)\right]}{\left[(3b^2+1)^2 + \frac{2b^3}{3}\left(\frac{16b^3}{a} + 16b^2 + 3\right)^2\right]}$$

(b) *Figure 29(b). Cantilevers on both sides*

$$K = \frac{24\left[(6b^2+1) + 2b^2\left(\frac{16b^3}{a} + 24b^2 + 3\right)\right]}{\left[(6b^2+1)^2 + \frac{4b^3}{3}\left(\frac{16b^3}{a} + 24b^2 + 3\right)^2\right]}$$

where

$$a = \frac{I_{bc}}{I_{be}}$$

$$b = \frac{B_c}{B}$$

I_{be}, I_{bc}, B and B_c are as defined in **9.15.3.2**.

NOTE. The equations given in (a) and (b) are only valid provided that the provisions of **9.15.3.2** (a), (b) and (c) are satisfied. For other cases, the equations given in appendix F should be used.

G.15 Figure 36. Coefficient K_c for plate panels under direct compression

(a) *Curve 1: bolted or riveted edges*

$K_c = 1$ when $\lambda \leqslant 30$

$K_c = \left(\frac{30}{\lambda}\right)^{0.75}$ when $30 < \lambda \leqslant 90$

$K_c = \left(\frac{36}{\lambda}\right)^{0.90}$ when $90 < \lambda \leqslant 130$

$K_c = 0.38 - \frac{\lambda}{2000}$ when $130 < \lambda \leqslant 200$

$K_c = 0.33 - \frac{\lambda}{4000}$ when $200 < \lambda \leqslant 300$

(b) *Curve 2: rolled sections or welded construction*

$K_c = 1$ when $\lambda \leqslant 24$

$K_c = \left(\frac{24}{\lambda}\right)^{0.75}$ when $24 < \lambda \leqslant 47$

$K_c = \left(\frac{26}{\lambda}\right)^{0.85}$ when $47 < \lambda \leqslant 130$

$K_c = 0.274 - \frac{\lambda}{7000}$ when $130 < \lambda \leqslant 300$

where

λ is as defined in figure 36.

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G.16 Figure 37. Ultimate compressive stress σ_c

$$\frac{\sigma_c}{\sigma_y} = 0.5\left[\left\{1 + (1+\eta)\frac{5700}{\lambda^2}\right\} - \sqrt{\left\{1 + (1+\eta)\frac{5700}{\lambda^2}\right\}^2 - \frac{22800}{\lambda^2}}\right]$$

where

$\eta = 0$ when $\lambda \leqslant 15$

$= \alpha(\lambda - 15)$ when $\lambda > 15$

$\alpha = 0.0025$ for curve A

$= 0.0045$ for curve B

$= 0.0062$ for curve C

$= 0.0083$ for curve D

$$\lambda = \frac{\ell_e}{r}\sqrt{\frac{\sigma_y}{355}}$$

ℓ_e, r and σ_y are as defined in **10.6.1.1**.

NOTE. Guidance on the appropriate use of curves A, B, C or D is given in note 1 to figure 37.

G.17 Figure 58. Distortional warping stress parameters. Figure 60. Distortional bending stress parameters

(a) *Figures 58(a) and 60(a). Rectangular box*

$$R_D = \frac{2.0}{\left(\frac{D_{YT}d}{D_{YC}B_T} + 0.5\right) - V_D\left[3\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{3\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{2 + 12\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 2\left(\frac{D_{YT}}{D_{YB}}\right)}$$

(b) *Figures 58(b) and 60(b). 30° trapezoidal box,* $\phi_T = 0.3$

$$R_D = \frac{2.43}{0.412\left[2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right] - V_D\left[2.7\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{2.7\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{2.43 + 10.62\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 0.834\left(\frac{D_{YT}}{D_{YB}}\right)}$$

(c) *Figures 58(c) and 60(c). 30° trapezoidal box,* $\phi_T = 0.5$

$$R_D = \frac{3.0}{0.333\left[2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right] - V_D\left[2.5\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{2.5\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{3 + 10.5\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 0.375\left(\frac{D_{YT}}{D_{YB}}\right)}$$

(d) *Figures 58(d) and 60(d). 30° trapezoidal box,* $\phi_T = 0.8$

$$R_D = \frac{6.0}{0.167\left[2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) - 1\right] - V_D\left[2.2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1\right]}$$

$$V_D = \frac{2.2\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 1}{6 + 14.88\left(\frac{D_{YT}d}{D_{YC}B_T}\right) + 0.048\left(\frac{D_{YT}}{D_{YB}}\right)}$$

where

$$\phi_T = \frac{d}{B_T}$$

B_T is as defined in **B.2** (b)

d, D_{YT}, D_{YB} and D_{YC} are as defined in **B.3.2**.

G.18 Figure 62. Coefficients for torsional buckling

$$F_1 = \frac{\pi^2}{12}\left(\frac{K_1^3 K_2^3 + 4}{4 + K_1 K_2^3 + 12 K_1 K_2}\right)$$

$$F_2 = \pi^2\left(\frac{K_1 K_2^3}{4 + K_1 K_2^3 + 12 K_1 K_2}\right)$$

$$F_3 = 1.54\left(\frac{1 + K_1^3 K_2}{4 + K_1 K_2^3 + 12 K_1 K_2}\right)$$

where

$$K_1 = \frac{t_{so}}{t_s}$$

$$K_2 = \frac{b_s}{d_s}$$

t_s, t_{so}, b_s and d_s are as defined in **9.3.4** and shown in figure 1.

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Add new Appendix H

Appendix H

Derivation of nominal yield stress for assessment

H.1 General

The following methods may be used to derive values of the nominal yield stress, σ_y, for use in the assessment of existing bridges. Although written in terms of yield stress some methods may also be used to assess ultimate tensile stress.

H.2 Yield stress based on specifications

In the assessment of existing bridges the nominal yield stress for steels specified to comply with BS EN 10 025 or BS 4360 shall be taken as the values defined in **6.2**. For steels to BS 15, BS 548, BS 968 or BS 2762 and thickness up to 63mm the nominal yield stress may be taken as the minimum value specified in the relevant Standard for material appropriate to the thickness of 16mm irrespective of the actual thickness of the component. The issue of the Standard referred to should be that current at the date of fabrication. When the material quality specified is not known and no test information is obtained the steel may be assumed to be a mild steel grade with specified minimum yield stress in BS 15 or BS 4360 appropriate to the date of construction provided that the steel can be identified, by means of trade marks or names, as being made by a British supplier.

H.3 Yield stress based on tests of the material in the component to be assessed

If a tensile test in accordance with BS 4360 is undertaken on a sample taken from a particular component to be assessed at the location within its cross section defined in BS 4360 the nominal yield stress of that component may be taken as the measured value.

H.4 Yield stress based on mill test certificates or tests on samples

H.4.1 Yield stress based on mill test certificates or tests on samples taken from existing structures composed of BS EN 10 025, BS 4360, BS 15, BS 548, BS 968 or BS 2762 steel

When in assessment mill test certificates for the material used are available or tests are undertaken on the materials for representative parts the yield stresses derived from these may be used to derive the nominal yield stress as follows, provided that the materials were specified to one of the above Standards.

(a) If mill test certificates are available which can be identified as applying to the cast number and product type of the component being assessed but not necessarily to a particular batch from which the component was rolled, or the results of tests in accordance with BS 4360 on samples taken from components of the same profile and the same structure as the part to be assessed are obtained, the nominal yield stress of that component may be taken as the greatest of:

(i) σ_y = the value derived from **H.2** above

(ii) $$\sigma_y = \sigma_{ym}\left(1 - 0.128\left(\frac{n+1}{n}\right)\right)$$

where σ_{ym} is the mean of the yield stresses on the relevant certificates or obtained from the tests;

n is the number of relevant certificates or test results.

(iii) $$\sigma_y = \frac{\sigma_{ym} - 1.2\,k\,s^*}{0.93 + 17.4\left(\dfrac{s^*}{\sigma_{ym}}\right)^2}$$

where σ_{ym} *is* as defined in H.4.1(a) above

s* is the standard deviation from σ_{ym} of the relevant test results

k is a statistical coefficient values of which are given in Table H.4A for various numbers, n, of relevant test results

If a mill test certificate is available which can be identified as applying to the particular batch of material from one cast or a 40 tonne part of cast from which component being assessed was rolled, or the result a test on a sample taken from the component is obtained the nominal yield stress may be taken as:

$$\sigma_y = \sigma_{yt} - 10\ \text{N/mm}^2$$

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where σ_{yt}, is the yield stress given on the certificate or obtained from the test in N/mm^2

Table H.4A: Values of statistical coefficient k

n	**2***	**3***	**4***	**5**	**6**	**7**	**8**	**9**	**10**	**11**
k	25	7.66	5.14	4.20	3.71	3.40	3.19	3.03	2.91	2.82

n	**12**	**13**	**14**	**15**	**16**	**17**	**18**	**19**	**20**	**21**
k	2.74	2.67	2.61	2.57	2.52	2.49	2.45	2.42	2.40	2.37

n	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**
k	2.35	2.33	2.31	2.29	2.28	2.26	2.24	2.23	2.22

n	**35**	**40**	**45**	**50**	**60**	**70**	**80**	**90**	**100**	**∞**
k	2.17	2.13	2.09	2.07	2.02	1.99	1.96	1.94	1.93	1.65

NOTE: * The use of less than five test results is not recommended.

H.4.2 Yield stress in existing structures composed of other or unidentified steels:

When the steel material is not known to comply with BS EN 10 025, BS 4360, BS 15, BS 548, BS 968 or BS 2723, tests should be undertaken on samples taken from the components or similar components in the same structure to determine the yield stress and the nominal yield stress should be derived in accordance with **H.4.1** (a) (iii) above.

H.5 Worst credible yield stress

When none of the methods in **H.2** to **H.4** can be applied, the nominal yield stress may be taken as the worst credible yield stress, being the value judged to be the least that the actual yield stress would have. In this context, the results of hardness testing (see **6.3**) may be used to provide an estimate of the U.T.S. from which the grade of steel may be judged.

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Add new Appendix I

Appendix I

Inspections for assessment

I.1 General

Inspections for Assessment should comply with the aims and provisions of BD21 and with the recommendations given below.

I.2 Criticality ratings

The nature and extent of inspections should be related to:

(a) the prior knowledge of the construction of the bridge, including as-built drawings, construction procedures, dimensional surveys and material property data; and

(b) the criticality of each part of the bridge in relation to its overall and local structural adequacy.

A preliminary inspection should be undertaken to establish the following:

(1) In the absence of drawings or previous dimensional surveys of any part of the whole of a bridge, the layout dimensions and nominal component sizes should be recorded. Details of all accessible connections should be measured and the locations of any inaccessible parts or connections should be noted. Locations of significant visible damage, deterioration or cracking should be recorded.

(2) If design drawings but no as-built drawings or previous dimensional surveys are available, the layout dimensions should be checked against the design drawings and nominal component sizes used should be verified. Connections should be visually inspected for compliance with the drawings and any variation in location or arrangement noted. Locations of significant damage, deterioration or cracking should be recorded.

(3) For bridges in which the load effects are sensitive to errors in level inclination or common planarity of bearings and for which no as built records of these are available the bearings should be surveyed and the errors recorded.

(4) If as-built drawings and/or previous surveys are available no preliminary inspection is needed.

Following any preliminary inspection an initial assessment of the adequacy of the structure should be undertaken using the best information then available. This should be used to establish the relative criticality of each part and to identify what further information is required to enable the final assessment to be undertaken. At this stage pessimistic assumptions should be made with respect to any relevant parameters for which information is lacking (eg material properties, or constructional imperfections). Those parts shown as likely to be inadequate should be identified on drawings to be used for reference in a detailed inspection. They should include parts shown by the preliminary inspection to be significantly corroded or deteriorated.

I.3 Detailed inspections

The detailed inspection of a bridge should supplement the information concerning the details and conditions obtained in the preliminary survey as set out in **I.4** to **I.6**.

I.4 Structural arrangements and sizes

The section dimensions of components at critical locations should be measured. Dimensions of connections and their connectors should be recorded, including weld sizes.

I.5 Constructional imperfections

All components should be visually inspected for gross deformations from intended flatness or straightness. Additionally for all critical parts, the strengths of which are related to geometric imperfections, detailed measurement of deviations should be made in accordance with Clause 5.6 of BS5400 Part 6 or as otherwise defined in the assessment addenda to BS5400 Part 3.

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The alignments of all bearings in relation to load bearing stiffeners and/or diaphragms should be measured. The coincident ambient temperature of the main bridge members should be recorded and appropriate adjustment made to the eccentricities or alignment to allow for the differences between the observed and the effective bridge temperature relative to the assessments required.

I.6 Condition

At all locations where corrosion, deterioration or damage is apparent, its significance should be assessed by reference to the criticality ratings and where appropriate detailed measurements should be made of loss of section and/or investigations should be undertaken of potential influences on fatigue life or fracture propensity.

Connections in critical regions should be subjected to detailed inspection. Paint should be removed from welds and the welds subjected to 100% MPI inspection. For fatigue critical connections the welds should be also subjected to full ultrasonic examination. All bolted and riveted connections should be inspected for loss or looseness of connectors. Friction grip bolted connections should be tested for tightness by application of the appropriate torque to a representative sample of nuts.

I.7 Material properties

Where there is inadequate information concerning material properties for critical parts, samples should be taken for mechanical testing. To make use of results of such tests in accordance with Section 6, the samples should be taken in the same structure and considered to be likely to have been supplied from the same batch of material. Where possible such samples should be taken from a position on a member remote from its critical region. The locations of the samples within a section should be relevant to the strength criteria being used (e.g. within a flange of a beam when considering bending capacity) and in accordance with BS 4360.

Test specimens and tensile testing should be accordance with BS 4360. Strain rates in testing should be similar to those used in mill testing. In taking samples great care must be exercised to avoid the introduction of unacceptable stress concentrations in the structure and the modifications of the properties of the samples due to any heat input.