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1983

Rules for the design of cranes —

Part 1: Specification for classification, stress calculations and design criteria for structures

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Committees responsible for this British Standard

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Associated Offices Technical Committee
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British Constructional Steelwork Association
British Ports Association and The National Association of Ports Employers
British Transport Docks Board
Bureau of Engineer Surveyors
Construction Plant-hire Association
Control and Automation Manufacturers' Association (BEAMA)
Crown Agents for Oversea Governments and Administrations
Department of Industry, Mechanical Engineering
Department of the Environment (Building Research Establishment)
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Institution of Mechanical Engineers
Institution of Production Engineers
Institution of Structural Engineers
National Federation of Building Trades Employers
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Contents

Foreword

This Part of BS 2573 has been prepared under the direction of the Mechanical Engineering Standards Committee. It supersedes BS 2573-1:1977 which is withdrawn. The fourth revision of this standard has been prepared having due regard for the work of ISO/TC 96, Cranes, lifting appliances and related equipment, in preparing the International Standard ISO 4301 covering crane classification. The classification requirements have been revised to align with the requirements of the International Standard.

This new edition of BS 2573-1 also aligns the method of classification with that of BS 2573-2 which covers permissible stresses in crane mechanisms and in which account had already been taken of the ISO 4301 requirements.

An approach to classification based on ISO work is considered to provide scope for precise definition of the purchaser's requirements on the one hand and for rational design and economic production on the other. The change in the method of classification will not lead to any substantial change in the design of cranes although it has entailed alterations to related clauses of the standard, notably those dealing with duty and impact factors and fatigue. Because the classification of a crane is determined by the number of hoisting cycles and the state of loading, as defined by the load spectrum factor, comparisons with the four former classes in BS 2573-1:1977 can be readily made.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to iv, pages 1 to 86, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

Section 1. General

1.1 Scope

This Part of BS 2573 specifies the basis for computing stresses in crane structures and the way in which permissible stresses in crane structures shall be determined in order to secure economy in design and reliability in operation. For this purpose it specifies a classification system that enables the purchaser and the manufacturer to match a particular crane to the required duty and utilization.

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

1.2 Definitions and applications of terms

1.2.1 Definitions. For the purposes of this British Standard the following definitions apply.

1.2.1.1
dead loads

all the loads of constant magnitude and position that act permanently on the structure or member

1.2.1.2
live loads

any load except wind load that gives rise to variation of stress in a member. Such variation may be due to any change of position or magnitude of an externally applied load or to the movement of the crane structure itself

1.2.1.3
inertia forces

the forces produced by change of velocity

1.2.1.4
wind load

the forces produced by the velocity of the wind, which is assumed to act horizontally

1.2.2 Applications of terms. For the purposes of this British Standard the following terms apply as stated.

1.2.2.1
basic stresses

the stresses as set out in **5.1**, **5.2** and section 6

1.2.2.2
permissible working stress

the stress numerically equal to the basic stress multiplied by the relevant duty factor and the factor corresponding to the load combinations as specified in **3.1**. Permissible working stresses are specified in **4.3.2**

1.2.2.3
permissible fatigue stresses

stresses as set out in section 8

1.2.2.4
service conditions

the crane is deemed to be under service conditions when it is operating without load or with a load up to the maximum safe working load

Section 2. Classification of the crane as a whole

2.1 General

To provide a rational and uniform basis for certain aspects of the design of the crane structure, a group classification for the crane as a whole in the range of A1 to A8 (see Table 3) and based on the requirements of **2.2** to **2.4** shall be established.

NOTE 1 The group classification of the crane as a whole provides a framework of reference between purchaser and manufacturer for contractual and technical purposes by means of which a crane may be matched to the service for which it is required. It also enables a fatigue analysis of the design to be based on the specified life and conditions of service (see section 8).

Cranes shall be classified into groups according to:

a) *class of utilization:* as determined from the required number of operating cycles for the crane, in accordance with **2.2**.

b) *state of loading:* as determined from the conditions of loading to which the crane will be subjected, in accordance with **2.3**.

NOTE 2 It is recommended that the group classification (with the class of utilization and state of loading from which it is derived) are clearly and permanently marked on the crane.

2.2 Class of utilization

2.2.1 General. The class of utilization of the crane shall be determined from its assumed total number of all operating cycles during its intended life. For the purpose of classification, an operating cycle shall be considered to commence when a load is picked up and end at the moment when the crane is ready to pick up the next load.

NOTE Where a crane is fitted with a lifting attachment, the weight of which forms a significant proportion of the crane capacity, the effect of this on fatigue life should be carefully evaluated.

2.2.2 Determining the class of utilization

2.2.2.1 Where sufficient information is available, the number of operating cycles shall be calculated from a knowledge of the duties which the crane will be expected to perform, e.g. for a crane performing part of a continuous, repetitive process the number can readily be derived from the number of operating cycles per hour and the total number of working hours during the intended life.

The class of utilization of the crane shall then be selected from Table 1 according to the tabulated number of operating cycles that is nearest to, but not less than, the number calculated by the procedure outlined in the previous paragraph.

2.2.2.2 Where insufficient information is available for calculating the total number of operating cycles, as may be the case when the crane is used for a variety of duties, a suitable value, estimated on the basis of experience, shall be used. Where there is difficulty in assigning a suitable value, the next highest figure in Table 1 shall be taken.

2.2.2.3 Typical values or ranges of the class of utilization normally associated with particular types of crane and crane applications are given in Appendix A.

2.3 State of loading and nominal load spectrum factor

2.3.1 General. The state of loading of the crane and the corresponding nominal load spectrum factor shall characterize the extent to which the crane lifts the maximum permitted load for the configuration, or smaller loads. They depend both on the magnitudes of the lifted loads relative to the maximum permitted load, and on the number of cycles in which each is lifted relative to the total number of cycles.

Assessment of the state of loading shall be based on the same cycles as are used in **2.2** to determine the class of utilization. Also, the weight of the lifted load shall include the weight of any lifting attachment such as bucket, grab, magnet or lifting beam whether permanently fitted or detachable, and the weights of other suspended components such as the bottom block, hook and of the hoist rope that are lifted together with the load. The maximum permitted load is thus the specified safe working load plus the weights of any such lifted components or attachments that are not included in the specified safe working load.

NOTE Typical values or ranges of the state of loading normally associated with particular types of crane and crane applications are given in Appendix A.

Table 1 — Class of utilization

NOTE The number of cycles used in selecting the class of utilization is a figure used only for classification purposes and as a design parameter. It does not imply a guaranteed life.

Class of utilization	Max. number of operating cycles	Remarks
U1	3.2×10^4	Infrequent use
U2	6.3×10^4	
U3	1.25×10^5	
U4	2.5×10^5	Fairly frequent use
U5	5×10^5	Frequent use
U6	1×10^6	Very frequent use
U7	2×10^6	Continuous or near-continuous use
U8	4×10^6	
U9	Greater than 4×10^6	

2.3.2 Determining the load spectrum factor

2.3.2.1 Where details are available of the magnitudes of the loads and the number of times each will be lifted during the intended life of the crane, the load spectrum factor, K_p', shall be calculated as follows:

Let: P_i = The magnitudes of the individual lifted loads characteristic of the duty of the crane (i.e. $P_1, P_2, P_3 \ldots P_n$);

P_{max} = The maximum permitted load (see **2.3.1**);

C_i = The estimated number of cycles which occur at the individual load level P_i (i.e. $C_1, C_2, C_3 \ldots C_n$);

C_t = The total number of all the individual cycles at all load levels

$$= \sum C_i = (C_1 + C_2 + C_3 + \ldots + C_n)$$

Then: $$K_p' = \sqrt[m]{\left[\sum \left\{ \frac{C_i}{C_t} \left(\frac{P_i}{P_{max}}\right)^m \right\}\right]}$$

For the purpose of classification of the crane as a whole, $m = 3$.

Expanded, the above equation becomes:

$$K_p' = \sqrt[3]{\left\{ \frac{C_1}{C_t}\left(\frac{P_1}{P_{max}}\right)^3 + \frac{C_2}{C_t}\left(\frac{P_2}{P_{max}}\right)^3 + \frac{C_3}{C_t}\left(\frac{P_3}{P_{max}}\right)^3 + \ldots + \frac{C_n}{C_t}\left(\frac{P_n}{P_{max}}\right)^3 \right\}}$$

The state of loading shall be selected from Table 2 according to the tabulated value of nominal load spectrum factor K_p, that is the nearest to, but not less than the calculated value of K_p'.

2.3.2.2 Where details of the numbers and weights of the lifted loads during the intended life of the crane are not known, the descriptive definitions in Table 2 shall be used to assist the selection of an appropriate state of loading and corresponding nominal load spectrum factor. Where there is doubt in the selection of appropriate values, the next highest figures in Table 2 shall be used.

2.4 Determination of group classification of the crane

The group classification of the crane shall be determined from Table 3 using the class utilization and state of loading obtained in accordance with **2.2** and **2.3**.

No change in the group classification of a crane, or in its combination of state of loading and class of utilization within the same group classification shall be made without reference to the manufacturer or having a thorough design check carried out by a competent person.

Table 2 — State of loading

State of loading	Nominal load spectrum factor K_p	Descriptive definition
Q1 Light	0.5	Cranes which hoist the safe working load very rarely and, normally, light loads.
Q2 Moderate	0.63	Cranes which hoist the safe working load fairly frequently and, normally, moderate loads.
Q3 Heavy	0.8	Cranes which hoist the safe working load frequently and, normally, heavy loads.
Q4 Very heavy	1.0	Cranes which are normally loaded close to safe working load.
NOTE The values of the nominal load spectrum factor as defined in ISO 4301 are equivalent to the cube of those in Table 2 above. This difference from ISO 4301 does not affect the state of loading of the crane or its group classification but results in values that can be applied directly in the simple treatment for fatigue design given in **8.4**.		

Table 3 — Group classification of the crane as a whole

State of loading	Nominal load spectrum factor K_p	Class of utilization and maximum number of operation cycles of the crane								
		U1	U2	U3	U4	U5	U6	U7	U8	U9
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5×10^5	1×10^6	2×10^6	4×10^6	$> 4 \times 10^6$
Q1 Light	0.5	A1	A1	A2	A3	A4	A5	A6	A7	A8
Q2 Moderate	0.63	A1	A2	A3	A4	A5	A6	A7	A8	A8
Q3 Heavy	0.8	A2	A3	A4	A5	A6	A7	A8	A8	A8
Q4 Very heavy	1.0	A3	A4	A5	A6	A7	A8	A8	A8	A8

Section 3. Loads and load combinations

3.1 Loads and load combinations to be considered in design

3.1.1 General. The structure as a whole and each part of it shall be designed to withstand the load combinations given in **3.1.2** using the loads listed in **3.1.3**.

3.1.2 Load combinations

3.1.2.1 ***Crane in use without wind***

$L_1 + L_3 + H_1$

3.1.2.2 ***Crane in use with in-service wind***

$L_1 + L_3 + H_1 + V_1$

3.1.2.3 ***Fatigue check for each member in which fluctuating stresses occur***

3.1.2.3.1 Each member in which fluctuating stresses occur shall be checked for fatigue in accordance with section 8.

3.1.2.3.2 Where the simple method given in **8.4** is used the following loads shall be taken into account:

$L_1 + L_4 + H_2$

3.1.2.3.3 Where the more detailed method of calculation given in the note to **8.4** is used, the following loads shall be taken into account:

$L_1 + L_3 + H_2$

3.1.2.4 ***Crane in out-of-service condition***

$\mathrm{L}_1 + V_2$

3.1.2.5 ***Crane being erected or dismantled***

3.1.2.5.1 *During erection and/or dismantling operations*

$L_1 + V_1'$

3.1.2.5.2 *Partially erected or dismantled crane subjected to out-of-service wind*

$L_1 + V_2$

In calculating the effect of V_2 for the partially erected/dismantled structure, the support provided by temporary erection equipment such as guy ropes or staging shall be taken into account. Where such equipment is not used the structure shall be considered as free standing.

3.1.2.6 ***Crane in collision with resilient buffers***

$L_1 + B$

3.1.3 List of loads

L_1	Dead loads due to dead weight.
L_2	Live loads, including the hook load.
L_3	Live loads, including the hook load multiplied by the impact factor (see **3.1.4**).
L_4	Live loads, including the hook load multiplied by the impact factor and the nominal load spectrum factor (see **3.1.4** and **2.3**).
H_1	The combined effect of the two most severe horizontal loads (see **3.1.5**).
H_2	H_1 excluding the skew loads due to travelling.
V_1	Load due to the service wind acting horizontally in any direction where applicable.
V_1'	Load due to the maximum permissible wind speed for the erection/dismantling operation as specified by the manufacturer. V_1' shall not exceed V_1.
V_2	Load due to the out-of-service wind acting horizontally in any direction where applicable.
B	Load due to collision with resilient buffers (see **3.1.6**).

3.1.4 Impact factor. The impact factor shall apply to the motion of the hook load in a vertical direction and covers inertia forces including shock. In calculating live loads in members of the structure, the hook load shall be multiplied by the impact factor, given in Table 4 a) to Table 4 g), appropriate to the type of crane and its application.

Table 4 — Impact and duty factors according to crane type and application

NOTE The table does not cover all crane types and duties. Cranes such as floating cranes, breakdown cranes and pedestal cranes for off-shore use, for example, all have special design requirements associated with their particular applications which have to be taken into account when determining appropriate impact and duty factors.

a) *Overhead travelling industrial type cranes (O.T.C.)*

Type and/or application		Impact factor	Duty factor
Power stations, engine houses, etc.		1.1	0.95
Light work shop duty (maintenance, repairs, assembly, etc.)		1.1	1.0
Light stores duty		1.1	1.0
Medium and heavy warehouse duty		1.3	0.95
Medium and heavy workshop duty		1.3	0.95
Goliath cranes for general hook service		1.3	0.95
Goliath cranes for container handling		1.3	0.95
Goliath and bridge cranes for grabbing work	intermittent duty	1.4	0.90
	continuous duty	1.5	0.85
Shipyard cranes		1.3	0.95
Ladle cranes for foundry work		1.2	0.90
Magnet cranes for steel stockyard		1.4	0.90
Magnet cranes for scrapyard work		1.5	0.85
Forging cranes		2.0[a]	0.85

b) *Overhead travelling steelworks cranes*

Type and/or application	Impact factor	Duty factor
Ladle cranes	1.2	0.85
Pig/scrap breaking cranes	1.5	0.85
Ingot strippers	1.2	0.85
Soaking pit mould handling cranes	1.5	0.85
Vertical ingot chargers	[b]See below	0.85
Furnace charging cranes	1.5	0.85
Forging cranes	2.0[a]	0.85
Process cranes: on line	1.5	0.85
Process cranes: off line	1.3	0.9
Heavy mill service	1.4	0.9
Service and maintenance cranes	1.2	0.95

[a] This factor may be modified when an overload protection device is fitted.

[b] L_3 is taken as 1.2 (W_1 + W_2) or 1.5 W_1 + 1.0 W_2, whichever is the greater, where:

W_1 is the load in the tongs;

W_2 is the weight of the mast and the tongs.

Table 4 — Impact and duty factors according to crane type and application

c) *Transporters*

Type and/or application	Impact factor	Duty factor
Medium duty: general use	1.3	0.95
Heavy duty: intermittent grabbing and magnet work	1.4	0.90
Extra heavy duty: continuous grabbing and magnet work	1.5	0.85

d) *High pedestal or portal jib cranes and derrick cranes*

Type and/or application		Impact factor	Duty factor
Medium duty in docks and elsewhere: handling general cargo and piece goods		1.3	0.95
Full-time container handling		1.4	0.90
Grabbing or magnet work	intermittent	1.4	0.90
	continuous	1.5	0.85
Shipyard cranes		1.3	0.95

e) *Tower cranes*

Type and/or application	Impact factor	Duty factor
Normal duty: general use on building sites	1.2	0.95
Medium duty: general use at a permanent location, etc.	1.3	0.95

f) *Freight container cranes*

Type and/or application		Impact factor	Duty factor
Freight container transporters	medium duty	1.3	0.95
	heavy duty	1.4	0.90
Goliath for container handling duty		1.3	0.95

g) *Power driven mobile cranes*

Type and/or application	Impact factor	Duty factor
General use	1.2[a]	0.95
Heavy duty: including intermittent grabbing and magnet work	1.3	0.90

[a] For mobile cranes and appliances used for plant erection, lifting hook loads of 75 tonnes up to 100 tonnes, graded impact factors in the range 1.2 to 1.05 may be used where special consideration is given to the duty and the hoist drive characteristics. In this case, for hook loads above 100 tonnes an impact value of 1.05 may be used.

3.1.5 Horizontal loads

3.1.5.1 *Inertia forces.* Inertia forces shall be calculated having regard to the linear or angular accelerations produced by the motion drives or brakes.

3.1.5.2 *Skew loads due to travelling.* When two wheels (or two bogies) roll along a rail, the horizontal forces normal to the rail and tending to skew the structure shall be taken into consideration. The horizontal forces shall be obtained by multiplying the vertical load exerted on each wheel (or bogie) by a coefficient λ (see Figure 1) that depends upon the ratio of the span to the wheel base.

3.1.6 Buffer loads. No design condition is imposed for the crane structure in respect of collision with solid buffers. In cases where adequate resilient buffers are provided for traverse or travel motions and these may come into use at some time in the duty of the crane, when the load is free to swing the buffer force shall be calculated on the assumption that collision takes place with the crane unloaded at 0.7 times the rated speed and that the buffer system is capable of absorbing the energy within its designed stroke. Where the load cannot swing, the buffer effect shall be calculated in the same manner but taking account of the value of the working load. Where a reliable automatic device for slowing down the motion is installed, the appropriate reduced speed shall be used in place of the rated speed.

NOTE Whether buffers should be provided in a particular case is outside the scope of this standard and is usually a matter for decision between the purchaser and the manufacturer having regard to the nature of the duty, dimensional constraints, the size and speed of the crane and the nature of the control and safety devices provided. If resilient buffers are provided, it is recommended that their capacity should be in accordance with the provisions of this clause.

Attention is drawn to the use of buffers with collapsible impact sections for emergency stopping of heavy machines where there is a risk of storm wind forces overcoming the travel brakes or the adhesion of the wheels.

3.1.7 Wind loads

3.1.7.1 *Wind action.* It shall be assumed that the wind can blow horizontally from any direction at a constant velocity, and that there is a static reaction to the loadings it applies to a crane structure.

3.1.7.2 *Wind pressure.* The dynamic wind pressure shall be calculated from $q = 0.613\ V_s^2$

where

q is the dynamic pressure (in N/m^2)

V_s is the design wind speed (in m/s)

A conversion chart covering V_s in knots, mile/h and m/s, and q in lb/ft^2, N/m^2 and kgf/m^2 is given in Figure 2.

3.1.7.3 *Design wind conditions.* Two design wind conditions shall be taken into account in calculating wind loads on cranes as follows.

a) *In-service wind.* This shall be the maximum wind, irrespective of height, in which the crane is designed to operate. The wind loadings shall be assumed to be applied in the least favourable direction in combination with the appropriate service loads specified in **3.1**. “In-service” design wind pressures and corresponding speeds shall be in accordance with Table 5.

Action of wind on suspended load. On all cranes, the action of the wind on the suspended load shall be taken into account.

Where a crane is designed to handle only loads of specific shape and size, the wind loading shall be calculated for the appropriate dimensions and configuration. In other cases, the wind force on the suspended load shall be calculated according to Table 6.

b) *Out-of-service wind.* This shall be the maximum (storm) wind that a crane is designed to withstand when in an out-of-service condition. The speed, and thus the pressure, varies with the height of the crane above the surrounding ground level.

For cranes that are to be used in the UK, the minimum out-of-service wind pressures and corresponding velocities to be used as a basis for design shall be those given in Table 7.

Where special precautions are necessary for a crane to withstand the out-of-service wind, e.g. when lowering the jib of a mobile crane, these shall be clearly stated by the manufacturer.

The out-of-service wind pressures on parts of a crane that are between 20 m and 200 m above ground level shall be calculated from the formula:

$$q = 1340 \left(\frac{h}{20}\right)^{0.16}$$

where

q is the wind pressure (in N/m^2);

h is the height (in m) above ground level.

NOTE The results of this calculation may also be used as an alternative to the figures given in Table 7 for parts of a crane up to 80 m above ground level.

3.1.7.4 *Wind load calculations.* For most complete and part structures, and individual members used in crane structures, the wind load shall be calculated from:

$$F = \gamma A q C_f$$

where

F is the wind load (in N);

γ is a factor related to the design application of the calculated wind load;
for structural strength calculations $\gamma = 1.0$;
for anchoring and securing devices $\gamma = 1.2$;

A is the effective frontal area of the part under consideration, i.e. the shadow area of its solid parts projected onto a plane perpendicular to the wind direction (in m^2);

q is the wind pressure corresponding to appropriate design condition (in N/m^2);

C_f is the force coefficient in the direction of the wind, for the part under consideration.

The total wind load on the structure shall be taken as the sum of the loads on its component parts.

In calculating wind moment for out-of-service conditions, either:

a) the wind pressure at the top shall be taken as constant over the entire height of the structure;

or

b) the structure shall be divided into the horizontal zones of assumed constant pressure given in Table 7 and the appropriate value used for each zone.

3.1.7.5 *Individual members, single lattice frames, etc: force coefficients.* Force coefficients for individual members, single lattice frames, machinery houses, crabs, etc. shall be in accordance with Table 8.

NOTE The values for individual members vary according to the aerodynamic slenderness and, in the case of large box sections, with the section ratio. Aerodynamic slenderness and section ratio are defined in Figure 3.

Where a frame is made up of flat-sided and circular sections, or of circular sections in both flow regimes ($DV_s < 6$ m^2/s and $DV_s \geqslant 6$ m^2/s where D is the diameter of the section in metres) the appropriate force coefficients shall be applied to the corresponding frontal areas.

3.1.7.6 *Multiple members, multiple frames, etc: shielding factors.* Where parallel frames or members are positioned so that the windward parts have a shielding effect on those behind them, the wind load on the unsheltered parts shall be calculated from the formula given in **3.1.7.4**, taking A as the area in square metres of the windward frame or member plus the unsheltered parts of those behind it. The wind load on sheltered parts shall be calculated from:

$$F_s = \gamma A_s q C_f \phi$$

where

γ, qC_f are as defined in **3.1.7.4**;

F_s is the wind load on the sheltered parts (in N);

A_s is the area of the sheltered parts under consideration (in m^2);

ϕ is the shielding factor given in Table 9 according to the solidity ratio of the front frame and the spacing ratio. These ratios are defined in Figure 3.

Where there are a number of identical frames or members spaced equidistantly behind each other such that each frame completely shields those behind it, the wind loads shall be calculated (in N) as follows.

On the 1st frame $F_1 = \gamma A q C_f$

On the 2nd frame $F_2 = \gamma A q C_f \phi$

On the nth frame (where n is between 3 and 8) $F_n = \gamma A q C_f \phi^{(n-1)}$

On the 9th and subsequent frames $F_9 = \gamma A q C_f \phi^8$

where there are up to 9 frames the total wind load

$$F_{total} = \gamma A q C_f \left[1 + \phi + \phi^2 + \phi^3 \ldots\ldots + \phi^{(n-1)}\right] \quad (n \leqslant 9)$$

$$= \gamma A q C_f \left(\frac{1-\phi^n}{1-\phi}\right)$$

where there are more than 9 frames the total wind load

$$F_{total} = \gamma A q C_f \left[\left(\frac{1-\phi^9}{1-\phi}\right) + (n-9)\,\phi^8\right] \quad (n > 9)$$

where

F, γ, q, C_f	are as defined in **3.1.7.4**;
A	is the area of one frame (in m^2);
ϕ	is as defined above.

NOTE For design purposes the value of the term ϕ^n is assumed to have a lower limit of 0.10. It is taken as 0.10 whenever numerically $\phi^n < 0.10$.

3.1.7.7 *Lattice towers of square cross section.* In calculating the "face-on" wind load on square towers, the solid area of the windward face shall be multiplied by the following overall force coefficients:

for towers composed of flat-sided sections $1.7\gamma q(1 + \phi)$

for towers composed of circular sections

where $DV_s < 6\ m^2/s$ $1.2\gamma q(1 + \phi)$

where $DV_s \geqslant 6\ m^2/s$ $1.4\gamma q$

The value of ϕ shall be taken from Table 9 for $a/b = 1$ according to the solidity ratio of the windward face.

The maximum wind load on a square tower, which occurs when the wind blows on to a corner, shall be taken as 1.2 times the face-on load.

3.2 Loads due to climatic conditions and natural phenomena

For conditions of service outside the UK, loads due to wind, snow and temperature variation shall be taken into account as appropriate.

NOTE 1 These should be the subject of agreement between the purchaser and the manufacturer.

NOTE 2 Attention is drawn to the fact that the laws or requirements of a country may require the inclusion of earthquake forces. Such forces should be determined in accordance with those requirements and included in the loads to be considered in design.

Table 5 — In-service design wind pressures

Type of crane	In-service design wind pressure	Approximate equivalent in-service wind speed
	N/m^2	m/s
a) Cranes that are easily secured against wind action, and are designed for operation in light winds only (e.g. cranes of low chassis height with booms that can be readily lowered to the ground)	125	14
b) All normal types of crane installed in the open	250	20
c) Transporter-type unloaders that have to continue to work in high winds	450	27

Table 6 — Wind force on suspended load

SWL	Type of crane from Table 5	Wind force on suspended load
		N
up to 1	a) b) c)	125 250 450
from 1 to 5	a) b) c)	125 per tonne of SWL 250 per tonne of SWL 450 per tonne of SWL
from 5 to 25	a) b) c)	for SWL ⩽ 8 tonne: 125 per tonne of SWL for SWL > 8 tonne: a maximum of 1 000 a further 125 per tonne of SWL a further 225 per tonne of SWL
25 and above	a) b) c)	a maximum of 1 000 a maximum of 3 750 a maximum of 6 750

Table 7 — Out-of-service design wind pressures

Height above ground level	Out-of-service design wind pressure	Approximate equivalent out-of-service wind speed
m	N/m^2	m/s
Parts of crane up to 30	1 340	47
Parts of crane between 30 and 60	1 530	50
Parts of crane between 60 and 80	1 670	52

Table 8 — Force coefficients C_f

Type	Description		Aerodynamic slenderness l/b or l/D[a]					
			5	10	20	30	40	50
Individual members	Rolled sections, rectangular hollow sections, flat plates fabricated box sections with b and D not greater than 0.5 m		1.3	1.35	1.6	1.65	1.7	1.8
		Section ratio b/d[a]						
	Fabricated box sections with b or d greater than 0.5 m	⩾ 2	1.55	1.75	1.95	2.1	2.2	
		1	1.40	1.55	1.75	1.85	1.9	
		0.5	1.0	1.2	1.3	1.35	1.4	
		0.25	0.8	0.9	0.9	1.0	1.0	
	Circular sections							
	Where $DV_s < 6\ m^2/s$		0.75	0.80	0.90	0.95	1.0	1.1
	Where $DV_s \geqslant 6\ m^2/s$		0.60	0.65	0.70	0.70	0.75	0.8
Single lattice frames	Flat-sided sections	1.7						
	Circular sections							
	Where $DV_s < 6\ m^2/s$	1.2						
	Where $DV_s \geqslant 6\ m^2/s$	0.8						
Machinery houses, crabs, etc.	Rectangular clad structures on ground or solid base (air flow beneath structure prevented)	1.1						

[a] See Figure 3.

Table 9 — Shielding factors ϕ

Spacing ratio[a] a/b	Solidity ratio[a] A/A_e					
	0.1	0.2	0.3	0.4	0.5	⩾ 0.6
0.5	0.75	0.4	0.32	0.21	0.15	0.1
1.0	0.92	0.75	0.59	0.43	0.25	0.1
2.0	0.95	0.8	0.63	0.5	0.33	0.2
4.0	1	0.88	0.76	0.66	0.55	0.45
5.0	1	0.95	0.88	0.81	0.75	0.68
6.0	1	1	1	1	1	1

[a] See Figure 3.

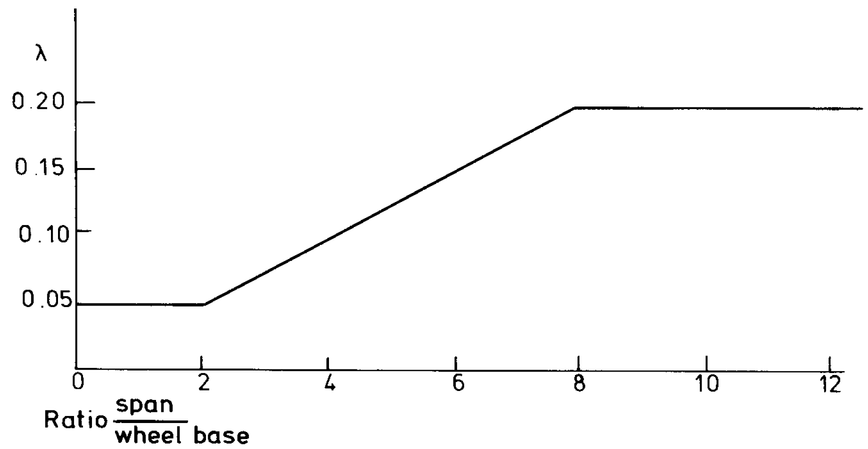


Figure 1 — Skew load coefficients

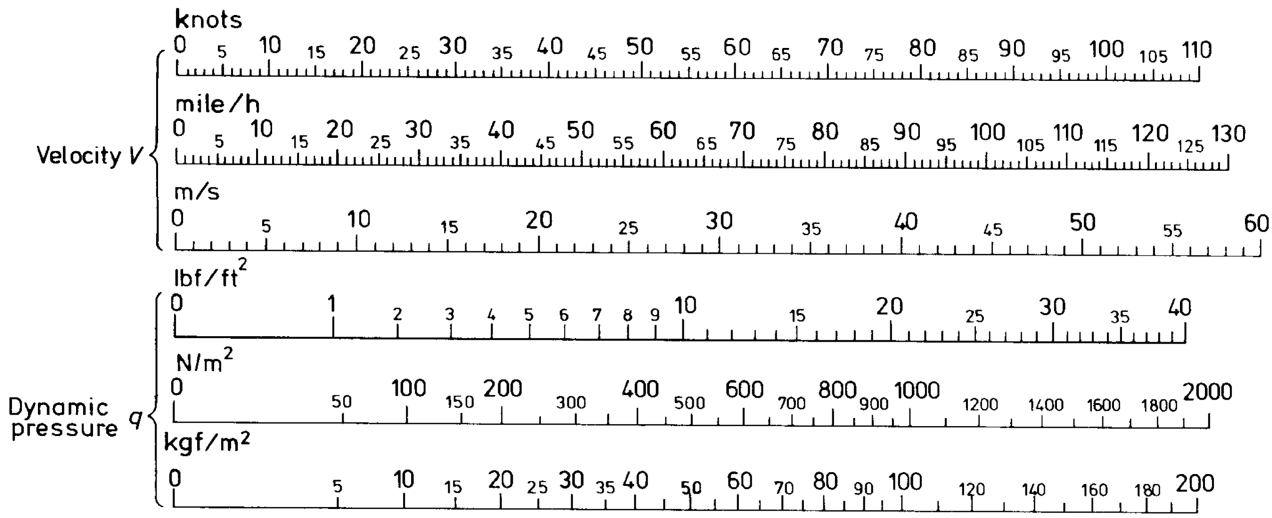
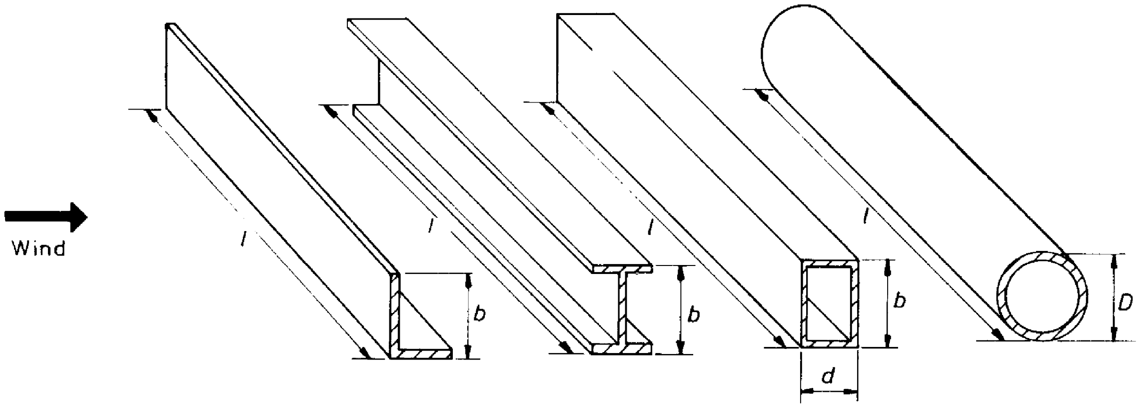


Figure 2 — Conversion chart for wind speed and pressure

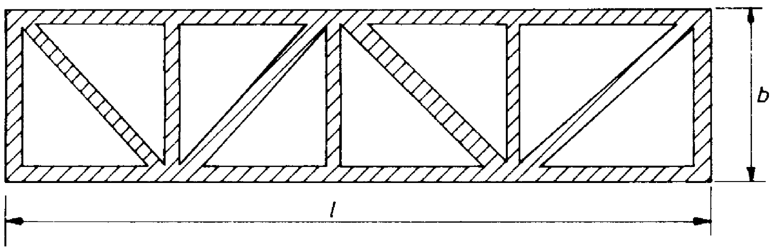
(a) Aerodynamic slenderness and section ratio



aerodynamic slenderness = $\dfrac{\text{length of member}}{\text{breadth of section across wind front}} = \dfrac{l}{b}$ or $\dfrac{l}{D}$

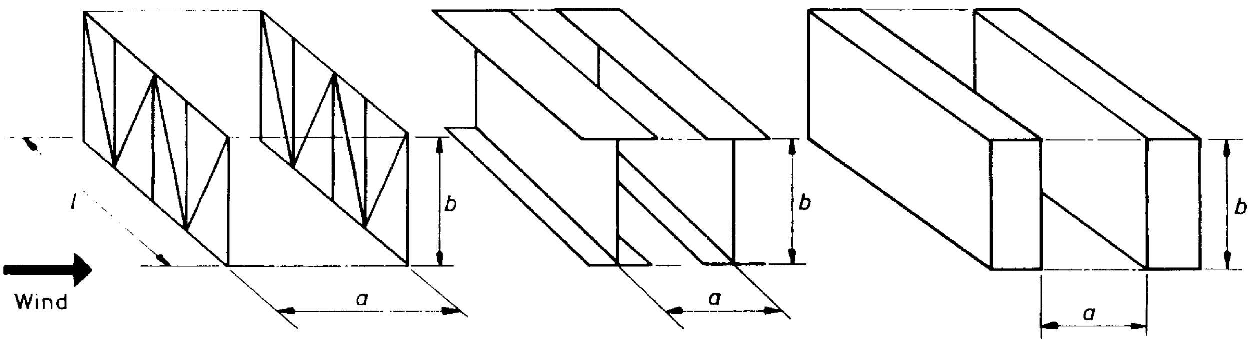
section ratio (for box sections) = $\dfrac{\text{breadth of section across wind front}}{\text{depth of section parallel to wind flow}} = \dfrac{b}{d}$

(b) Solidity ratio



solidity ratio $\dfrac{A}{A_e} = \dfrac{\text{area of solid parts (shown shaded)}}{\text{enclosed area}} = \dfrac{\Sigma A_{members}}{b \times l}$

(c) Spacing ratio



spacing ratio = $\dfrac{\text{distance between facing sides}}{\text{breadth of member across wind front}} = \dfrac{a}{b}$

Figure 3 — Definitions: aerodynamic slenderness, section ratio, solidity ratio and spacing ratio

Section 4. Selection of steel, minimum thickness and working stresses

4.1 Selection of steel

4.1.1 Steel shall be selected from either:

a) standard structural steels to BS 4360; or

b) other steels, provided that the crane manufacturer shows that they have comparable properties to steels to BS 4360 and they have been subjected to equivalent tests.

4.1.2 Where thicknesses of steel are specified that exceed the maximum values given in BS 4360 for Charpy V-notch impact tests, the impact test requirements on standard specimens shall not be less than the value given in BS 4360 for the type of steel under consideration on the standard specimen.

4.1.3 Where cranes are to be used at low temperatures such that brittle fracture might occur, the material used for loadbearing members shall have specified low temperature impact properties, adequate to meet the service conditions inherent in the design.

4.1.4 For temperate or tropical conditions, steels having no guaranteed impact test values are acceptable, with the exception of the following, which shall not be used unless impact or other tests show that the material is suitable for service:

a) plates and sections above 30 mm thickness where brittle fracture might occur under tension loads;

b) plates and sections above 25 mm thickness where brittle fracture under tension loads would result in major structural collapse.

NOTE For further information regarding selection of steels to counter brittle fracture see chapter 2 of BS 449-2:1969.

4.2 Minimum thickness of plates and sections

The proportioning of members of crane structures shall follow from consideration of the stresses engendered by service conditions, and shall have regard to other practical considerations including the requirements of manufacturing processes, vulnerability to accidental damage, the incidence of corrosion in relation to protective coatings used, etc.

NOTE This standard does not impose minimum thicknesses, but reference should be made to other standards covering specific types of crane. Attention is drawn to the requirements laid down in BS 4395 and BS 4604 for the thicknesses of members at joints made with high strength friction grip bolts.

4.3 Working stresses

4.3.1 Duty factor. The duty factors given in Table 4 according to the type of crane and its application shall be applied to all basic stresses set out in **5.1**, **5.2** and section 6.

NOTE A duty factor applied to all basic stresses allows for a degree of imperfection in the methods of calculation of stresses and for unforeseen contingencies. In particular, there is a greater probability in high speed and in more intensively used cranes handling loads near their maximum capacity that combined stresses higher than those derived from calculation will occur. It is necessary to take account of this when checking in relation to the elastic limit, even though the separate fatigue calculation also deals with the influence of intensity of service conditions.

4.3.2 Permissible working stresses. The calculated stresses in each part of the structure due to the load combinations specified in **3.1** shall not exceed any of the following.

a) *Under the load combinations specified in* **3.1.2.1**. The basic stress multiplied by the duty factor.

b) *Under the load combinations specified in* **3.1.2.2**. The basic stress multiplied by the duty factor and multiplied by a factor of 1.13.

c) *Under the load combinations specified in* **3.1.2.3**. The permissible fatigue stress.

d) *Under the load combinations specified in* **3.1.2.4**, **3.1.2.5** and **3.1.2.6**. The basic stress multiplied by a factor of 1.36.

Section 5. Stresses in structural components

5.1 Individual members, rolled sections, hollow sections and members with plated webs: verification relative to the yield stress

5.1.1 Basic stresses. Basic stresses for steels to BS 4360 for use in the application of this standard shall comply with **5.1.2** to **5.1.8**.

NOTE 1 In general, the basic stress is expressed as a proportion of the yield stress of the grade of steel under consideration. The formulae for deriving basic stresses and tabulated values are both given.

NOTE 2 *Members subjected to secondary stresses.* Relaxations in some of the requirements of **5.1.2** to **5.1.8** are allowed in cases where secondary stresses are calculated and taken into account in the design (see **5.3**).

If steels with higher tensile strengths than those of BS 4360 steels are used, the specific requirements of Appendix B shall be met.

5.1.2 Members subject to simple axial tension. The basic tensile stress $P_{at.bas}$ (in N/mm^2) shall not exceed the value

$P_{at.bas}$ (on net section) = 0.6 Y_s

where Y_s is the yield stress of the steel under consideration (in N/mm^2).

Tabulated values of $P_{at.bas}$ for the range of steels covered by BS 4360 are given in Table 10.

The maximum widths of tension flange plates with stiffened or unstiffened edges are specified in **7.2.3**.

5.1.3 Members subject to simple axial compression. The basic compressive stress $P_{ac.bas}$ shall not exceed $P_{at.bas}$ as defined in **5.1.2** or the value (in N/mm^2) obtained from

$P_{ac.bas} = 0.6\ F_{crip}$

where F_{crip} is the applied stress at failure of a member subjected to overall flexural buckling due to axial compression as given by the equation:

$$F_{crip} = \frac{Y_s + (\eta + 1)C_o}{2} - \sqrt{\left\{\left(\frac{Y_s + (\eta + 1)C_o}{2}\right)^2 - Y_s C_o\right\}}$$

where

F_{crip} is the applied stress at failure (in N/mm^2)

C_o is the Euler critical stress $\left(= \frac{\pi^2 E}{s^2}\right)$

Y_s is the yield stress of the steel under consideration. For sections fabricated from plate by welding, the yield strength Y_s shall be reduced by 25 N/mm^2.

NOTE This provision need not be applied to welded compound rolled sections or to rolled sections with welded flange cover plates.

E is Young's modulus (= 205 000 N/mm^2)

η is the Perry coefficient ($= \alpha(s - s_o) \times 10^{-3}$, but not less than zero)

α is the Robertson constant from Table 11

s is the slenderness ratio ($= l/r$)

s_o is the limiting slenderness ratio for stub columns $= 0.2\pi\sqrt{E/Y_s}$

r is the radius of gyration about the appropriate axis.

l is the effective length relative to the same axis, as defined in **7.1**.

Tabulated values of F_{crip} for the range of steels covered by BS 4360 are given in Table 12 for the values of α given in Table 11.

For slenderness ratio less than s_o, $F_{crip} = Y_s$.

The effective and maximum widths of plates in compression are specified in **7.2.1** and **7.2.2** respectively.

The slenderness ratio s for any strut shall be obtained by dividing its effective length l as given in **7.1** by the minimum radius of gyration r of any cross section within the middle third of the length. Where the end fixing conditions of the strut in the X and Y planes are different, its effective lengths in these planes will also differ.

Table 10 — Basic stresses in structural members

All stresses are in newtons per square millimetre.

Description	Basic stress for steels to BS 4360										
	Grade 43 steel with a yield stress of				Grade 50 steel with a yield stress of			Grade 55 steel with a yield stress of			
	215	**230**	**245**	**280**	**325**	**340**	**355**	**400**	**415**	**430**	**450**
Parts in axial tension On effective sectional area (see **5.1.2**)	129	138	147	168	195	204	213	240	249	258	270
Parts in axial compression On effective gross section (see **5.1.3**)	129	138	147	168	195	204	213	240	249	258	270
Parts in bending, (tension or compression) (see **5.1.4**) On effective sectional area for extreme fibre stress a) for plates, flats, tubes, rounds, square and similar sections	140	150	160	182	211	221	231	260	270	280	293
b) for rolled beams, channels angles and tees, and for plate girders with single or multiple webs with: d_1/t not greater than 85 for steel of grade 43 d_1/t not greater than 75 for steel of grade 50 d_1/t not greater than 65 for steel of grade 55	133	143	152	174	202	211	220	248	257	267	279
c) for plate girders with single or multiple webs with: d_1/t greater than 85 for steel of grade 43 d_1/t greater than 75 for steel of grade 50 d_1/t greater than 65 for steel of grade 55	127	136	145	165	192	201	209	236	245	254	266
In the above, d_1 is the clear distance between flange angles or, where there are no flange angles, between flanges (ignoring fillets); where, however, tongue plates having a thickness not less than twice the thickness of the web plate are used, d_1 is the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thickness of the tongue plates, whichever is the lesser t is the web thickness.											
Parts in shear Average shear stress on the gross effective sectional area of webs of plate girders, rolled beams, channels, angles and tees (see **5.1.5**)	80	85	91	104	120	126	131	148	154	159	167
Parts in bearing On flat surfaces, fixed axles and pins (see **5.1.6**)	172	184	196	224	260	272	284	320	332	344	360
Parts subjected to shear combined with other stresses. Basic equivalent stress (see **5.1.7**)	200	214	228	260	302	316	330	372	386	400	419
NOTE 1 Basic stresses corresponding to intermediate values of yield stress may be obtained by interpolation. NOTE 2 To obtain the permissible working stress, the appropriate basic stress has to be multiplied by the factors given in **4.3** that correspond to the loading conditions under consideration.											

5.1.4 Members subject to bending

5.1.4.1 *Areas in tension.* The basic tensile bending stress $P_{bt.bas}$ (in N/mm²) shall not exceed the following values:

a) for plates, flats, tubes, rounds, square and similar sections bending about their minor axis. $P_{bt.bas} = 0.65Y_s$

b) for rolled beams, channels angles and tees, and for plate girders with single or multiple webs with
  d_1/t not greater than 85 for steel of grade 43
  d_1/t not greater than 75 for steel of grade 50
  d_1/t not greater than 65 for steel of grade 55
  $P_{bt.bas} = 0.62Y_s$

c) for plate girders with single or multiple webs with
  d_1/t greater than 85 for steel of grade 43
  d_1/t greater than 75 for steel of grade 50
  d_1/t greater than 65 for steel of grade 55
  $P_{bt.bas} = 0.59Y_s$

where Y_s is as defined in **5.1.2** and d_1 and t are as defined in Table 10 for parts in bending. Tabulated values of $P_{bt.bas}$ for the range of steels covered by BS 4360 are given in Table 10.

The maximum widths of tension flange plates with stiffened or unstiffened edges are specified in **7.2.3**.

Table 11 — Values of Robertson constant α for struts of various sections

Type of section	Thickness of flange or plate	Axis of buckling	α
Rolled I section (universal beams, UB)		xx yy	2.0 3.5
Rolled H section (universal columns, UC): See note 1	up to 40 mm over 40 mm	xx yy xx yy	3.5 5.5 5.5 8.0
Welded plate I or H sections: See notes 1, 2,and 3	up to 40 mm over 40 mm	xx yy xx yy	3.5 5.5 3.5 8.0
Rolled I or H sections with welded flange cover plates: See notes 1 and 4		xx yy	3.5 2.0
Welded box sections: See notes 1, 3 and 5	up to 40 mm over 40 mm	any any	3.5 5.5
Rolled channel sections, rolled angle sections and T-bars (rolled or cut from UB or UC)		any	5.5
Hot-rolled structural hollow sections		any	2.0
Rounds, square and flat bars: See note 1	up to 40 mm over 40 mm	any any	3.5 5.5
Compound rolled sections (two or more I, H or channel sections, I section plus channel, etc.)		any	5.5
Two rolled angle, channel or tee sections back-to-back		any	5.5
Two rolled sections laced or battened		any	5.5
Composite members of closed lattice construction		any	2.0

NOTE 1 For thicknesses between 40 mm and 50 mm the value of F_{crip} may be taken as the average of the value for thicknesses less than 40 mm and the value for thicknesses greater than 40 mm.

NOTE 2 For welded plate I or H sections where it can be guaranteed that the edges of the flanges will only be flame-cut, α = 3.5 may be used for buckling about the y-y axis for flanges up to 40 mm thick and α = 5.5 for flanges over 40 mm thick.

NOTE 3 Yield strength for sections fabricated from plate by welding reduced by 25 N/mm^2.

NOTE 4 To qualify under the category "rolled I or H section with welded flange cover plates" the widths of the flange and the plate have to be within the greater of 25 mm or 25 % of the larger width. If the smaller width is less than 25 % of the larger, the category "welded plate I or H sections" shall apply, otherwise the category shall be taken as "rolled I section" or "rolled H section" as appropriate.

NOTE 5 "Welded box sections" includes those fabricated from four plates, two angles or an I or H section and two plates but not box sections composed of two channels or plates with welded longitudinal stiffeners.

Table 12 — Values of F_{crip} for steels to BS 4360

All stresses are in newtons per square millimetre

	Grade 43 steel with a yield stress in N/mm² of				Grade 50 steel with a yield stress in N/mm² of			Grade 55 steel with a yield stress in N/mm² of			
	215	230	245	280	325	340	355	400	415	430	450
Slenderness ratio l/r	**Limiting slenderness ratio** s_0 **below which** $F_{crip} = Y_s$										
	19	19	18	17	16	15	15	14	14	14	13
a) α = 2.0 (see **5.1.3**)											
s_0	215	230	245	280	325	340	355	400	415	430	450
20	213	228	243	277	322	337	350	394	409	424	444
30	208	223	237	271	314	329	342	385	399	413	432
40	203	217	231	264	305	319	331	372	386	399	417
50	197	210	224	254	293	306	317	354	366	378	394
60	190	202	214	242	276	288	297	327	337	346	358
70	180	191	202	226	253	262	268	290	296	302	310
80	168	178	186	205	225	230	234	248	251	255	259
90	155	162	168	181	194	198	200	208	210	212	215
100	139	145	149	158	166	169	170	175	176	178	179
110	124	128	131	137	143	144	145	148	149	150	151
120	110	112	115	119	123	124	124	127	127	128	129
130	97	99	101	104	106	107	108	109	110	110	111
140	86	87	89	91	93	93	94	95	95	96	96
150	77	78	79	80	82	82	83	83	84	84	84
160	68	69	70	71	72	73	73	74	74	74	74
170	61	62	63	64	65	65	65	66	66	66	66
180	55	56	56	57	58	58	58	59	59	59	59
190	50	51	51	52	52	52	53	53	53	53	53
200	46	46	46	47	47	48	48	48	48	48	48
210	42	42	42	43	43	43	43	44	44	44	44
220	38	38	39	39	39	40	40	40	40	40	40
230	35	35	35	36	36	36	36	37	37	37	37
240	32	33	33	33	33	33	33	34	34	34	34
b) α = 3.5 (see **5.1.3**)											
s_0	215	230	245	280	325	340	355	400	415	430	450
20	211	226	241	275	320	334	346	390	405	419	439
30	204	218	232	265	307	321	332	374	388	402	420
40	195	209	222	253	292	305	316	354	367	380	396
50	186	198	211	239	275	286	295	329	340	351	365
60	175	186	198	223	253	263	270	297	306	314	325
70	163	173	183	203	228	235	241	260	266	272	279
80	150	158	166	182	201	206	210	223	227	231	235
90	136	143	149	161	174	178	181	190	192	195	197
100	123	128	132	141	151	153	155	161	163	164	166
110	110	113	117	124	130	132	133	138	139	140	141
120	98	101	103	108	113	114	115	118	119	120	121
130	87	89	91	95	99	100	100	103	104	104	105
140	78	79	81	84	87	88	88	90	90	91	92
150	70	71	72	75	77	77	78	79	80	80	81
160	63	64	65	67	68	69	69	70	71	71	71
170	57	57	58	60	61	62	62	63	63	63	64
180	51	52	53	54	55	55	56	56	57	57	57
190	47	47	48	49	50	50	50	51	51	51	52
200	43	43	44	44	45	46	46	46	46	47	47
210	39	39	40	41	41	42	42	42	42	42	43
220	36	36	37	37	38	38	38	39	39	39	39
230	33	33	34	34	35	35	35	35	36	36	36
240	31	31	31	32	32	32	32	33	33	33	33

Table 12 — Values of F_{crip} for steels to BS 4360

	Grade 43 steel with a yield stress in N/mm² of				Grade 50 steel with a yield stress in N/mm² of			Grade 55 steel with a yield stress in N/mm² of			
	215	**230**	**245**	**280**	**325**	**340**	**355**	**400**	**415**	**430**	**450**
Slenderness ratio l/r	**Limiting slenderness ratio s_O below which $F_{crip} = Y_s$**										
	19	**19**	**18**	**17**	**16**	**15**	**15**	**14**	**14**	**14**	**13**
c) α = 5.5 (see **5.1.3**)											
S_O	215	230	245	280	325	340	355	400	415	430	450
20	210	224	239	273	317	331	342	358	399	414	433
30	198	211	225	257	298	311	321	361	374	387	405
40	186	198	211	240	277	289	298	334	345	357	373
50	173	185	196	222	254	265	272	303	313	322	335
60	160	170	180	202	230	238	244	268	276	283	293
70	147	155	164	182	204	211	215	233	239	244	251
80	133	140	147	162	179	184	187	200	204	208	213
90	120	126	131	143	156	159	162	171	174	177	180
100	108	112	117	126	135	138	140	147	149	150	153
110	96	100	104	111	118	120	121	126	128	129	131
120	86	89	92	97	103	105	106	109	111	112	113
130	77	80	82	86	91	92	93	96	96	97	98
140	69	71	73	77	80	81	82	84	85	85	86
150	63	64	66	68	71	72	73	74	75	76	76
160	57	58	59	61	64	64	65	66	67	67	68
170	51	52	53	55	57	58	58	59	60	60	61
180	47	48	49	50	52	52	53	54	54	54	55
190	43	44	44	46	47	47	48	49	49	49	49
200	39	40	40	42	43	43	43	44	44	45	45
210	36	37	37	38	39	39	40	40	41	41	41
220	33	34	34	35	36	36	36	37	37	37	37
230	31	31	32	32	33	33	34	34	34	34	34
240	29	29	29	30	31	31	31	31	32	32	32
d) α = 8.0 (see **5.1.3**)											
S_O	215	230	245	280	325	340	355	400	415	430	450
20	207	222	236	270	313	327	336	378	393	407	426
30	191	204	217	248	287	300	308	346	358	371	388
40	175	187	199	226	261	272	279	312	323	334	348
50	160	170	181	204	234	244	249	277	286	295	306
60	145	154	163	183	208	215	220	242	249	255	264
70	131	139	146	163	182	189	192	209	214	219	225
80	118	124	130	144	159	164	167	179	183	187	191
90	106	111	116	127	139	142	145	154	157	160	163
100	95	99	103	112	121	124	126	133	135	137	139
110	85	88	92	99	106	108	110	115	117	118	120
120	76	79	82	87	93	95	96	100	102	103	104
130	68	71	73	78	82	84	85	88	89	90	91
140	62	64	66	69	73	74	75	78	79	80	80
150	56	58	59	62	66	66	67	69	70	71	71
160	51	52	54	56	59	60	60	62	63	63	64
170	46	47	49	51	53	54	54	56	56	57	57
180	42	43	44	46	48	49	49	50	51	51	52
190	39	40	41	42	44	44	45	46	46	46	47
200	36	37	37	39	40	40	41	42	42	42	43
210	33	34	34	36	37	37	37	38	39	39	39
220	31	31	32	33	34	34	34	35	35	36	36
230	28	29	29	30	31	32	32	32	33	33	33
240	26	27	27	28	29	29	29	30	30	30	31

5.1.4.2 *Areas in compression*

5.1.4.2.1 *Maximum widths of plates.* The maximum widths of plates in compression shall be as specified in **7.2.2**.

5.1.4.2.2 *For sectional shapes with I_y equal to or greater than I_x.* Where

I_y is the moment of inertia of the whole section about the axis lying in the plane of bending (the y-y axis), and

I_x is the moment of inertia of the whole section about the axis normal to the plane of bending (the x-x axis),

the basic compressive bending stress shall not exceed the value of $P_{bt.bas}$ given in **5.1.4.1**.

5.1.4.2.3 *For sectional shapes with I_y smaller than I_x*

5.1.4.2.3.1 Where I_y and I_x are as defined in **5.1.4.2.2**, the basic compressive bending stress $P_{bc.bas}$ shall not exceed $P_{bt.bas}$ as defined in **5.1.4.1**, or the value of $P_{bc.bas}$ corresponding to C_s, the critical stress in the compression element (in N/mm^2) calculated as set out in **5.1.4.2.3.2** and **5.1.4.2.3.3**.

5.1.4.2.3.2 For sections with a single web, including *I* sections with stiffened or unstiffened edges, channels, angles, tees, etc., but excluding *I* sections where the thickness of one flange is more than 3 times the thickness of the other flange.

a) Where the flanges have equal moments of inertia about the y-y axis

$$C_s = \left(1644\frac{r_y}{l}\right)^2 \sqrt{\left\{1 + \frac{1}{20}\left(\frac{lT}{r_y D}\right)^2\right\}} = A$$

except that the value of C_s calculated above shall be increased by 20 % for rolled beams and channels, and for plate girders provided that:

T/t	is not greater than 2 and
d_1/t	is not greater than 85, for steel of grade 43 to BS 4360
d_1/t	is not greater than 75, for steel of grade 50 to BS 4360
d_1/t	is not greater than 65, for steel of grade 55 to BS 4360

where

l is the effective length of compression flange (see **7.1.3**);

r_y is the radius of gyration about the y-y axis of the gross section of the member, at the point of maximum bending moment;

D is the overall depth of member, at the point of maximum bending moment;

T is the effective thickness of the compression flange. $T = K_1 \times$ mean thickness of the horizontal portion of the compression flange at the point of maximum bending moment. (For rolled sections, $T = K_1 \times$ thickness given in reference books.) The coefficient K_1 makes allowance for reduction in thickness or breadth of flanges between points of effective lateral restraint and depends on N, the ratio of the total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between such points of restraint.

d_1 and t are as defined in Table 10 for parts in bending.

Flanges shall not be reduced in breadth to give a value of N lower than 0.25.

Values of K_1 for different values of N are given in Table 13.

Where the value of N calculated for the compression flange alone is smaller than that when both flanges are combined, this smaller value of N shall be used.

Table 13 — Values of K_1

N	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K_1	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

b) Where the moment of inertia of the compression flange about the y-y axis exceeds that of the tension flange

$$C_s = \left(1644\frac{r_y}{l}\right)^2 \sqrt{\left\{1 + \frac{1}{20}\left(\frac{lT}{r_y D}\right)^2\right\}} + K_2\left(1644\frac{r_y}{l}\right)^2$$
$$= A + K_2 B$$

where

l, r_y and D are as defined in a) above;

T is the effective thickness of flange; $T = K_1 \times$ mean thickness of the horizontal portion of the flange of greater moment of inertia about the y-y axis of the member at the point of maximum bending moment, where K_1 is obtained from Table 13;

K_2 is a coefficient to allow for inequality of tension and compression flanges, and depends on M, the ratio of the moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the compression and tension flanges, each calculated about its own axis parallel to the y-y axis of the member, at the point of maximum bending moment.

NOTE For flanges of equal moment of inertia $M = 0.5$ and $K_2 = 0$. For tees and angles $M = 1.0$ and $K_2 = 0.5$.

Values of K_2 for different values of M are given in Table 14.

c) Where the moment of inertia of the tension flange about the y-y axis exceeds that of the compression flange

$$C_s = \left[\left(1644\frac{r_y}{l}\right)^2 \sqrt{\left\{1 + \frac{1}{20}\left(\frac{lT}{r_y D}\right)^2\right\}} + K_2\left(1644\frac{r_y}{l}\right)^2\right]\frac{Y_c}{Y_t}$$
$$= (A + K_2 B)\frac{Y_c}{Y_t}$$

where

l, r_y, D, T and K_2 are as defined above;

Y_c is the distance from the neutral axis of girder to extreme fibre in compression;

Y_t is the distance from the neutral axis of girder to extreme fibre in tension.

Values of K_2 for different values of M are given in Table 14.

NOTE For tees and angles, $M = 0$ and $K_2 = -1$.

Table 14 — Values of K_2

M	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K_2	0.5	0.4	0.3	0.2	0.1	0.0	– 0.2	– 0.4	– 0.6	– 0.8	– 1.0

Table 15 gives values of A and B for different ratios of l/r and D/T to be used for calculating C_s in newtons per square millimetre.

Table 16 gives values of $P_{bc.bas}$ for different values of C_s.

5.1.4.2.3.3 For sections other than those described in **5.1.4.2.3.2**.

a) Where the section is symmetrical about the x-x axis, C_s shall be calculated from the formula given in Appendix D.

b) Where the section is not symmetrical about the x-x axis, C_s shall be calculated using either:

1) the formula given in **5.1.4.2.3.2**, which will give conservative values; or

2) more precise methods.

5.1.4.2.4 *Thin-walled fabricated box sections.* These sections constitute a special design case and, in addition to meeting the requirements of **5.1.4.2.2** and **5.1.4.2.3**, shall be given detailed consideration in respect of the following factors.

a) Residual stresses due to welding.

b) Distortion and misalignment of panels during fabrication.

c) Diaphragms need to be of adequate proportions to prevent distortion of the cross section under the applied loads.

d) Panel stiffeners and stub stiffeners used at loading points need adequate restraint at their ends if they are assumed to participate in carrying the reaction loads.

e) Shear stresses due to pure torsion need to be considered in conjunction with the direct compressive stresses.

A suitable method for the design of stiffened compression panels in which factors a), b) and e) are taken into account is given in Appendix E. Where other methods are used, these factors shall be taken into account by detailed calculation or by selecting conservative factors of safety.

5.1.5 Members subjected to shear

5.1.5.1 *Rolled beams, channels, angles and tees.* The basic average shear stress $P_{q.bas}$ (in N/mm^2) on the effective sectional area shall not exceed the value

$$P_{q.bas} = 0.37\ Y_s$$

where Y_s is as defined in **5.1.2**.

Tabulated values of $P_{q.bas}$ for the range of steels covered by BS 4360 are given in Table 10.

5.1.5.2 *Solid web plates.* Solid web plates and stiffeners shall be proportioned in accordance with **7.3**.

The basic average shear stress $P_{q.bas}$ (in N/mm^2) on the effective sectional area of a solid web shall not exceed the value given in **5.1.5.1** or that given by the following equations.

For steel of grade 43 to BS 4360

$$P_{q.bas} = 91\left[1.3 - \frac{b/t}{250\left\{1 + \tfrac{1}{2}\left(\frac{b}{a}\right)^2\right\}}\right]$$

For steel of grade 50 to BS 4360

$$P_{q.bas} = 131\left[1.3 - \frac{b/t}{200\left\{1 + \tfrac{1}{2}\left(\frac{b}{a}\right)^2\right\}}\right]$$

For steel of grade 55 to BS 4360

$$P_{q.bas} = 167\left[1.3 - \frac{b/t}{180\left\{1 + \tfrac{1}{2}\left(\frac{b}{a}\right)^2\right\}}\right]$$

In the above formulae:

a is the greater clear dimension of the web in a panel, not greater than $270t$;

b is the lesser clear dimension of the web in a panel, not greater than $180t$;

t is the thickness of web.

Tabulated values of $P_{q.bas}$ for stiffened webs for varying ratios of depth of panel d to thickness of web t and various spacings of stiffeners are given in Table 17 for the range of steels covered by BS 4360. The depth of panel d is defined as follows.

For webs without horizontal stiffeners, d is the clear distance between flange angles or, where there are no flange angles, between flanges (ignoring fillets); where tongue plates having a thickness not less than twice the thickness of the web plate are used, d is the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thicknesses of the tongue plates, whichever is the less.

For webs with horizontal stiffeners, d is the clear distance between the tension flange (angles or flange plate or tongue plate) and the horizontal stiffener.

5.1.6 Members subjected to bearing. The basic bearing stress $P_{b.bas}$ (in N/mm^2) on flat surfaces and on the projected area of fixed axles and pins shall not exceed the value

$$P_{b.bas} = 0.80\ Y_s$$

where Y_s is as defined in **5.1.2**.

Tabulated values of $P_{b.bas}$ for the range of steels covered by BS 4360 are given in Table 10.

Table 15 — Values of *A* and *B* to be used for calculating values of C_s in newtons per square millimetre where

$$A = \left(1644\frac{r_y}{l}\right)^2 \sqrt{\left\{1 + \frac{1}{20}\left(\frac{lT}{r_y D}\right)^2\right\}}$$

$$B = \left(1644\frac{r_y}{l}\right)^2$$

NOTE Where flanges are equal and of constant cross section $C_s = A$.

l/r_y	*A*															*B*
	D/T															
	8	**10**	**12**	**14**	**16**	**18**	**20**	**25**	**30**	**35**	**40**	**50**	**60**	**80**	**100**	
25	5 276	4 954	4 771	4 656	4 581	4 528	4 490	4 431	4 399	4 379	4 366	4 351	4 343	4 335	4 331	4 324
30	3 919	3 616	3 440	3 330	3 256	3 205	3 167	3 109	3 077	3 058	3 045	3 030	3 022	3 014	3 010	3 003
35	3 087	2 802	2 634	2 528	2 456	2 406	2 369	2 312	2 280	2 261	2 248	2 233	2 225	2 217	2 213	2 206
40	2 534	2 266	2 107	2 005	1 935	1 886	1 850	1 794	1 763	1 743	1 731	1 716	1 708	1 700	1 696	1 689
45	2 145	1 893	1 742	1 644	1 577	1 529	1 494	1 439	1 408	1 389	1 376	1 361	1 353	1 345	1 341	1 335
50	1 858	1 622	1 478	1 384	1 319	1 273	1 239	1 184	1 154	1 135	1 123	1 108	1 100	1 092	1 088	1 081
55	1 639	1 416	1 279	1 189	1 127	1 082	1 049	996	966	947	935	920	912	904	900	893
60	1 466	1 256	1 126	1 040	980	936	904	852	822	804	792	777	769	761	757	751
65	1 327	1 129	1 005	922	864	822	791	740	711	693	681	666	658	650	646	640
70	1 212	1 025	907	827	772	731	700	651	622	604	592	578	570	562	558	552
75	1 116	938	826	750	696	657	627	579	550	533	521	507	499	491	487	480
80	1 034	865	758	685	633	595	567	519	492	474	463	449	441	433	429	422
85	964	803	701	631	581	544	516	470	443	426	414	400	392	384	381	374
90	903	750	652	584	536	501	473	428	402	385	374	360	352	344	340	334
95	850	703	609	544	498	463	437	393	367	350	339	325	318	310	306	299
100	802	662	572	509	464	431	405	363	337	321	310	296	288	281	277	270
110	722	593	509	452	410	378	354	313	289	273	262	249	241	234	230	223
120	657	537	460	406	366	337	314	275	252	237	226	213	206	198	194	188
130	603	492	419	369	332	304	282	245	223	208	198	185	178	170	167	160
140	557	453	385	338	303	277	256	221	199	185	175	163	156	148	144	138
150	518	420	357	312	279	254	235	201	180	166	157	145	138	130	127	120
160	484	392	332	290	259	235	216	184	164	151	142	130	123	116	112	106
170	454	368	311	271	241	219	201	170	151	138	129	117	111	104	100	94
180	428	346	292	254	226	204	187	158	140	127	118	107	100	93	90	83
190	405	327	275	239	212	192	176	148	130	118	109	98	92	85	81	75
200	384	310	261	226	201	181	166	138	121	110	101	91	84	77	74	68
210	365	294	248	215	190	171	156	130	114	103	95	84	78	71	68	61
220	348	280	236	204	181	163	148	123	107	96	89	78	72	66	62	56
230	332	268	225	195	172	155	141	117	101	91	83	73	67	61	57	51
240	318	256	215	186	164	148	134	111	96	86	79	69	63	57	53	47
250	305	246	206	178	157	141	128	106	91	81	74	65	59	53	50	43
260	293	236	198	171	151	135	123	101	87	78	71	61	56	49	46	40
270	282	227	190	164	145	130	118	97	83	74	67	58	53	46	43	37
280	272	219	183	158	139	125	113	93	80	71	64	55	50	44	41	34
290	262	211	177	152	134	128	109	89	77	68	61	53	47	41	38	32
300	254	204	171	147	129	116	105	86	74	65	59	50	45	39	36	30

Table 16 — Basic stress $P_{bc.bas}$ for different values of critical stress C_s (see also Table 10)

All stresses are in newtons per square millimetre

C_s	$P_{bc.bas}$ for steels to BS 4360										
	Grade 43 steel with a yield stress of				Grade 50 steel with a yield stress of			Grade 55 steel with a yield stress of			
	215	230	245	280	325	340	355	400	415	430	450
20	11	11	11	11	11	11	11	11	11	11	11
30	16	16	16	16	16	16	16	17	17	17	17
40	20	21	21	21	22	22	22	22	22	22	22
50	25	25	26	26	27	27	27	27	27	27	27
60	29	30	30	31	31	32	32	32	32	32	33
70	34	34	35	35	36	36	37	37	37	38	38
80	38	38	39	40	41	41	41	42	42	42	43
90	41	42	43	44	46	46	46	47	47	47	48
100	45	46	47	48	50	50	51	52	52	52	53
110	48	50	51	52	54	55	55	56	57	57	57
120	52	53	54	56	58	59	60	61	61	62	62
130	55	56	57	60	63	63	64	65	66	66	67
140	58	59	61	64	67	67	68	70	70	71	71
150	60	62	64	67	70	71	72	74	75	75	76
160	63	65	67	70	74	75	76	78	79	80	80
170	65	67	70	74	78	79	80	82	83	84	85
180	67	70	72	77	81	82	83	86	87	88	89
190	70	72	75	79	84	86	87	90	91	92	93
200	71	74	77	82	88	89	90	94	95	96	97
210	73	76	79	85	91	92	94	98	99	100	101
220	75	78	81	87	94	95	97	101	102	104	105
230	77	80	83	90	96	98	100	105	106	107	109
240	78	82	85	92	99	101	103	108	110	111	112
250	80	83	87	94	102	104	106	111	113	114	116
260	81	85	89	96	104	107	109	115	116	118	120
270	82	86	90	98	107	109	112	118	119	121	123
280	84	88	92	101	110	113	115	122	124	126	128
290	86	91	95	104	113	116	119	126	129	131	133
300	88	93	97	106	116	120	123	130	133	135	138
310	90	94	99	109	119	123	126	134	137	139	142
320	91	96	101	111	122	126	129	138	141	143	147
330	93	98	103	113	125	129	132	141	144	147	151
340	95	100	105	115	127	131	135	145	148	151	155
350	96	101	106	117	130	134	138	148	151	154	158
360	97	103	108	119	132	136	140	151	155	158	162

Table 16 — Basic stress $P_{bc.bas}$ for different values of critical stress C_s (see also Table 10)

All stresses are in newtons per square millimetre

C_S	$P_{bc.bas}$ for steels to BS 4360										
	Grade 43 steel with a yield stress of				Grade 50 steel with a yield stress of			Grade 55 steel with a yield stress of			
	215	230	245	280	325	340	355	400	415	430	450
370	99	104	109	121	135	139	143	154	148	161	165
380	100	106	111	123	137	141	145	157	161	164	169
390	101	107	112	125	139	143	148	160	163	167	172
400	102	108	114	126	141	145	150	162	166	170	175
420	105	111	116	129	145	149	154	167	171	176	181
440	107	113	119	132	148	153	158	172	176	181	186
460	109	115	121	135	151	157	162	176	181	185	191
480	111	117	123	137	154	160	165	180	185	190	196
500	112	119	125	140	157	163	168	184	189	194	200
520	114	121	127	142	160	166	171	188	193	198	204
540	115	122	129	144	163	169	174	191	196	202	208
560	117	124	131	146	165	171	177	194	200	205	212
580	118	125	132	148	167	174	180	197	203	208	216
600	120	127	134	150	170	176	182	200	206	212	219
620	121	128	135	152	172	178	184	203	209	215	222
640	122	129	137	153	174	180	187	205	211	217	225
660	123	131	138	155	176	182	189	208	214	220	228
680	124	132	139	156	177	184	191	210	217	223	231
700	125	133	141	158	179	186	193	213	219	225	234
720	126	134	142	159	181	188	195	215	221	228	236
740	127	135	143	161	182	189	196	217	223	230	238
760	128	136	144	162	184	191	198	219	226	232	241
780	129	137	145	163	185	193	200	221	228	234	243
800	130	138	146	164	187	194	201	223	229	236	245
850	132	140	148	167	190	198	205	227	234	241	250
900	134	142	150	169	193	201	209	231	238	245	255
950	135	144	152	172	196	204	212	235	242	249	259
1 000	137	145	154	174	199	207	215	238	246	253	263
1 050	138	147	156	176	201	209	217	241	249	257	267
1 100	140	148	157	178	203	211	220	244	252	260	270
1 150	141	150	159	179	205	214	222	247	255	263	273
1 200	142	151	160	181	207	216	224	249	258	266	276
1 300	144	153	163	184	211	220	228	254	262	271	282
1 400	146	155	165	187	214	223	232	258	267	275	287
1 500	148	157	167	189	217	226	235	262	271	279	291
1 600	149	159	169	191	219	229	238	265	274	283	295
1 700	151	160	170	193	222	231	241	268	277	286	298
1 800	152	162	172	195	224	233	243	271	280	290	302
1 900	153	163	173	196	226	236	245	274	283	292	305
2 000	154	164	174	198	228	237	247	276	286	295	308

Table 17 — Basic average shear stress $P_{q.bas}$ in stiffened webs of steel to BS 4360
(see also **5.1.5.2** and Table 10)

a) *Steel of grade 43 to BS 4360*

d/*t*	$P_{q.bas}$ in newtons per square millimetre for different distances between stiffeners													
	0.2*d*	**0.3***d*	**0.4***d*	**0.5***d*	**0.6***d*	**0.7***d*	**0.8***d*	**0.9***d*	**1.0***d*	**1.1***d*	**1.2***d*	**1.3***d*	**1.4***d*	**1.5***d*
70	91	91	91	91	91	91	91	91	91	91	91	91	91	91
75	91	91	91	91	91	91	91	91	91	91	91	91	91	91
80	91	91	91	91	91	91	91	91	91	91	91	91	91	91
85	91	91	91	91	91	91	91	91	91	91	91	91	91	91
90	91	91	91	91	91	91	91	91	91	91	91	91	91	91
95	91	91	91	91	91	91	91	91	91	91	91	91	91	90
100	91	91	91	91	91	91	91	91	91	91	91	90	89	89
105	91	91	91	91	91	91	91	91	91	91	90	89	88	87
110	91	91	91	91	91	91	91	91	91	90	89	87	86	86
115	91	91	91	91	91	91	91	91	90	89	87	76	85	84
120	91	91	91	91	91	91	91	90	89	87	86	85	83	83
125	91	91	91	91	91	91	91	89	88	86	85	83	82	81
130	91	91	91	91	91	91	90	88	87	85	83	82	81	80
135	91	91	91	91	91	91	89	87	86	84	82	80	79	78
140	91	91	91	91	91	90	87	86	84	82	80	79	78	77
150	91	91	91	91	91	88	85	83	82	80	78	76	75	74
160	91	91	91	91	89	86	83	81	79	77	75	73	72	71
170	91	91	91	91	87	84	81	79	77	75	72	71	69	68
180	91	91	91	89	85	81	79	76	75	72	70	68	66	65
190	91	91	91	88	83	79	76	74						
200	91	91	91	86	81	77	74	72						
210	91	91	90	84	79	75	72							
220	91	91	89	83	78	73	70							
230	91	91	87	81	76	71								
240	91	91	86	79	74	69								
250	91	91	85	78	72	67								
260	91	91	83	76	70									
270	91	90	82	75	68									

The stepped line applies to steels of Y_s 280 N/mm² and 245 N/mm² for which the maximum value of $P_{q.bas}$ is 91 N/mm²

For steels of Y_s 230 N/mm² the maximum value of $P_{q.bas}$ is 85 N/mm²

For steels of Y_s 215 N/mm² the maximum value of $P_{q.bas}$ is 77 N/mm²

Table 17 — Basic average shear stress $P_{q.bas}$ in stiffened webs of steel to BS 4360
(see also **5.1.5.2** and Table 10)

b) *Steel of grade 50 to BS 4360*

d/*t*	$P_{q.bas}$ in newtons per square millimetre for different distances between stiffeners													
	0.2*d*	**0.3***d*	**0.4***d*	**0.5***d*	**0.6***d*	**0.7***d*	**0.8***d*	**0.9***d*	**1.0***d*	**1.1***d*	**1.2***d*	**1.3***d*	**1.4***d*	**1.5***d*
70	131	131	131	131	131	131	131	131	131	131	131	131	131	131
75	131	131	131	131	131	131	131	131	131	131	131	131	131	130
80	131	131	131	131	131	131	131	131	131	131	131	130	129	127
85	131	131	131	131	131	131	131	131	131	131	129	127	126	125
90	131	131	131	131	131	131	131	131	131	129	127	125	123	122
95	131	131	131	131	131	131	131	130	129	126	124	122	121	119
100	131	131	131	131	131	131	131	128	127	124	122	120	118	117
105	131	131	131	131	131	131	129	126	124	122	119	117	116	114
110	131	131	131	131	131	130	127	124	122	119	117	115	113	111
115	131	131	131	131	131	128	125	122	120	117	114	112	110	109
120	131	131	131	131	130	126	123	120	118	115	112	110	108	106
125	131	131	131	131	129	124	121	118	116	112	110	107	105	103
130	131	131	131	131	127	122	119	116	114	110	107	105	102	101
135	131	131	131	131	125	121	117	114	111	108	105	102	100	98
140	131	131	131	130	124	119	115	112	109	105	102	100	97	95
150	131	131	131	127	120	115	111	107	105	101	97	94	92	90
160	131	131	131	124	117	111	107	103	100	96	93	89	87	85
170	131	131	129	121	114	108	103	99	96	92	88	84	82	79
180	131	131	127	118	110	104	99	95	92	87	83	79	76	74
190	131	131	124	115	107	100	95	91						
200	131	131	122	112	104	97	91	86						
210	131	131	119	109	100	93	87							
220	131	129	117	106	97	89	83							
230	131	127	115	103	94	86								
240	131	125	112	100	90	82								
250	131	123	110	98	87	78								
260	131	121	107	95	84									
270	121	120	105	92	80									

The stepped line applies to steels of Y_s 355 N/mm² for which the maximum value of $P_{q.bas}$ is 131 N/mm²

For steels of Y_s 340 N/mm² the maximum value of $P_{q.bas}$ is 126 N/mm²

For steels of Y_s 325 N/mm² the maximum value of $P_{q.bas}$ is 120 N/mm²

Table 17 — Basic average shear stress $P_{q.bas}$ in stiffened webs of steel to BS 4360
(see also **5.1.5.2** and Table 10)

c) *Steel of grade 55 to BS 4360*

d/t	$P_{q.bas}$ in newtons per square millimetre for different distances between stiffeners													
	0.2*d*	**0.3***d*	**0.4***d*	**0.5***d*	**0.6***d*	**0.7***d*	**0.8***d*	**0.9***d*	**1.0***d*	**1.1***d*	**1.2***d*	**1.3***d*	**1.4***d*	**1.5***d*
70	167	167	167	167	167	167	167	167	167	167	167	167	167	167
75	167	167	167	167	167	167	167	167	167	167	167	167	167	166
80	167	167	167	167	167	167	167	167	167	167	167	166	164	162
85	167	167	167	167	167	167	167	167	167	167	164	162	161	159
90	167	167	167	167	167	167	167	167	16	164	161	159	157	156
95	167	167	167	167	167	167	167	166	164	161	158	156	154	152
100	167	167	167	167	167	167	166	164	161	158	155	153	151	149
105	167	167	167	167	167	167	164	161	159	155	152	149	147	145
110	167	167	167	167	167	165	161	158	156	152	149	146	144	142
115	167	167	167	167	167	163	159	156	153	149	146	143	141	139
120	167	167	167	167	166	161	156	153	150	146	143	140	137	135
125	167	167	167	167	164	158	154	150	148	143	140	137	134	132
130	167	167	167	167	162	156	151	148	145	140	137	133	131	128
135	167	167	167	167	160	154	149	145	142	137	133	130	127	125
140	167	167	167	165	158	151	146	142	139	134	130	127	124	121
150	167	167	167	161	153	147	141	137	134	128	124	120	117	115
160	167	167	167	158	149	142	136	132	128	123	118	114	111	108
170	167	167	165	154	145	137	131	126	122	117	112	108	104	101
180	167	167	161	150	141	133	126	121	117	111	106	101	97	94
190	167	167	158	147	136	128	121	115						
200	167	167	155	143	132	123	116	110						
210	167	167	152	139	128	119	111							
220	167	164	149	135	124	114	106							
230	167	162	146	132	119	109								
240	167	160	143	128	115	104								
250	167	157	140	124	111	100								
260	167	155	137	121	107									
270	167	152	134	117	102									

The stepped line applies to steels of Y_s 450 N/mm² for which the maximum value of $P_{q.bas}$ is 167 N/mm²

For steels of Y_s 430 N/mm² the maximum value of $P_{q.bas}$ is 159 N/mm²

For steels of Y_s 415 N/mm² the maximum value of $P_{q.bas}$ is 154 N/mm²

For steels of Y_s 400 N/mm² the maximum value of $P_{q.bas}$ is 148 N/mm²

5.1.7 Members subjected to a combination of stresses

5.1.7.1 *Proportioning of members*

5.1.7.1.1 Members subjected to a combination of coexistent bending and axial loads shall be designed in accordance with **5.1.7.1.2** and **5.1.7.1.3** those subjected to a combination of shear and other stresses shall be designed in accordance with **5.1.7.1.4** and **5.1.7.1.5**.

5.1.7.1.2 Members subjected to bending and axial compression, shall be so proportioned that

$$\frac{f_{ac}}{P_{ac}} + \frac{f_{bc}}{P_{bc}} \leqslant 1$$

where

f_{ac} is the calculated axial compressive stress;

P_{ac} is the permissible compressive stress in axially loaded compression members (see **4.3** and **5.1.3**);

f_{bc} is the calculated maximum compressive stress due to bending about both principal axes;

P_{bc} is the permissible compressive stress in bending, using the lesser value when bending occurs about both axes (see **4.3** and **5.1.4**).

5.1.7.1.3 Members subjected to bending and axial tension shall be so proportioned that

$$\frac{f_{at}}{P_{at}} + \frac{f_{bt}}{P_{bt}} \leqslant 1$$

where

f_{at} is the calculated axial tensile stress;

P_{at} is the permissible tensile stress in axially loaded tension members (see **4.3** and **5.1.2**);

f_{bt} is the calculated maximum tensile stress due to bending about both principal axes;

P_{bt} is the permissible tensile stress in bending (see **4.3** and **5.1.4**).

5.1.7.1.4 Members subjected to shear and bending shall be so proportioned that the equivalent stress f_e (in N/mm^2) calculated from

$$f_e = \sqrt{(f_{bt}^2 + 3f_q^2)} \text{ or from}$$
$$f_e = \sqrt{(f_{bc}^2 + 3f_q^2)}$$

is not greater than P_e

where

f_q is the calculated shear stress;

f_{bt} and f_{bc} are as defined in **5.1.7.1.3** and **5.1.7.1.2** respectively;

P_e is the permissible equivalent stress in N/mm^2 (see **4.3** and **5.1.7.2**).

5.1.7.1.5 Members subjected to shear, bearing and bending, shall be so proportioned that the equivalent stress f_e (in N/mm^2) calculated from

$$f_e = \sqrt{(f_{bt}^2 + f_b^2 + f_{bt}f_b + 3f_q^2)}$$

or from

$$f_e = \sqrt{(f_{bc}^2 + f_b^2 - f_{bc}f_b + 3f_q^2)}$$

is not greater than P_e

where

f_b is the calculated bearing stress;

f_q, f_{bc}, f_{bt} and P_e are as defined in **5.1.7.1.4**.

5.1.7.2 *Basic equivalent stress.* The basic equivalent stress $P_{e.bas}$ (in N/mm^2) due to a combination of shear and other stresses shall not exceed the value

$P_{e.bas} = 0.93\ Y_s$

where Y_s is as defined in **5.1.2**.

Tabulated values of $P_{e.bas}$ for the range of steels covered by BS 4360 are given in Table 10.

5.1.8 Members with flanges subjected to transverse bending stress. The design of members subjected to this type of loading shall take into account both the longitudinal and transverse bending stresses.

NOTE A suitable method is that given for the design of overhead runway beams in BS 2853.

5.2 Lattice girders and trusses: verification relative to the yield stress

5.2.1 Design procedure. For lattice members, design verification relative to the yield stress shall be carried out in accordance with **5.2.2** to **5.2.4**:

a) for the lattice as a whole;

b) for the individual members comprising the lattice.

NOTE *Secondary stresses in lattice girders and trusses.* Relaxations in some requirements of **5.2.2** to **5.2.4** are allowed in cases where secondary stresses are calculated and taken into account in the design (see **5.3**).

5.2.2 The lattice as a whole

5.2.2.1 *Subjected to axial tension.* The lattice shall be designed as an axially loaded tie. The basic stress shall not exceed the value of $P_{at.bas}$ given in **5.1.2**.

5.2.2.2 *Subjected to axial compression.* The lattice shall be designed as an axially loaded strut using the maximum effective slenderness ratio s as defined in **5.1.3** for the lattice as a whole. The basic stress shall not exceed the value of $P_{ac.bas}$ given in **5.1.3**.

5.2.2.3 ***Subjected to bending***

5.2.2.3.1 *Lattice box girders.* For lattice box girders having an l/r_y not exceeding 140 and a depth-to-breadth ratio not exceeding 6, the basic stress $P_{at.bas}$ and $P_{ac.bas}$ shall not exceed the value of $P_{at.bas}$ as given in **5.1.2** (l and r_y as defined in **5.1.3**). Lattice box girders having a depth-to-breadth ratio exceeding 6 shall be designed as lattice trusses. The girder shall be stiffened to prevent distortion of the cross-sectional shape when the girder deflects.

5.2.2.3.2 *Lattice trusses.* For lattice trusses, and lattice box girders having a depth-to-breadth ratio exceeding 6, the main compression members shall be designed as axially loaded struts using the basic compressive stresses $P_{ac.bas}$ given in **5.1.3** and the effective lengths specified in **7.1**. The main tension members shall be designed as axially loaded ties. The basic stresses shall not exceed the value of $P_{at.bas}$ given in **5.1.2**.

5.2.2.4 *Subjected to axial tension and bending, and axial compression and bending.* The lattice shall be so proportioned that in the tension chord members

$$\frac{f_{at}}{P_{at}} + \frac{f_{bt}}{P_{bt}} \leqslant 1$$

where f_{at}, P_{at}, f_{bt}, P_{bt} are as defined in **5.1.7.1.2**.

In the compression chord members

$$\frac{f_{ac}}{P_{ac}} + 1.1\frac{f_{bc}}{P_{bc}} \leqslant 1$$

where

f_{ac} is the calculated axial compressive stress;

P_{ac} is the permissible axial compressive stress corresponding to the maximum effective slenderness ratio of the lattice as a whole;

f_{bc} is the calculated maximum compressive stress due to bending about the principal axes of the lattice as a whole;

P_{bc} is the permissible compressive stress in bending based upon the value of the basic stress given in Table 10 for parts in bending (tension or compression).

5.2.3 Individual members of a lattice

5.2.3.1 The basic stresses in the individual members of a lattice shall not exceed those given in **5.1**.

5.2.3.2 In the case of an individual member subjected to axial compression due to loadings applied to the lattice as a whole at panel points, the total compressive stress in the member shall not exceed the permissible stress corresponding to the effective slenderness of the member between panel point as given in **7.1**.

5.2.3.3 In the case of an individual member subjected to a combination of bending stresses due to loads applied to the member between panel points and axial stresses due to loadings on the lattice as a whole at panel points, the combined stress formulae given in **5.1.7.1.2** and **5.1.7.1.3** shall be used.

5.2.4 Lattice and battened struts. The requirements of **5.2** shall also apply, where appropriate, to the design of lattice and battened struts.

5.3 Secondary stresses

Secondary stresses shall be added to the coexistent (primary) stresses in the individual members and shall be checked in accordance with the following.

NOTE For the purposes of this standard, stresses in the individual members of lattice or braced structures that are the result of eccentricity of connections, elastic deformation of the structure, and rigidity of joints are defined as secondary stresses. Where secondary stresses are computed and added to the coexistent (primary) stresses calculated in accordance with **5.1** and **5.2**, higher stress levels are permitted.

a) *Members subjected to axial compression and bending*

$$\frac{f_{ac}}{P_{ac}} + \frac{f_{bc}}{P_{bc}} \leqslant 1.20$$

subject to the limitation that

$$\frac{f_{ac}}{P_{ac}} \leqslant 1.0$$

where f_{ac}, P_{ac}, f_{bc} and P_{bc} are as defined in **5.1.7.1.2**.

b) *Members subjected to axial tension and bending*

$$\frac{f_{at}}{P_{at}} + \frac{f_{bt}}{P_{bt}} \leqslant 1.20$$

subject to the limitation that

$$\frac{f_{at}}{P_{at}} \leqslant 1.0$$

where f_{at}, P_{at}, f_{bt} and P_{bt} are as defined in **5.1.7.1.3**.

Section 6. Basic stresses in connections

6.1 Welds

6.1.1 General. All welding on loadbearing structures shall be carried out in accordance with BS 5135.

6.1.2 Butt welds: general

6.1.2.1 All butt welds shall be made using a type of electrode (or other welding consumable) that will produce all-weld tensile test specimens as specified in BS 709 having both a yield strength and a tensile strength not less than that of the parent metal.

Where electrodes complying with BS 639 are used to weld steel complying with BS 4360 the matching electrodes for butt welds are:

Steel grade in BS 4360	Electrodes complying with BS 639 Classification
43	E43--R
50	E51--B
WR 50	E51--B[a]
55	E51--B

[a] Special electrodes may be necessary to suit weather-resisting steel.

Electrodes for use with grade 55 steel shall have a minimum all-weld yield stress of 450 N/mm^2 and a minimum tensile strength of 550 N/mm^2.

6.1.2.2 The basic strength of a butt weld shall be taken as equal to that of the parent metal, provided that the weld complies with the requirements of **6.1.2.1**.

6.1.2.3 Intermittent complete-penetration butt welds shall be used only to resist shear. The effective length of an intermittent weld shall be taken as its overall length minus $2t'$, where t' is the thickness of the thinner part joined. The minimum effective length of any such weld shall be not less than $4t'$ or less than 40 mm, and the longitudinal space between the effective lengths of weld shall be not more than $12t'$.

NOTE Where fatigue is a design criterion, intermittent butt welds are not to be used.

6.1.3 Butt welds: partial penetration

6.1.3.1 A continuous partial-penetration butt weld welded from one side only or from both sides can be used provided that it is not subjected to a bending moment about the longitudinal axis of the weld other than that resulting from the eccentricity of the weld metal relative to the parts joined or from secondary moments.

A partial-penetration butt weld welded from one side only shall not be subjected to any loading that would cause the root of the weld to be in tension if failure due to such tension would be liable to be progressive and to lead to structural collapse unless it can be demonstrated that proper attention has been paid to the detailed design of the joint and testing and operational experience has shown this detail to be satisfactory.

6.1.3.2 The throat thickness of a partial-penetration butt weld welded from one side only shall be taken as the depth of penetration and the adverse effect of the eccentricity of the weld metal relative to the parts joined shall also be allowed for when calculating the strength.

The specified penetration of such a weld shall be not less than $2\sqrt{t'}$ where t' is the thickness (in mm) of the thinner part joined.

6.1.3.3 The throat thickness of a partial-penetration butt weld welded from both sides shall not be taken as more than the total depth of penetration relative to the surfaces of the thinner part joined. Except where it can be shown that greater penetration can consistently be achieved, the depth of penetration from each side shall not be taken as more than the depth of grooved weld preparation on that side in the case of a J or U weld, or more than the depth of groove less 3 mm in the case of a V or bevel weld.

Where the weld metal is placed asymmetrically relative to the axis of the parts joined, the adverse effect of the eccentricity shall also be allowed for when calculating the strength of the weld.

The specified penetration from each side of such a weld shall be not less than $2\sqrt{t'}$ where t' is the thickness (in mm) of the thinner part joined.

6.1.3.4 The basic strength of a compound weld comprising a partial-penetration butt weld reinforced by a fillet weld shall be calculated as for a deep-penetration fillet weld (see **6.1.4.3**).

6.1.4 Fillet welds

6.1.4.1 The effective throat thickness a_w of a fillet weld (other than a deep-penetration fillet weld covered by **6.1.4.3**) shall be taken as the maximum perpendicular distance from the root of the weld to a straight line joining the fusion faces that lies within the cross section of the weld (as shown in Figure 4). However a_w shall not be taken as more than $0.7S_w$, where S_w is the effective leg length of the weld as defined by the figure (or the average if the legs are unequal).

6.1.4.2 Fillet welds shall not be considered capable of transmitting primary loadings between connecting parts the fusion faces of which form an angle of more than 120° or less than 60°, except in the case of hollow sections continuously welded around the periphery, where the normal limitations are 150° and 30°, which can be exceeded subject to proof of efficiency (see Appendix D of BS 5135:1974).

6.1.4.3 Deep-penetration fillet welds shall be used only where it can be shown that the required penetration can consistently be achieved, as for example by automatic welding processes. The depth of penetration d_w shall be measured as shown in Figure 5 and shall be at least 2 mm. The effective leg length S_w and the effective throat thickness a_w shall be taken as shown in the figure.

6.1.4.4 The maximum stress in a fillet weld shall be taken as the vector sum of the stresses due to all forces and moments transmitted by the weld, each based on a thickness equal to the effective throat thickness a_w.

The basic stress $P_{w.bas}$ in a fillet weld, based on a thickness equal to the effective throat thickness a_w, shall not exceed 0.3 U_{se}, where U_{se} is the tensile strength of the electrode or other welding consumable based on all-weld tensile tests as specified in BS 709. However $P_{w.bas}$ shall not be taken as more than 0.3 U_s, where U_s is the minimum ultimate tensile strength of the parent metal.

Where electrodes complying with the requirements of BS 639 are used to weld steel complying with the requirements of BS 4360 the basic weld stresses $P_{w.bas}$ given in Table 18 shall apply.

6.1.4.5 The effective length of a discontinuous run of fillet weld shall be taken as the overall length less $2S_w$. The effective length of a fillet weld required to transmit primary loading shall be not less than 40 mm or less than $4S_w$.

Table 18 — Basic stresses in welds

Steel grade in BS 4360	Electrodes to BS 639		
	Classification E43--R U_{se} = 430 N/mm^2	Classification E51--B U_{se} = 510 N/mm^2	Classification E51--B $U_{se} \geqslant$ 550 N/mm^2
	N/mm^2	N/mm^2	N/mm^2
43	118	126	126
50	118	144	144
WR 50	118	141[a]	141[a]
55	118	147	162[b]

[a] 147 N/mm^2 for structural hollow sections of grade WR 50.
[b] This applies only when electrodes with a minimum yield stress of 450 N/mm^2 and a minimum tensile strength of 550 N/mm^2 are used.

6.1.4.6 The space along any one edge of an element between consecutive effective lengths of intermittent fillet welds (other than those interconnecting the components of back-to-back tension or compression members) shall not exceed 300 mm nor shall it exceed $16t'$ for elements in compression or $24t'$ for elements in tension, where t' is the thickness of the thinner part joined.

An intermittent fillet weld connecting components subject to primary loadings shall extend to the end of the part connected.

6.1.4.7 Where the end of an element is connected only by intermittent fillet welds the transverse spacing of the welds shall not exceed 200 mm and the length of each weld shall be not less than the transverse spacing.

6.1.4.8 A single fillet weld shall not be subjected to a bending moment about its longitudinal axis that is produced by primary loading.

A single fillet weld shall not be subjected to any loading that would cause the root of the weld to be in tension if failure due to such tension would be liable to be progressive and to lead to structural collapse unless it can be demonstrated that proper attention has been paid to the detailed design of the joint and testing and operational experience has shown this detail to be satisfactory.

6.2 Basic stresses for bolts, studs and rivets

6.2.1 Bolts and studs

6.2.1.1 *Friction grip bolts.* These bolts shall comply with the requirements of BS 4395-1, BS 4395-2 and BS 4395-3 and shall be fitted in accordance with BS 4604-1, BS 4604-2 and BS 4604-3.

In the design of joints using friction grip bolts, the duty factor (see **4.3.1**) shall be taken as 1.0 irrespective of the crane classification.

6.2.1.2 ***Precision bolts***

6.2.1.2.1 *General.* Precision bolts shall be turned or cold finished and fitted into reamed or drilled holes whose diameter shall not exceed the diameter of the bolts by more than 0.4 mm.

6.2.1.2.2 *Bolts in tension*

a) *Bolts not tightened by controlled means.* The basic permissible tensile stress $P_{at.bas}$ at the root of the thread for these bolts shall not exceed

$P_{at.bas} = 0.4\ Y_{R0.2}$

where $Y_{R0.2}$ is the yield stress or 0.2 % proof stress of the material.

Where there is a fluctuating load or a reversal of load across the joint, the number of bolts or studs required shall be determined in accordance with **8.7** except in the case of bolts or studs having a yield stress in excess of 250 N/mm^2. In such case, the difference between the stresses corresponding to f_{max} and f_{min} shall be not greater than 10 % of the ultimate tensile strength of the material and the mean stress shall be not greater than 15 % of the ultimate strength of the material.

b) *Bolts tightened by controlled means.* These bolts shall be tightened by controlled means so that the pretensioned stress P_{at} at the root of the thread is not greater than 0.8 $Y_{R0.2}$ or less than 0.7 $Y_{R0.2}$.

The virtual permissible stress $P_{at.virt}$ at the root of the thread induced in these bolts by external loading shall not exceed:

$P_{at.virt} = 0.48\ Y_{R0.2}$ for non-fluctuating loads;

$P_{at.virt} = 0.40\ Y_{R0.2}$ for fluctuating loads.

6.2.1.2.3 *Bolts in shear.* The basic shear stress $P_{q.bas}$ for the section of the bolt at the interface of the joint shall not exceed

$P_{q.bas} = 0.375\ Y_{R0.2}$

Where there is a fluctuating load or a reversal of load across the joint, the number of bolts or studs required shall be determined in accordance with section 8.

6.2.1.2.4 *Bolts subjected to combined tension and shear.* A check shall be made that

$$f_{at} \leqslant p_{at}$$
$$f_q \leqslant p_q$$
$$\sqrt{(f_{at}^2 + 3f_q^2)} \leqslant 1.2\, p_{at}$$

6.2.1.2.5 *Bolts in bearing.* The basic permissible pressure $p_{b.bas}$ in the hole shall not exceed the value

$p_{b.bas} = 0.9\ Y_{R0.2}$

where $Y_{R0.2}$ is the yield stress or 0.2 % proof stress for the bolt or for the joint material, whichever gives the lowest value.

6.2.1.3 *Black bolts other than friction grip bolts.* Black bolts shall not be used in main members, in shear for joints in stress-bearing members, or in joints subjected to fatigue.

For other applications of use the basis permissible stresses shall not exceed:

$P_{at.bas} = 0.4\ Y_{R0.2}$ tension

$P_{q.bas} = 0.33\ Y_{R0.2}$ shear

where $Y_{R0.2}$ is as defined in **6.2.1.2.2** a);

$P_{m.bas} = 0.66\ Y_{R0.2}$ bearing

where $Y_{R0.2}$ is as defined in **6.2.1.2.5**.

6.2.2 Rivets. The basic stresses for rivets shall be as given in Table 19. Where there is a fluctuating load or a reversal of load across the joint, the number of rivets shall be determined in accordance with section 8.

Table 19 — Basic stresses in rivets as a percentage of $Y_{R0.2}$

Type	In tension	In shear	In bearing
	%	%	%
Power-driven shop rivets	40	43.5	90[a]
Power-driven field rivets	40	40	85[a]
Hand-driven rivets	40	36.5	80[a]

[a] The $Y_{R0.2}$ of the rivet or the joint material, whichever is the lower, should be used to determine the basic bearing stress.

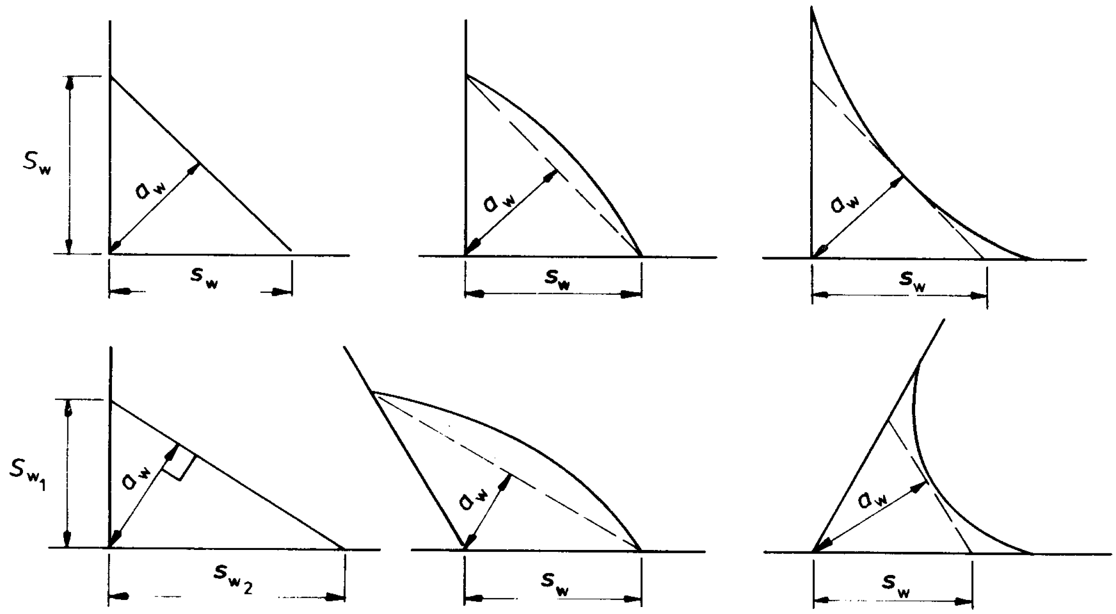


Figure 4 — Effective throat thickness of fillet welds

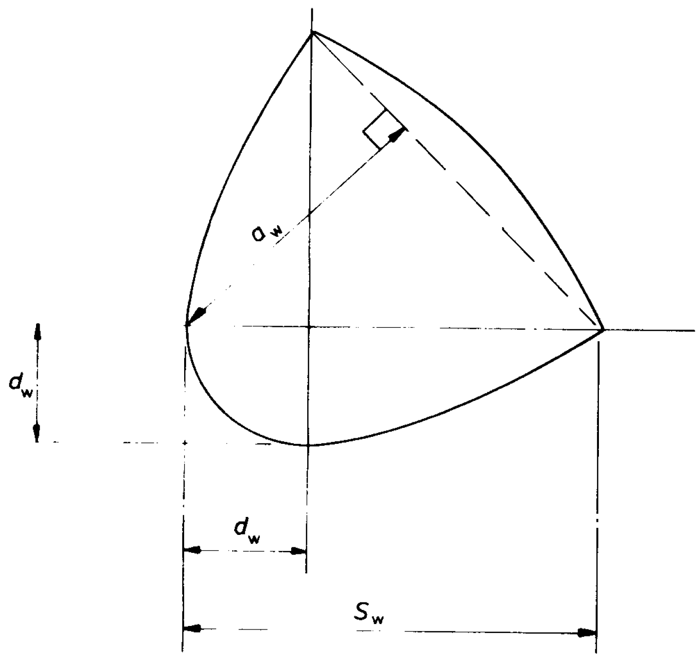


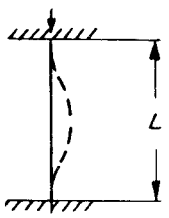
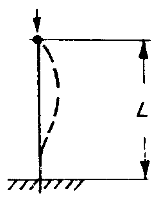
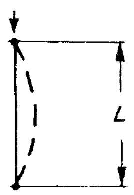
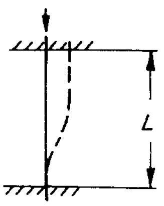
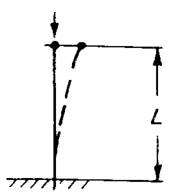
Figure 5 — Effective throat thickness of deep-penetration fillet welds

Section 7. Proportions of structural components, plates and web stiffeners

7.1 Effective lengths of parts in compression

7.1.1 Struts. For the purpose of calculating slenderness ratio l/r for struts, the effective length (l) given in Table 20 shall be taken, where L is the actual length of the member as shown in the appropriate diagram of Table 20.

Table 20 — Effective lengths of parts in compression

Diagrammatic representation	Restraint conditions	Effective length l
	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	$1.0L$
	Effectively held in position and restrained in direction at one end and partially restrained in direction but not held in position at the other	$1.5L$
	Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end	$2.0L$

7.1.2 Jibs. A method for determining the overall effective lengths of rope supported (strut type) and cantilever jibs is given in Appendix C.

The effective lengths of the individual members of a lattice jib are given in **7.1.4**.

7.1.3 Single web plate girders and rolled beams

7.1.3.1 The effective length (l) of the compression flange for buckling normal to the plane of the girder to be used as described in **10.4.2.2** shall be as given in **7.1.3.2** and **7.1.3.3**.

7.1.3.2 Where there are no lateral bracings between compression flanges and no cross frames the effective length shall be as shown in Table 21.

Restraint against torsion shall be provided by web or flange cleats, bearing stiffeners, lateral end frames or other supports to the end of the compression flanges.

7.1.3.3 Where there is effective lateral bracing direct to compression flanges the effective length shall be as shown in Figure 6.

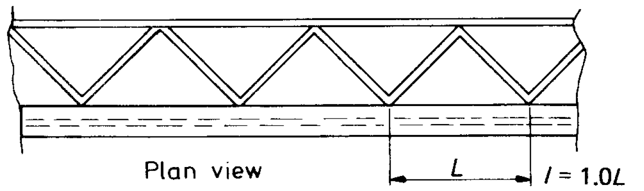


Figure 6 — Effective length with lateral bracing

7.1.4 Lattice girders and trusses

NOTE For the overall effective lengths of lattice jibs see Appendix C.

7.1.4.1 *Effective length of a lattice girder or truss as a whole.* Where there is no adequate lateral bracing, the effective length shall be taken as the span when considering buckling normal to the plane of the member.

7.1.4.2 ***Effective lengths of the individual members of a lattice girder or truss***

7.1.4.2.1 *Main boom members.* For main boom members the effective length shall be 0.85 times the distance between centres of intersection of bracing members in the plane in which buckling is being considered.

Where there is no adequate lateral bracing, the effective length when considering buckling normal to the plane of the truss or girder shall be taken as the span, as in **7.1.4.1**.

7.1.4.2.2 *Bracing (or web) members axially loaded.* For bracing or web members axially loaded the effective length shall be:

0.70 times the distance between centres of intersection with the main booms for buckling in the plane of the girder;

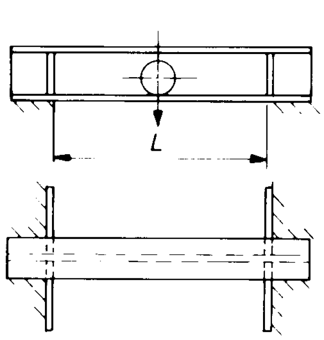
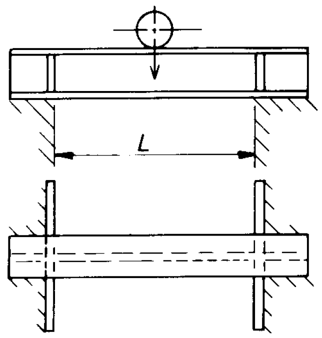
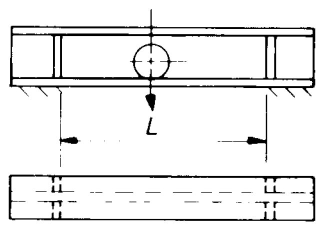
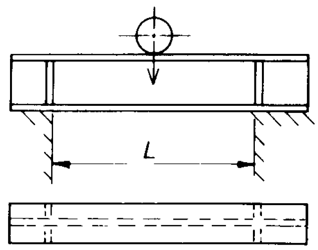
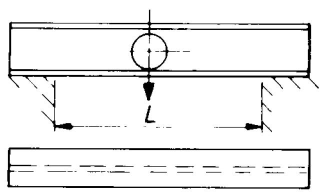
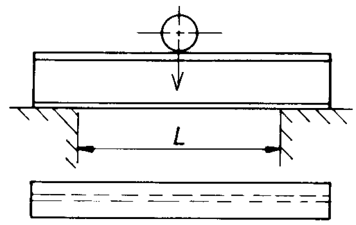
0.85 times the distance between centres of intersection with the main booms for buckling normal to the plane of the girder.

In the case of cross-braced systems the effective length of a member shall be taken as 0.85 times the greatest distance between any two intersections when considering buckling in the plane of the girder.

7.1.4.2.3 *Single-angle discontinuous struts connected to gussets or to a section.* For single-angle discontinuous struts connected to gussets or to a section, either by riveting or by bolting with not less than two bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut can be ignored and the strut designed as an axially-loaded member provided that the calculated average stress does not exceed the allowable stresses derived from the basic stresses given in **5.1.3** in which l is the length of the strut, between the centres of the fastenings at each end, and r is the minimum radius of gyration.

7.1.4.2.4 *Single-angle discontinuous struts intersected by, and effectively connected to, another angle in cross bracing.* For single-angle discontinuous struts intersected by, and effectively connected to, another angle in cross bracing, the effective length in the plane of the bracing shall be taken as in **7.1.4.2.2**. In the plane normal to the plane of the bracing, the effective length l shall be taken as the distance between the points of intersection and the centroids of the main members. In calculating the slenderness ratio, the radius of gyration about the appropriate rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration for buckling in the plane of the bracing.

Table 21 — Effective length with no lateral bracing

Load applied to the tension flange	Restraint conditions against lateral bending and torsion of section	Load applied to the compression flange and both load and flange free to move laterally
 $l = 0.7L$	Ends completely restrained	 $l = 0.85L$
 $l = 0.85L$	Ends partially restrained	 $l = 1.0L$
 $l = 1.0L$	Ends unrestrained	 $l = 1.2L$

7.1.5 Cantilever beams without intermediate lateral support

7.1.5.1 The effective length (l) of cantilever beams of projecting length L to be used in **5.1.4.2** shall be as follows:

a) *Built-in at the support*

1) free at the end $l = 0.85L$
2) restrained against torsion at the free end by contiguous construction $l = 0.75L$
3) restrained against lateral deflection and torsion at the free end $l = 0.50L$

b) *Continuous at the support*

1) unrestrained against torsion at the support and free at the end $l = 3L$
2) with partial restraint against torsion at the support and free at the end $l = 2L$
3) restrained against torsion at the support and free at the end $l = L$

7.1.5.2 For cases 1), 2) and 3) of **7.1.5.1** b) in which the cantilever end is not free but is subject to a degree of restraint the effective length shall be multiplied by a factor as follows.

a) Where the end is restrained against torsion by contiguous construction the effective lengths given in **7.1.5.1** b) shall be multiplied by a factor of 0.88.

b) Where the end is restrained against lateral deflection and torsion the effective lengths given in **7.1.5.1** b) shall be multiplied by a factor of 0.59.

7.1.5.3 For cantilever beams loaded on the compression flange, the effective lengths given in **7.1.5.1** a) and b) and **7.1.5.2** shall be increased by a factor of 1.2.

7.2 Widths of plates

7.2.1 Effective widths of plates in compression. For the computation of the effective cross-sectional area of a compression member subjected to the design checks set out in **5.1.3** the effective width of an unstiffened plate, in terms of its width b measured between adjacent lines of rivets, bolts or welds connecting it to other parts of the section, shall be taken as:

a) *For riveted, bolted or stress relieved welded members*

1) *For steel of grade 43 to BS 4360*

for $b/t' \leqslant 45$, the effective width = b

for $b/t' > 45$, the effective width = $45t'$

2) *For steel of grade 50 to BS 4360*

for $b/t' \leqslant 40$, the effective width = b

for $b/t' > 40$, the effective width = $40t'$

3) *For steel of grade 55 to BS 4360*

for $b/t' \leqslant 35$, the effective width = b

for $b/t' > 35$, the effective width = $35t'$

b) *For as-welded members*

1) *For steel of grade 43 to BS 4360*

for $b/t' \leqslant 30$, the effective width = b

for $b/t' > 30$, the effective width $= 40t' (b/t' - 18)/(b/t' - 14)$

2) *For steel of grade 50 to BS 4360*

for $b/t' \leqslant 27$, the effective width = b

for $b/t' > 27$, the effective width $= 34t' (b/t' - 15)/(b/t' - 12)$

3) *For steel of grade 55 to BS 4360*

for $b/t' \leqslant 23$, the effective width = b

for $b/t' > 23$, the effective width $= 30t' (b/t' - 13)/(b/t' - 10)$

In the above, t' is the thickness of the thinnest plate, or the aggregate thickness of two or more plates provided these plates are adequately connected together.

7.2.2 Maximum width of plates in compression

7.2.2.1 The width of a plate, measured between adjacent lines of rivets, bolts or welds connecting it to other parts of the section, unless effectively stiffened, shall not exceed the values given in Table 22, t' being the plate thickness as defined in **7.2.1**.

Table 22 — Maximum width of plates in compression

Grade of steel to BS 4360	Riveted, bolted or stress-relieved welded members	As-welded members
43	$90t'$	$80t'$
50	$80t'$	$70t'$
55	$70t'$	$60t'$

7.2.2.2 Compression flange plates unstiffened at their edges shall not project beyond the outer line of connections to the flange angles (or where there are no flange angles to the tongue plates) by more than the values given in Table 23, t' being the plate thickness as defined in **7.2.1**.

Table 23 — Projection of unstiffened compression flange plates

Grade of steel to BS 4360	Riveted, bolted or stress-relieved welded members	As-welded members
43	$16t'$	$12t'$
50	$14t'$	$12t'$
55	$12.5t'$	$12t'$

7.2.3 Maximum widths of plates in tension. In all cases, tension flange plates, stiffened or unstiffened at their edges, shall not project beyond the line of connections to the web or tongue plates by more than $12t'$, where t' is as defined in **7.2.1**.

7.3 Web plates and web stiffeners

7.3.1 Minimum thickness of web plates for open sections

7.3.1.1 *For unstiffened webs.* The thickness t of the web plate shall be not less than:

$d_1/85$ for steel of grade 43 to BS 4360;

$d_1/75$ for steel of grade 50 to BS 4360;

$d_1/65$ for steel of grade 55 to BS 4360;

where d_1 is the clear distance between flange angles or, where there are no flange angles, between flanges (ignoring fillets); where tongue plates having a thickness not less than twice the thickness of the web plate are used, d_1 is the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thickness of the tongue plates, whichever is the less.

7.3.1.2 *For vertically stiffened webs.* The thickness t of the web plate shall be not less than:

1/180 of the small clear panel dimension;

1/270 of the greater clear panel dimension and $d_2/200$ for steel of grade 43 to BS 4360 or $d_2/180$ for steel of grade 50 to BS 4360 or $d_2/155$ for steel of grade 55 to BS 4360;

where d_2 is twice the clear distance from the compression flange angles or plate or tongue plate to the neutral axis.

7.3.1.3 *For webs stiffened both vertically and horizontally and with the horizontal stiffener at a distance from the compression flange of 2/5 of the distance from the compression flange to the neutral axis.* The thickness t of the web plate shall be no less than:

1/180 of the smaller clear dimension in each panel;

1/270 of the greater clear panel dimension and $d_2/250$ for steel of grade 43 to BS 4360 or $d_2/225$ for steel of grade 50 to BS 4360 or $d_2/190$ for steel of grade 55 to BS 4360.

When there is also a horizontal stiffener at the neutral axis of the girder, the thickness t of the web plate shall be not less than:

1/180 of the smaller clear dimension in each panel;

1/270 of the greater clean panel dimension and $d_2/400$ for steel of grade 43 to BS 4360 or $d_2/360$ for steel of grade 50 to BS 4360 or $d_2/310$ for steel of grade 55 to BS 4360:

where d_2 is as defined in **7.3.1.2**.

7.3.2 Web stiffeners for open sections

7.3.2.1 *Loadbearing web stiffeners*

7.3.2.1.1 *Rolled I beams and channels.* For rolled I beams and channels, loadbearing stiffeners shall be provided at points of concentrated load (including points of support) where the concentrated load or reaction exceeds the value

$P_{ac} \times t \times L_b$

where

P_{ac} is the permissible axial stress for struts as given in **5.1.3** corresponding to a slenderness ration of 1.7 d_3/t and $\alpha = 5.5$;

t is the web thickness;

d_3 is the clear depth of web between root fillets;

L_b is the length of the stiff portion of the bearing plus the additional length given by dispersion at 45° to the level of the neutral axis, and measured along the neutral axis.

The stiff portion of a bearing is the length that cannot deform appreciably in bending, and shall not be taken as greater than half the depth of the beam for simply supported beams and the full depth of the beam for continuous beams.

7.3.2.1.2 *Plate girders.* For plate girders, loadbearing stiffeners shall be provided at points of support and at points of concentrated load where the web would otherwise be overstressed (see **7.3.2.1.1**).

7.3.2.1.3 *Details of stiffeners.* Loadbearing stiffeners shall be symmetrical about the web, where possible.

Loadbearing stiffeners, in which the concentrated load causes compression shall be designed as struts, assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal to 20 times the web thickness. The radius of gyration shall be taken about the axis parallel to the web of the beam or girder, and the calculated stress shall not exceed the allowable stress for a strut, assuming an effective length equal to 0.7 times the length of the stiffener.

The outstanding legs of each pair of loadbearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds does not exceed the bearing stress specified in **5.1.6**.

Loadbearing stiffeners shall be provided with sufficient rivets, bolts or welds to transmit to the web the whole of the load in the stiffeners.

Loadbearing stiffeners shall be fitted to provide a tight and uniform bearing upon the flange transmitting the load or reaction unless welds are provided between the flange and stiffener for this purpose. At points of support this requirement shall apply at both flanges. Where the ends of stiffeners are not fitted or connected to the flange they shall be kept well clear of the flange.

Loadbearing stiffeners shall not be joggled and shall be solidly packed throughout.

When loadbearing stiffeners at supports are the sole means of providing restraint against torsion the stiffener shall be so proportioned that

$$I \geqslant \frac{D^3 T_{max}}{250} \frac{R}{W}$$

where

I is the moment of inertia of the pair of stiffeners about the centre line of the web-plate;

D is the overall depth of girder;

T_{max} is the maximum thickness of compression flange;

R is the reaction on the bearing;

W is the total load on girder.

7.3.2.2 *Intermediate stiffeners*

7.3.2.2.1 *Vertical stiffeners.* To limit web buckling, vertical intermediate stiffeners shall be provided throughout the length of the girder at a distance apart not greater than $1.5d_1$ when the thickness of the web is less than $d_1/85$ for steel of grade 43 to BS 4360 or $d_1/75$ for steel of grade 50 to BS 4360 or $d_1/65$ for steel of grade 55 to BS 4360, where d_1 is the depth of web as defined in **7.3.1.1**.

These stiffeners shall be so designed that

$$I \geqslant 1.5\,\frac{{d_1}^3 \times t^3}{{S_t}^2}$$

where

I is the moment of inertia of a pair of stiffeners about the centre of the web, or of a single stiffener about the face of the web;

t is the minimum required thickness of web;

S_t is the maximum permitted clear distance between stiffeners for thickness t.

NOTE Where, on the basis of requirements of strength, the web thickness provided is greater than the minimum required, or the stiffener spacing is made closer than the maximum permissible, the moment of inertia of the stiffeners need not be correspondingly increased.

Intermediate vertical stiffeners, when not acting as loadbearing stiffeners, can be joggled and can be in pairs placed one on each side of the web or single, and shall extend to the full depth of the web. Unless they are connected to the flanges, they shall be kept well clear of them.

7.3.2.2.2 *Horizontal stiffeners.* Where horizontal stiffeners are used in addition to vertical stiffeners they shall be as follows.

One horizontal stiffener, on one or both sides of the web, shall be placed at a distance from the compression flange equal to two-fifths of the distance from the compression flange to the neutral axis when the thickness of the web is less than:

$d_2/200$ for steel of grade 43 to BS 4360;

$d_2/180$ for steel of grade 50 to BS 4360;

$d_2/155$ for steel of grade 55 to BS 4360;

where d_2 is the depth of the web as defined in **7.3.1.2**.

The stiffener shall have a moment of inertia I not less than $4S_1\, t^3$ where I and t are as defined in **7.3.2.2.1** and S_1 is the actual distance between the vertical stiffeners.

A second horizontal stiffener, on one or both sides of the web, shall be placed on the neutral axis of the girder when the thickness of the web is less than:

$d_2/250$ for steel of grade 43 to BS 4360;

$d_2/225$ for steel of grade 50 to BS 4360;

$d_2/190$ for steel of grade 55 to BS 4360.

This stiffener shall have a moment of inertia I not less than $d_2\, t^3$ where I and t are as defined in **7.3.2.2.1** and d_2 is as defined in **7.3.1.2**.

Horizontal stiffeners shall extend between vertical stiffeners but need not be continuous over them, or connected to them.

7.3.2.2.3 *External forces on intermediate stiffeners.* When vertical intermediate stiffeners are subject to bending moments and shears due to the eccentricity of vertical loads, or the action of transverse forces, the moment of inertia I of the stiffeners given by **7.3.2.2.1** shall be increased as follows.

For bending moment on stiffener due to eccentricity of vertical loading with respect to the vertical axis of the web

$$\text{increase of } I = \frac{1.5MD^2}{Et}$$

For lateral loading on stiffener

$$\text{increase of } I = \frac{3PD^3}{Et}$$

where

M is the applied bending moment;

- P is the lateral force to be taken by the stiffener and deemed to be applied at the compression flange of the girder;
- D is the overall depth of girder;
- t is the thickness of web;
- E is Young's modulus (= 205 000 N/mm^2).

7.3.2.2.4 *Connection of intermediate stiffeners to web.* Intermediate vertical and horizontal stiffeners not subjected to external loads shall be connected to the web by welds or rivets in order to withstand a shearing force, in kilonewtons per millimetre run, between each component of the stiffener and the web, of not less than $t^2/8h$, where t equals web thickness in millimetres and h equals the projection, in millimetres, of the stiffener component from the web.

7.3.2.3 *Outstand of all stiffeners.* Unless the outer edge of each stiffener is continuously stiffened, the outstand of all stiffeners from the web shall be not more than the following:

for sections	16t for steel of grade 43 to BS 4360
	14t for steel of grade 50 to BS 4360
	12.5t for steel of grade 55 to BS 4360
for flats	12t for all steels

where t is the thickness of the section or flat.

Section 8. Fluctuating loads: permissible fatigue stresses

8.1 Detail design

All details shall be designed to avoid stress concentrations likely to result in excessive reduction of the fatigue strength of members or connections. Care shall be taken to avoid sudden changes of shape of a member or part of a member, especially in regions of tensile stress or local secondary bending.

Except where specifically stated to the contrary, the fatigue stresses permissible under this clause for any particular detail shall apply to all steels.

NOTE Members subjected to fluctuations of stress are liable to suffer from fatigue failure and this may be caused by loads that are very much lower than those that would be necessary to cause failure under a single application. The initiation of fatigue cracks is due, primarily, to stress concentrations introduced by the constructional details. Discontinuities such as bolt or rivet holes, welds and other local or general changes in geometrical form set up such stress concentrations from which fatigue cracks may be initiated, and these cracks may subsequently propagate through the connected or fabricated member.

8.2 Number of stress cycles

For calculation purposes the number of stress cycles, N, for all members of the structure shall be derived from the number of operating cycles as set out in Table 1 according to the class of utilization of the crane.

8.3 Loads and stresses to be considered

8.3.1 Loads. A verification of the adequacy of structural members for fatigue shall be made on the basis of the load combinations specified in **3.1**.

The precise combination of loads to be used will depend on the treatment adopted. For the simple treatment outlined in **8.4** the load combination given in **3.1.2.3.2** shall be used. For the more detailed treatment in the note to **8.4** the load combination given in **3.1.2.3.3** shall be used.

8.3.2 Stresses. Under the specified loading combinations the elements of a structure will be subjected to a variety of stress cycles in which both the degree of stress fluctuation and the level of maximum stress will vary. The degree of stress fluctuation shall be expressed as the ratio f_{min}/f_{max} where f_{min} is the minimum stress in the element during a cycle and f_{max} is the maximum stress in the element during the same cycle. The maximum stress level, whether tension or compression, corresponds to f_{max}.

8.4 Methods

Simple treatment. The nominal load spectrum factor, K_p, given in Table 2 according to the state of loading of the crane, shall be applied to the hook load to make an allowance for the varying loads handled by the crane throughout its life, having regard to its type and classification. Using this factored load, the ratio f_{min}/f_{max} shall be determined for the extreme conditions of stress that occur in a single typical operating cycle due to the combination of loadings specified in **3.1.2.3.2**. The maximum stress thus determined shall meet the requirements of **8.5**.

NOTE *More detailed treatment.* In the case of cranes where the duty can be defined in more detail, a more rigorous examination of fatigue is permitted, in agreement with the purchaser, using spectra of load and stress derived from a knowledge of the parameters of the service the crane is required to perform.

A method of assessing more precisely the cumulative effect of load and stress spectra is outlined in Appendix F. Where this method is used the manufacturer should, if requested, inform the purchaser of the basis on which the load and stress spectra have been determined.

In addition to the method given in Appendix F, reference may be made to BS 153.

8.5 Permissible fatigue stress

8.5.1 Table 24 sets out the permissible tensile and compressive fatigue stresses, P_{ft} and P_{fc}, according to the number of stress cycles, the class of constructional detail given in **8.6** and the ratio f_{min}/f_{max}. The tabulated stresses are applicable to steels of grades 43, 50 and 55 to BS 4360.

The value of f_{max} shall not exceed the appropriate permissible tensile or compressive fatigue stress, P_{ft} or P_{fc}, from Table 24.

Where co-existent bending and shear stresses are present, the principal stress at the point under consideration shall not exceed the appropriate permissible tensile or compressive fatigue stress, P_{ft} or P_{fc}, from Table 24.

8.5.2 Under no circumstances shall f_{max} exceed the permissible working stresses given in **4.3.2** or lower stresses required by any other clause in this standard.

8.6 Classes of constructional details

The classes of constructional detail A to G, referred to in Table 24, are described below and illustrated in Figure 7 to Figure 10.

a) *Class A*

1) Plain steel in the as-rolled condition with no gas-cut edges.

2) Members fabricated with full-penetration longitudinal or transverse butt welds with the weld overfill dressed flush with the plate surface and the weld proved free from defects by non-destructive examination, provided also that the members do not have exposed gas-cut edges.

Welds shall be dressed flush by machining or grinding, or both, which shall be finished in the direction parallel to the direction of the applied stress.

b) *Class B*

1) Members fabricated with continuous longitudinal butt welds with full or partial penetration made with either a submerged or open arc automatic process but with no intermediate stop-start positions within the weld length.

c) *Class C*

1) Members fabricated with continuous longitudinal fillet welds made with either a submerged or open arc automatic process but with no intermediate stop-start positions within the weld length.

2) Members fabricated with transverse non-load-carrying fillet or butt welded attachments with the weld fully machined.

3) Members of steels of grade 50 or 55 to BS 4360 fabricated or connected with rivets or bolts.

d) *Class D*

1) Members fabricated with full-penetration transverse butt welds made in the shop in the flat position, manual welds not giving deep penetration and automatic welds made by a process other than submerged arc.

2) Members fabricated with continuous longitudinal fillet welds with stop-start positions within the weld length.

3) Members fabricated with transverse non-load-carrying fillet or butt welded attachments with the weld toe lightly ground.

4) Members fabricated with longitudinal non-load-carrying fillet or butt welded attachments with the weld ends fully machined.

5) Members of steel of grade 43 to BS 4360 fabricated or connected with rivets or bolts.

e) *Class E*

1) Members fabricated with longitudinal non-load-carrying fillet or butt welded attachments with the weld ends lightly ground.

2) Girder webs with stiffeners in regions of combined bending and shear.

3) Members with stud shear connectors.

f) *Class F*

1) Members fabricated with transverse butt welds made on permanent backing material.

2) Members fabricated with transverse butt welds made by submerged arc welding or manually by deep-penetration methods.

3) Members fabricated with transverse non-load-carrying fillet or butt welded attachments.

4) Members fabricated with transverse butt welds in which the load is resisted by bending in the plate.

5) Members fabricated with longitudinal non-load-carrying fillet or butt welded attachments.

6) Members fabricated with intermittent longitudinal fillet welds.

7) Members fabricated with full-penetration cruciform butt welds.

8) Members fabricated with transverse load-carrying fillet welds.

9) The main chord members of a lattice girder or truss at the point where a bracing member is connected to it by a butt or fillet weld.

g) *Class G*

1) Members with intermittent longitudinal non-load-carrying attachments butt or fillet welded to their edges.

2) Members connected by longitudinal load-carrying fillet welds.

3) Members with partial-length welded cover plates.

4) The bracing member of a lattice girder or truss at the point where it is connected to a main member by a butt or fillet weld.

5) Members connected by load-carrying cruciform fillet welds.

8.7 Connections: riveted or bolted

8.7.1 Connections made with rivets and bolts. No allowance for fatigue shall be made in calculating the required number of rivets or bolts in a riveted or bolted connection except that all rivets or bolts subjected to reversal of stress shall be proportioned for the arithmetical sum of the load in the member corresponding to f_{max} plus 50 % of the load of opposite sign corresponding to f_{min}.

8.7.2 Connections made with friction grip bolts. No allowance for fatigue shall be made in calculating the required number of bolts.

8.8 Connection: load-carrying fillet welds

Load-carrying fillet welds shall be designed such that the stress on their total effective throat area does not exceed the relevant value given in Table 24 for class G details.

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

NOTE The ratio f_{min}/f_{max} is positive or negative respectively if the maximum and minimum stresses are of like or unlike sign.
Permissible fatigue stresses for intermediate numbers of stress cycles and for numbers of stress cycles in excess of 4.0×10^6 may be obtained from the straight line that results when P_f and N for the class of constructional detail and ratio f_{min}/f_{max} under consideration are plotted on a log/log basis.

	$\frac{f_{min}}{f_{max}}$	**Permissible tensile fatigue stress P_{ft}** (in N/mm²)								**Permissible compressive fatigue stress P_{fc}** (in N/mm²)							
		Number of cycles N								**Number of cycles N**							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class A constructional detail	1.0					432	432	432	425								
	0.9	432	432	432	432	392	390	386	384								
	0.8	391	384	377	369	362	355	349	342								
	0.7	372	362	352	343	333	324	315	306								
	0.6	355	344	332	321	310	299	289	279								
	0.5	342	329	316	303	290	279	267	256								
	0.4	313	301	290	278	268	257	247	238								
	0.3	291	281	270	260	250	241	232	223								
	0.2	271	261	252	242	233	225	216	208					432	432	432	432
	0.1	257	247	238	229	220	212	204	196	432	432	432	432	416	401	386	372
	0	241	233	224	216	208	200	193	186	403	388	374	360	347	334	321	309
	– 0.1	229	220	212	204	197	189	182	175	345	333	320	308	297	286	275	265
	– 0.2	217	209	201	194	187	180	173	167	305	293	282	271	261	257	241	232
	– 0.3	206	198	191	184	177	170	164	158	270	260	250	241	232	223	215	207
	– 0.4	196	189	182	175	168	162	156	150	241	233	224	216	208	201	193	186
	– 0.5	186	179	172	166	160	154	148	143	220	212	205	197	190	183	176	170
	– 0.6	178	171	165	158	152	146	141	135	203	195	188	181	174	167	161	155
	– 0.7	171	165	159	153	147	141	136	131	186	179	172	166	160	154	148	143
	– 0.8	164	158	153	147	142	136	131	127	172	166	160	155	149	144	139	134
	– 0.9	158	152	146	141	135	130	125	120	161	155	150	144	139	133	128	123
	– 1.0	151	146	140	135	130	125	121	116	151	146	140	135	130	125	121	116

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

	$\frac{f_{min}}{f_{max}}$	**Permissible tensile fatigue stress P_{ft}** (in N/mm²)								**Permissible compressive fatigue stress P_{fc}** (in N/mm²)							
		Number of cycles N								**Number of cycles N**							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class B constructional detail	1.0	432	432	432	432	425	420	415	410								
	0.9	412	408	399	390	382	375	372	364								
	0.8	392	381	371	360	350	340	331	321								
	0.7	374	360	346	333	320	307	295	283								
	0.6	357	341	325	309	295	281	267	255								
	0.5	341	330	311	292	275	259	244	230								
	0.4	325	306	288	270	264	238	224	210								
	0.3	299	182	265	249	234	220	207	195							432	432
	0.2	278	262	247	232	218	205	193	182				432	432	432	405	395
	0.1	260	245	230	216	203	191	179	168	432	432	432	407	384	363	343	324
	0	246	232	218	205	192	181	170	160	410	386	363	342	321	302	284	267
	– 0.1	233	219	206	194	182	171	161	151	347	328	309	291	275	259	244	230
	– 0.2	220	207	195	184	173	163	153	144	308	290	273	256	241	227	213	200
	– 0.3	209	197	185	173	163	153	144	135	273	257	242	228	215	202	190	179
	– 0.4	198	187	176	165	155	146	138	129	246	232	218	205	192	181	170	160
	– 0.5	190	179	168	158	149	140	131	123	224	211	198	186	175	164	154	145
	– 0.6	181	171	161	151	142	133	125	118	206	193	182	171	160	150	141	133
	– 0.7	172	162	153	144	139	127	121	114	190	179	168	158	149	140	131	123
	– 0.8	165	156	147	139	131	123	116	109	177	166	156	146	137	129	121	113
	– 0.9	161	151	142	134	126	118	111	105	164	154	146	137	129	121	114	108
	– 1.0	154	145	136	128	121	113	107	100	154	145	136	128	121	113	107	100

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

	$\frac{f_{min}}{f_{max}}$	Permissible tensile fatigue stress P_{ft} (in N/mm²)								Permissible compressive fatigue stress P_{fc} (in N/mm²)							
		Number of cycles N								Number of cycles N							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class C constructional detail	1.0	432	432	432	432	432	432	432	432								
	0.9	416	407	398	389	380	371	363	355								
	0.8	400	384	369	354	339	325	312	299								
	0.7	384	364	344	325	307	290	273	258								
	0.6	369	345	322	300	280	262	244	228								
	0.5	352	326	301	278	257	237	219	203								
	0.4	320	296	274	253	234	216	199	184								
	0.3	296	274	253	234	216	199	184	170					432	432	432	432
	0.2	273	252	233	216	199	184	170	157	432	432	432	432	429	400	374	340
	0.1	254	235	217	200	185	171	158	145	432	432	404	374	346	300	297	275
	0	236	218	202	186	172	159	147	136	397	367	339	313	288	266	246	234
	– 0.1	221	205	190	176	162	150	139	129	338	313	289	267	246	227	210	200
	– 0.2	211	195	181	167	154	142	131	121	296	274	253	234	216	199	184	170
	– 0.3	202	186	172	158	146	134	124	114	262	243	224	207	192	177	164	151
	– 0.4	190	176	163	150	139	129	119	110	236	218	202	186	172	159	147	136
	– 0.5	181	168	155	148	132	122	113	104	214	199	184	170	157	145	134	124
	– 0.6	172	160	148	137	126	17	108	100	198	183	169	156	143	132	122	113
	– 0.7	165	153	142	131	121	112	104	96	181	168	155	143	132	122	113	104
	– 0.8	161	148	137	126	116	107	99	91	169	156	145	133	123	114	105	97
	– 0.9	153	142	131	121	112	104	96	89	156	145	134	124	115	107	99	92
	– 1.0	148	137	126	116	107	99	91	84	148	137	126	116	107	99	91	84

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

	$\frac{f_{min}}{f_{max}}$	Permissible tensile fatigue stress P_{ft} (in N/mm²)								Permissible compressive fatigue stress P_{fc} (in N/mm²)							
		Number of cycles N								Number of cycles N							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class D constructional detail	1.0	432	432	432	432	432	432	432	420								
	0.9	409	398	390	380	370	361	352	343								
	0.8	387	371	355	339	324	310	297	284								
	0.7	366	345	326	307	289	272	256	242								
	0.6	348	324	302	281	261	243	226	210								
	0.5	339	311	285	261	239	219	201	184								
	0.4	307	282	258	237	217	199	182	167								432
	0.3	280	257	236	216	198	182	167	153				432	432	432	432	400
	0.2	257	236	216	198	182	167	153	140	432	432	432	430	392	356	324	295
	0.1	239	219	201	185	169	155	142	130	432	407	373	341	312	265	261	239
	0	221	203	186	171	156	143	131	120	365	335	308	282	259	238	218	200
	– 0.1	209	192	176	161	147	135	124	113	313	288	262	242	222	204	187	171
	– 0.2	198	182	167	153	140	128	117	108	274	252	231	212	195	179	164	150
	– 0.3	187	172	158	145	132	121	111	102	244	224	206	189	173	158	145	133
	– 0.4	179	164	150	137	126	115	105	96	221	203	186	170	156	143	131	120
	– 0.5	170	156	143	131	120	110	100	92	200	183	168	154	141	130	119	109
	– 0.6	163	149	137	125	114	105	96	88	183	169	155	142	130	120	110	101
	– 0.7	155	142	131	120	110	101	93	85	170	156	143	131	120	110	100	92
	– 0.8	150	138	126	115	105	96	88	80	157	144	132	121	111	101	93	85
	– 0.9	143	131	120	110	101	93	85	78	146	134	123	113	103	95	87	79
	– 1.0	140	128	117	107	98	90	82	75	140	128	117	107	98	90	82	75

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

	$\frac{f_{min}}{f_{max}}$	Permissible tensile fatigue stress P_{ft} (in N/mm²)								Permissible compressive fatigue stress P_{fc} (in N/mm²)							
		Number of cycles N								Number of cycles N							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class E constructional detail	1.0	432	432	432	432	432	432	432	432								
	0.9	408	394	379	365	352	338	336	314								
	0.8	382	360	338	317	298	280	265	247								
	0.7	365	335	308	282	260	238	218	200								
	0.6	344	311	281	257	228	206	185	167								
	0.5	324	290	259	231	206	184	164	146								432
	0.4	286	255	228	203	181	162	144	128					432	432	432	410
	0.3	251	230	205	183	163	146	130	116			432	432	422	375	334	297
	0.2	233	209	186	166	148	132	117	105	432	432	398	355	317	282	252	224
	0.1	214	192	171	153	136	121	108	96	402	359	320	285	254	226	201	179
	0	200	179	159	142	127	113	100	90	335	299	266	237	211	188	167	148
	– 0.1	189	169	190	134	119	106	94	84	186	255	228	203	181	161	144	128
	– 0.2	177	158	141	126	113	100	90	80	251	224	200	177	158	141	125	111
	– 0.3	170	152	135	120	107	95	85	76	223	197	177	158	140	125	111	99
	– 0.4	161	144	128	114	101	90	80	72	197	177	158	141	126	112	100	90
	– 0.5	153	137	122	109	97	86	77	69	181	162	144	129	115	102	91	81
	– 0.6	148	132	118	105	93	83	74	66	166	149	132	118	105	94	83	74
	– 0.7	140	125	112	100	89	80	71	64	153	137	120	109	97	87	77	69
	– 0.8	134	120	107	96	85	76	68	61	142	127	113	101	90	80	71	63
	– 0.9	129	115	103	92	82	73	65	60	133	119	106	94	84	75	66	59
	– 1.0	125	112	100	89	80	71	63	57	125	112	100	89	80	71	63	57

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

	$\frac{f_{min}}{f_{max}}$	Permissible tensile fatigue stress P_{ft} (in N/mm²)								Permissible compressive fatigue stress P_{fc} (in N/mm²)							
		Number of cycles N								Number of cycles N							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class F constructional detail	1.0	432	432	432	432	432	432	432	410								
	0.9	413	391	370	350	331	313	297	281								
	0.8	385	353	323	295	270	247	226	206								
	0.7	361	322	287	256	228	203	181	161								
	0.6	337	296	259	226	198	173	151	132								432
	0.5	316	273	236	203	175	151	130	112					432	432	432	410
	0.4	278	241	208	179	154	133	114	99			432	432	387	344	306	272
	0.3	246	213	184	159	137	118	102	88	432	432	382	332	289	251	218	189
	0.2	225	195	168	145	125	108	93	80	409	354	306	264	228	197	170	147
	0.1	203	176	152	131	112	97	83	72	337	292	252	217	187	161	139	120
	0	187	162	140	121	104	90	77	67	284	245	212	183	158	136	117	101
	– 0.1	175	152	131	113	98	84	73	63	246	213	184	159	137	118	102	88
	– 0.2	166	143	124	107	92	79	68	59	217	188	162	140	121	104	90	77
	– 0.3	154	133	115	99	85	74	63	55	195	169	146	126	108	93	80	69
	– 0.4	146	126	109	94	81	70	60	52	175	152	131	113	98	84	73	63
	– 0.5	138	119	103	89	77	66	57	49	162	140	121	104	90	77	66	57
	– 0.6	133	114	99	85	73	63	54	47	150	130	112	97	83	72	62	53
	– 0.7	129	111	96	82	71	61	53	45	138	119	103	89	77	66	57	49
	– 0.8	124	107	93	80	69	59	51	44	129	111	96	82	71	61	52	46
	– 0.9	116	101	87	75	64	56	48	41	120	104	90	77	67	57	49	43
	– 1.0	112	97	84	72	62	54	46	40	112	97	84	72	62	54	46	40

Table 24 — Permissible tensile and compressive fatigue stresses P_{ft} and P_{fc} for various classes of constructional detail (see section 8) **in steels of grades 43, 50 and 55 to BS 4360**

	$\frac{f_{min}}{f_{max}}$	**Permissible tensile fatigue stress P_{ft} (in N/mm²)**								**Permissible compressive fatigue stress P_{fc} (in N/mm²)**							
		Number of cycles N								**Number of cycles N**							
		3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6	3.2×10^4	6.3×10^4	1.25×10^5	2.5×10^5	5.0×10^5	1.0×10^6	2.0×10^6	4.0×10^6
Class G constructional detail	1.0	432	432	432	432	432	432	432	400								
	0.9	407	376	347	320	295	272	250	231								
	0.8	375	331	292	257	227	200	176	155								
	0.7	344	295	253	216	184	157	134	115							432	432
	0.6	314	264	222	186	156	131	110	92				432	432	432	366	315
	0.5	287	238	198	164	135	112	93	77		432	432	370	320	260	227	180
	0.4	249	207	171	142	117	97	80	66	432	367	310	261	220	186	156	131
	0.3	215	179	148	123	101	84	70	58	346	291	243	204	170	142	119	99
	0.2	196	163	135	112	93	77	63	52	281	237	198	166	139	116	97	81
	0.1	177	147	122	101	83	69	57	47	241	202	169	141	118	98	82	68
	0	158	131	109	90	75	62	51	42	209	175	146	121	101	84	70	58
	– 0.1	147	123	102	84	70	58	48	40	183	154	128	107	89	74	62	52
	– 0.2	137	114	95	78	65	54	45	37	166	138	115	95	79	65	54	45
	– 0.3	128	107	89	73	61	50	42	35	149	124	103	90	72	60	49	41
	– 0.4	122	101	84	70	58	48	40	33	137	114	95	78	65	54	45	37
	– 0.5	116	97	80	66	54	45	37	31	126	105	87	73	60	50	42	35
	– 0.6	110	91	76	63	52	43	36	29	115	96	80	67	56	46	39	32
	– 0.7	103	86	71	59	49	41	34	28	110	91	76	63	52	43	36	29
	– 0.8	99	82	68	57	47	39	32	27	102	84	70	58	48	39	32	27
	– 0.9	95	79	65	54	45	37	31	26	95	79	65	54	45	37	31	26
	– 1.0	91	75	63	52	43	35	29	24	91	75	63	52	43	35	29	24

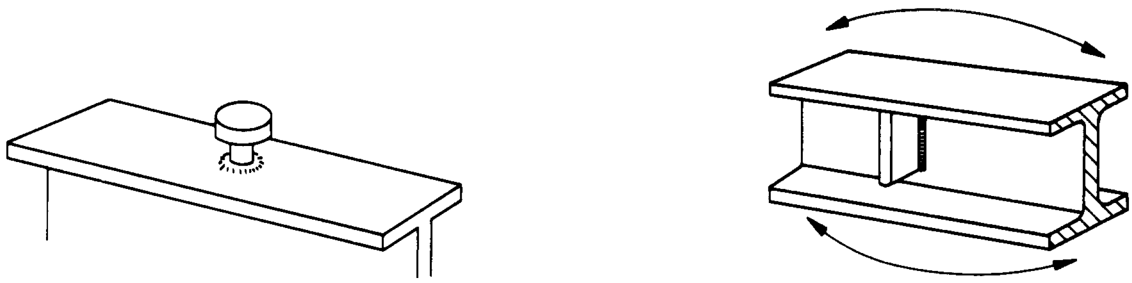


Figure 7 — Typical class E weld details

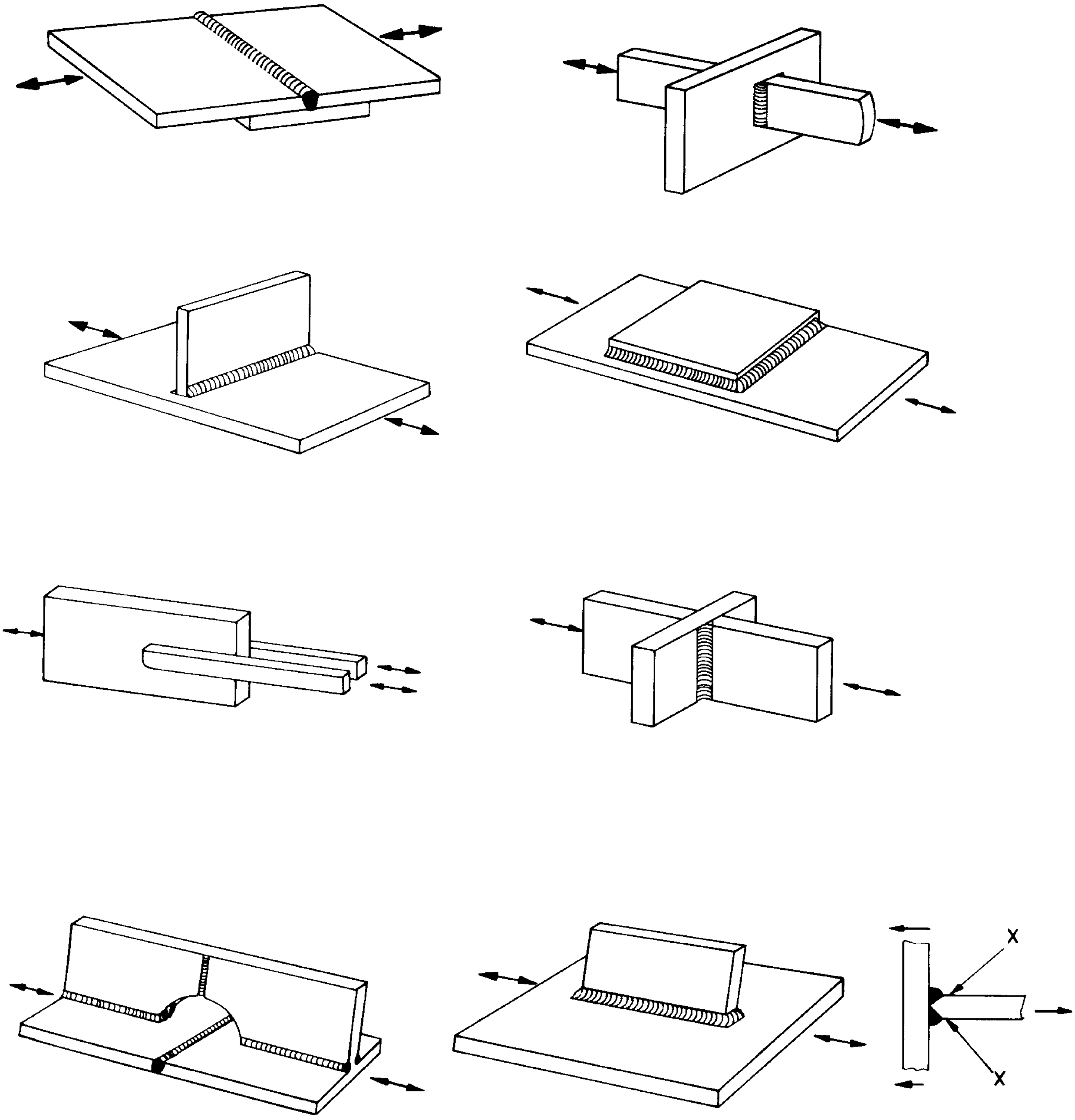


Figure 8 — Typical class F weld details

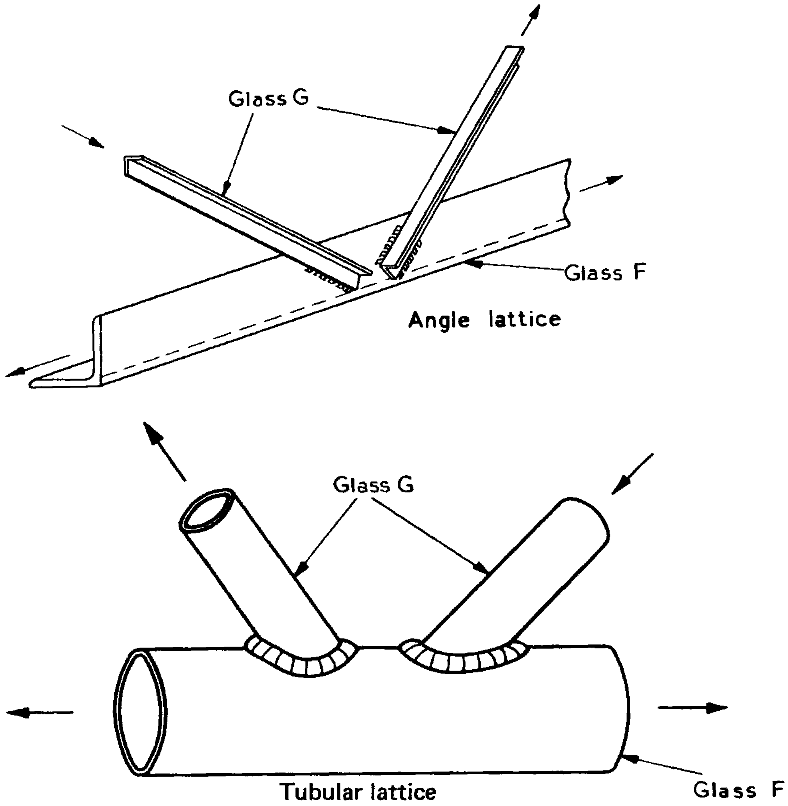


Figure 9 — Typical class F and class G weld details

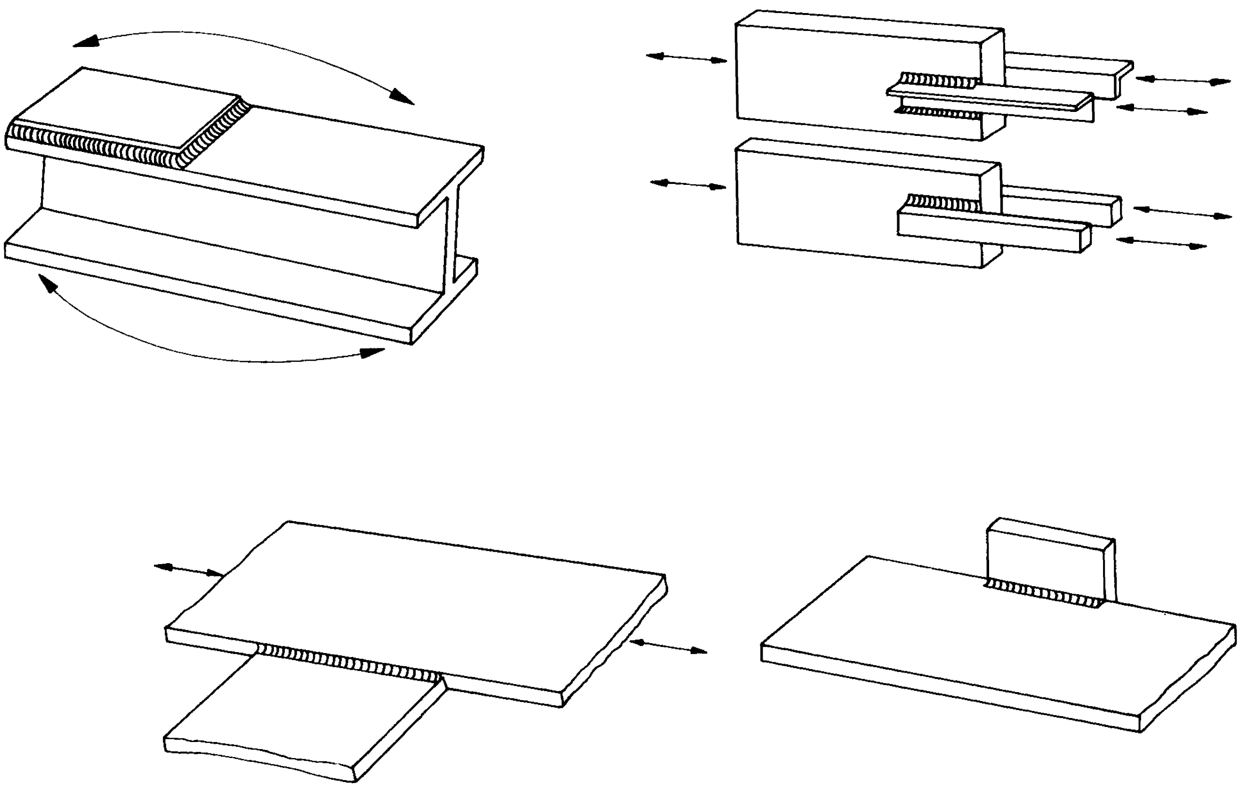


Figure 10 — Typical class G weld details

Appendix A Typical crane classifications

Table 25 to Table 30 show the ranges of classes of utilization, states of loading and the resulting group classifications normally associated with particular types and applications of crane. They should not be considered as a substitute for the procedure for determining the classification as specified in section 2. The lists of crane types and applications are not comprehensive. Cranes such as floating cranes, breakdown cranes and pedestal cranes for off-shore use, for example, all have special design requirements associated with their particular applications. In such cases, full account of these requirements should be taken when determining the appropriate group classifications.

Table 25 — Overhead travelling industrial type cranes (O.T.C.)

Type and/or application	Class of utilization	State of loading	Group classification
Power stations, engine houses, etc.	U2 to U4	Q1	A1 to A3
Light work shop duty (maintenance, repairs, assembly, etc.)	U2 to U4	Q1 to Q2	A1 to A4
Light stores duty	U2 to U4	Q1 to Q2	A2 to A4
Medium and heavy warehouse duty	U4 to U6	Q2 to Q3	A4 to A6
Medium and heavy workshop duty	U4 to U5	Q1 to Q3	A4 to A6
Goliath cranes for general hook service	U4 to U6	Q1 to Q3	A4 to A6
Goliath cranes for container handling	U4 to U7	Q2	A4 to A7
Goliath and bridge cranes for grabbing work	U5 to U8	Q4	A7 to A8
Shipyard cranes	U5 to U6	Q2 to Q3	A5 to A7
Ladle cranes for foundry work	U4 to U5	Q3 to Q4	A6 to A7
Magnet cranes for steel stockyards	U5 to U6	Q2 to Q3	A6 to A7
Magnet cranes for scrapyard work	U5 to U6	Q3 to Q4	A6 to A8
Forging cranes	U6 to U9	Q3 to Q4	A7 to A8

Table 26 — Overhead travelling steelworks cranes

Type and/or application	Class of utilization	State of loading	Group classification
Ladle cranes	U5 to U7	Q4	A7 to A8
Pig/scrap breaking cranes	U6 to U8	Q4	A8
Ingot strippers Soaking pit mould handling cranes Vertical ingot chargers	U6 to U8	Q4	A8
Furnace charging cranes	U6 to U8	Q4	A8
Forging cranes	U6 to U8	Q3 to Q4	A7 to A8
Process cranes: on line	U6 to U8	Q3 to Q4	A7 to A8
Process cranes: off line	U5 to U6	Q2 to Q3	A5 to A7
Heavy mill service	U6 to U8	Q2 to Q3	A6 to A8
Service and maintenance cranes	U4 to U5	Q1 to Q2	A3 to A5

Table 27 — Transporters

Type and/or application	Class of utilization	State of loading	Group classification
Medium duty: general use	U5 to U6	Q2 to Q3	A5 to A6
Heavy duty: intermittent grabbing and magnet work	U6 to U7	Q3 to Q4	A7 to A8
Extra-heavy duty: continuous grabbing and magnet work	U8 to U9	Q3 to Q4	A8

Table 28 — High pedestal or portal jib cranes and derrick cranes

Type and/or application		Class of utilization	State of loading	Group classification
Medium duty in docks and elsewhere handling general cargo and piece goods.		U3 to U5	Q1 to Q2	A2 to A5
Full-time container handling		U4 to U7	Q2	A4 to A7
Grabbing or magnet work	intermittent	U5 to U7	Q2 to Q3	A5 to A8
	continuous	U5 to U7	Q3 to Q4	A6 to A8
Shipyard cranes		U3 to U5	Q1 to Q2	A2 to A5

Table 29 — Tower cranes

Type and/or application	Class of utilization	State of loading	Group classification
Normal duty: general use on building sites	U2 to U4	Q1 to Q2	A2 to A3
Medium duty: general use at a permanent location, etc.	U4 to U6	Q2 to Q3	A4 to A6

Table 30 — Freight container cranes

Type and/or application	Class of utilization	State of loading	Group classification
Freight container transporter	U5 to U7	Q2	A5 to A7
Goliath for container handling duty	U4 to U7	Q2	A4 to A7

Appendix B The use of steels of higher tensile strengths than those of steels to BS 4360

The use of suitable steels with higher tensile strengths than those covered by BS 4360 is permissible in the case of mobile crane structures and similar appliances where the utmost economy in weight is essential, provided that working stresses are rigorously analysed having regard to loading conditions, and the design of the structure is verified by adequate testing.

The working stresses thus derived should not exceed the permissible stresses calculated according to **5.1**.

In all cases for steels having a yield stress greater than 82 % of the ultimate stress the basic stresses $P_{at.bas}$, $P_{ac.bas}$ for $l/r \leqslant s_O$, $P_{bt.bas}$, $P_{bc.bas}$ and $P_{qc.bas}$ should be taken respectively as the basic stresses for steel of grade 55 to BS 4360 according to **5.1**, increased in the ratio

$$\frac{Y_s + U_s}{Y_{s.55} + U_{s.55}}$$

where $U_{s.55}$ and $Y_{s.55}$ are the minimum ultimate tensile strength and the yield stress of steel of grade 55 to BS 4360, and U_s and Y_s are the minimum ultimate tensile strength and the yield stress for the steel under consideration.

Extreme care has to be taken in the use of these steels where the design criteria are crippling, buckling, or lateral instability, in applications where the increased deflections resulting from higher stresses may give rise to critical conditions. In all cases it is essential to ensure that any steel used has adequate properties in respect of impact at low temperature, weldability and fatigue.

Appendix C Effective lengths of crane jibs, considered as uniform struts

C.1 General

In this appendix, crane jibs are considered as uniform struts from the point of view of buckling in elevation and plan. The overall slenderness ratio (l/r) of the jib in each plane can be obtained by dividing the effective length of the jib by the least radius of gyration of the complete jib section occurring in the middle third of the actual length. It should be noted that the effective length and the radius of gyration taken have to be those applicable to that plane of the jib for which the slenderness ratio is required.

The middle third of the actual length of the jib extends along the jib for a distance of $L/3$ measured from a point which is $L/3$ from the jib head.

C.2 Rope-supported jibs

The following refers only to luffing crane jibs in which the jib head is supported by the derricking rope, and the hoist rope runs over the jib head pulley. The side elevations of typical arrangements are shown in Figure 11, Figure 12 and Figure 13.

a) *In elevation*. Considering buckling in the luffing plane, it is clear that both ends of the jib of a luffing crane are fixed in position but free to rotate. For all positions of the jib the effective length can thus be taken as equal to the actual length ($l = L$).

b) *In plan*. The lower end of the jib can be considered as completely restrained in the slewing plane by the jib pivots. The jib head is supported by the derricking rope and the hoist rope runs over the jib head pulley. The effective length of the jib in plan will thus depend upon the lateral restraint applied to the jib head by these supporting ropes, and will vary with the angle of the jib and the tensions in the two ropes.

1) A general expression for determining the effective length of the jib in plan at any particular angle is given by

$$l = L\left\{2 - \frac{C(D + KH)}{A_H D + KA_D H}\right\}$$

where

l is the effective length of the jib (lateral buckling) (in m);

L is the actual length of the jib (in m);

K is the ratio of load applied to jib head by the derricking rope to that applied by the non-vertical part of the hoist rope;

C, D, H,
A_D and A_H are dimensions (in m) shown in Figure 11.

2) In the special case where the fixed pulleys for derricking rope and hoist rope are in one vertical line (Figure 12) the general expression above then simplifies to

$$l = L\left\{2 - \frac{C}{A}\right\}$$

where A is the dimension in fact shown in Figure 12.

3) In the special case where the fixed pulleys for derrick rope and hoist rope are vertically above the jib pivot point (Figure 13), as $C = A$, the effective length is then equal to the actual length for all positions of the jib.

C.3 Cantilever jibs

The following refers only to cantilever crane jibs that are luffed by some means acting on an extension of the jib behind the jib pivot. The side elevations of typical arrangements are shown in Figure 14 and Figure 15.

a) *In elevation*. Considering buckling in the luffing plane, the jib is not free to rotate about the jib pivot as movement of the lower end is prevented by the luffing mechanism. The lower end of the jib can thus be considered as encastred up to the jib pivot. The hoist rope provides the only restraint to deflection at the jib head.

b) *In plan*. The lower end of the jib can be considered as completely restrained in the slewing plane by the jib pivots. The hoist rope again provides the only restraint to deflection at the jib head.

Both these cases are covered by the treatment that follows, the effective length ratio depending upon the tension in the hoist rope, its position, and the angular elevation of the jib.

1) Where the hoist rope runs parallel to the longitudinal axis of the jib (see Figure 4), the ratio of the effective length to the real length in elevation and plan can be obtained from Figure 16 where it is plotted against $1 + f \sin \theta$ for ratios of H/L from 0.7 to 1.30 .
In Figure 16:

f is the number of falls on the hook;

θ	is the angle of elevation of the jib;
H and L	are the dimensions shown in Figure 14.

2) Where the hoist rope does not run parallel to the longitudinal axis of the jib (see Figure 15), the ratio of the effective length to the real length in elevation and plan can be obtained from Figure 16 where it is plotted against $\cos \alpha + f \sin \theta$ for ratios of H/L from 0.7 to 1.30.
In Figure 16:

f	is the number of falls on the hook;
θ	is the angle of elevation of the jib;
α	is the angle in elevation at jib head between the hoist rope and longitudinal jib axis;
H and L	are the dimensions shown in Figure 15.

NOTE Attention is drawn to the Building Research Station paper "Stiffness of a crane jib" by J.F. Eden and R.H. Wood, published in *The Engineer* 29 July 1960.

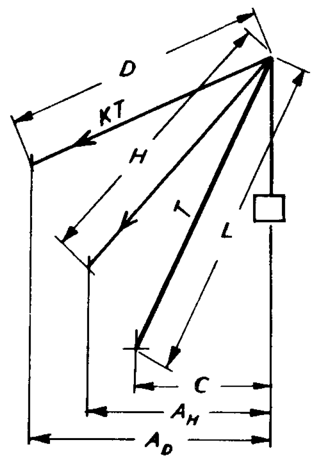


Figure 11 — General case for determining effective length of jib

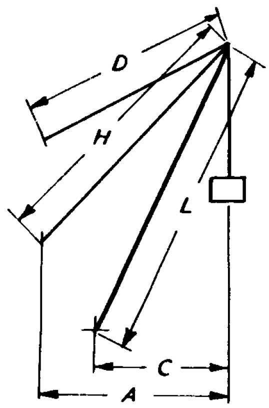


Figure 12 — Special case where fixed pulleys for derricking and hoist ropes are in one vertical line

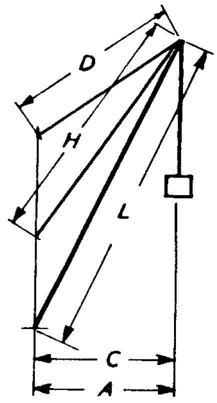


Figure 13 — Special case where fixed pulleys for derricking and hoist ropes are vertically above jib pivot point

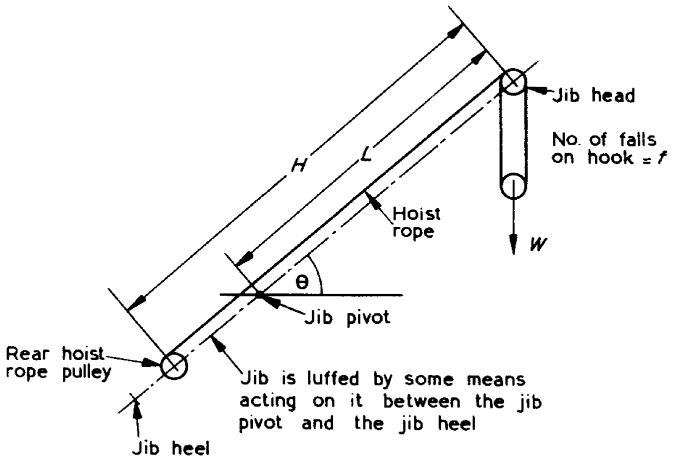


Figure 14 — Cantilever crane jib with hoist rope parallel to jib axis

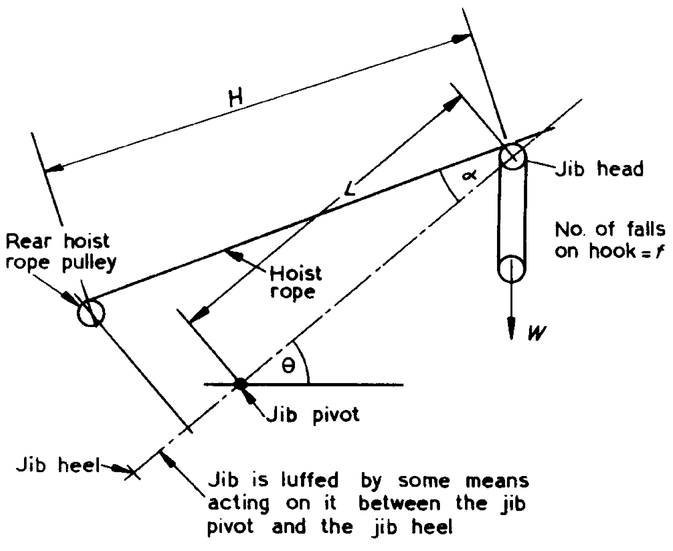


Figure 15 — Cantilever crane jib with hoist rope not parallel to jib axis

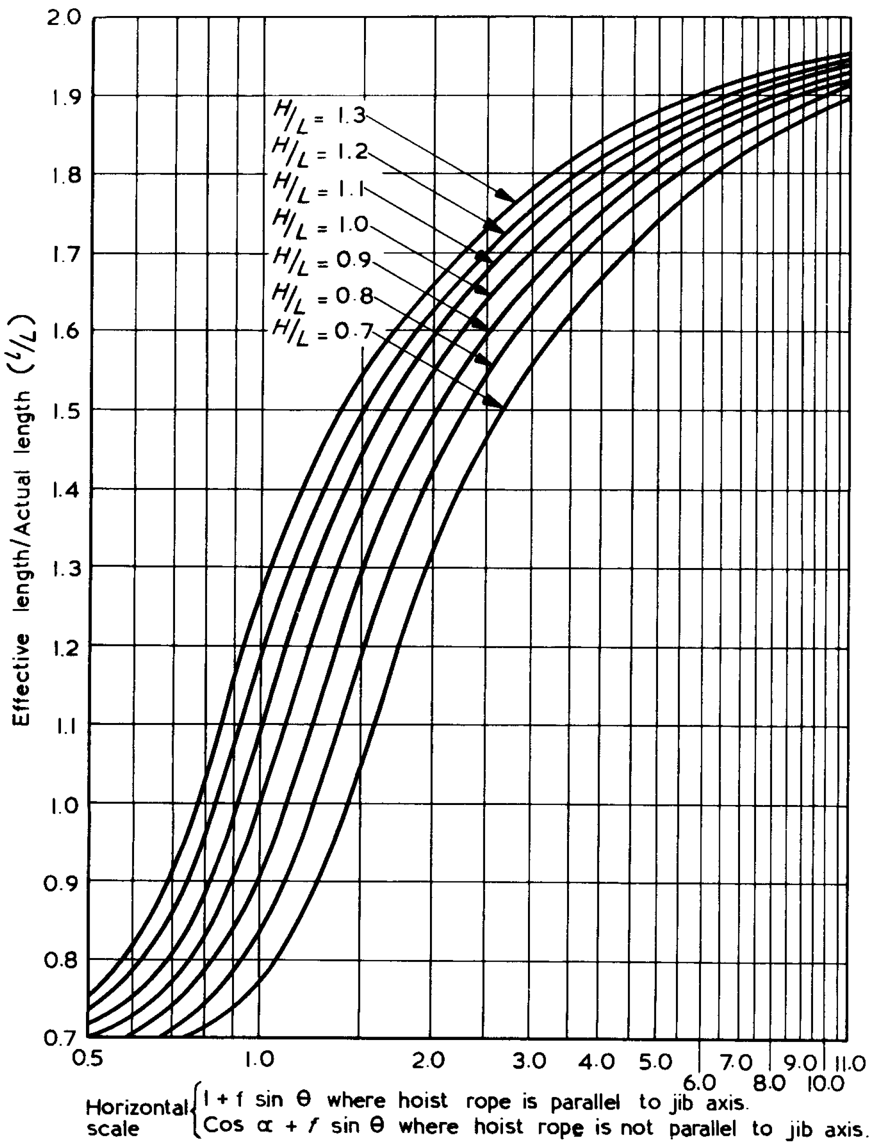


Figure 16 — Ratio of effective length/actual length for cantilever crane jibs

Appendix D Basic formula for calculation of C_s

The critical compression stress C_s (in N/mm²) for sections symmetrical about the x-x axis may be calculated from

$$C_s = \frac{\pi}{Z_x l}\sqrt{\left\{\frac{EI_y GK}{\mu}\left(1+\frac{\pi^2 w}{GKl^2}\right)\right\}}$$

where

Z_x is the gross section modulus about x-x axis;

l is the effective length of compression flange;

$$\mu = \frac{I_x - I_y}{I_x} \;;$$

I_x is the moment of inertia of the whole section about x-x axis;

I_y is the moment of inertia of the whole section about y-y axis;

E is Young's modulus (= 205 000 N/mm²);

G is the modulus of rigidity (taken as 0.4E);

K is the appropriate torsion constant;

w is the warping constant (= $\frac{EI_f h_c^2}{2}$ for I sections);

H_c is the distance between flange centroids;

I_f is the moment of inertia of the compression flange only about y-y axis of the girder.

a) *For I sections*. The above formula reduces to

$$C_s = \frac{410\,000}{Z_x l}\sqrt{\left\{\frac{I_y K}{\mu}\left(1 + 12.3\frac{I_f h_c^2}{K l^2}\right)\right\}}$$

For sections composed of approximately rectangular elements,

$$K = \Sigma\left(\frac{bt^3}{3}\right)$$

approximately, where b and t are breadth and average thickness of each element.

b) *For channel and Z sections*. The formula in a) above gives conservative values.

c) *For box members*. Conservative values of C_s are obtained by substituting in the formula in a) above,

$$K = \frac{4A_e^2}{\Sigma(S/t)} \text{ approximately}$$

where

A_e is the total enclosed area of section;

S is the length of each element of the periphery;

t is the thickness of each element (in the case of curtailed flanges, the effective thickness);

e.g. for a box of depth d, width b, and uniform thickness t,

$$K = \frac{2b^2d^2t}{d+b} \text{ approximately.}$$

d) *For a plate or flat in bending in a plane parallel to its surface*. Substituting appropriate values of K, etc.,

$$C_s = 410\,000\,\frac{t^2}{lD}$$

where

t is the thickness;

D is the depth;

I is the effective length of part in compression.

Appendix E Design checks for stiffened compression flanges

E.1 Introduction

E.1.1 *Basis of method*. The method set out in this appendix for checking the design of the stiffened compression flanges of box girders is based upon a paper by J D Dwight and C H Little of Cambridge University Engineering Department entitled "Stiffened steel compression panels — A design approach" [reference CUED/C-Struct/TR38 (1974)].

Throughout this appendix, linear dimensions are taken to be in millimetres, areas in square millimetres, stresses in newtons per square millimetre.

E.1.2 *Form of construction*. The form of construction covered by this appendix is shown in Figure 17. It comprises box-girder compression flanges composed of a skin plate to which identical longitudinal stiffeners are attached at equal spacings. The plate and stiffeners are supported at intervals by cross frames or diaphragms.

A stiffened panel is taken as that part of the compression flange, comprising the skin plate and stiffeners, lying between adjacent cross frames.

Typical stiffeners are shown in Figure 18. They may be of open cross section such as flats, tees and angles, or of closed cross section such as trough or vee sections.

Plate elements are those parts of the skin plating lying between adjacent lines at which stiffeners are attached, as shown by dimensions b, b_1 and b_2 in Figure 17.

This appendix considers a typical unit comprising a stiffener plus its associated width of skin plating that extends to midway between the stiffener and the stiffeners on each side of it. This plate stiffener unit is treated as a column between the supports provided by the cross frames, which are designed to have adequate stiffness. It is assumed that the equispaced stiffeners are of the same basic grade of steel as the skin plating, the requirements of **E.2** ensuring that they are of stocky cross section and would reach yield stress at their extreme fibres before suffering local collapse.

E.2 Requirements for stiffeners and connections

E.2.1 *Dimensional representation*. The symbols used in this appendix to represent the dimensions of stiffened panels and stiffeners are indicated in Figure 17 and Figure 18 respectively.

In Figure 18, for stiffeners of closed cross section dimensions b_3 and b_4 are measured between the intersections of the centre lines of the plate elements. For angle and tee stiffeners, dimensions are taken as the values given in specifications and section tables.

E.2.2 ***Dimensional requirements***

E.2.2.1 *Stiffeners of closed cross section*. The ratios b_3/t_O and b_4/t_O of the stiffener elements are not to exceed the values given in columns 2 and 3 of Table 31.

E.2.2.2 *Stiffeners of open cross section*

E.2.2.2.1 *Plain flats*. The ratio d/t_O is not to exceed the values given in column 4 of Table 31.

Table 31 — Dimensional requirements for stiffeners

Grade of steel to BS 4360	Type of stiffeners			
	Closed stiffeners		Plain flats	Angles and tees
	b_3/t_O	b_4/t_O	d/t_O	D
43	39	32	9	10.5
50	32	28	8	9
55	28	25	7	8

E.2.2.2.2 *Angles and tees*. Where the value of C_1 as defined below does not exceed that of D given in column 5 of Table 31 the local buckling resistance is satisfactory. Where C_1 exceeds D, it is essential that the stiffeners be given support by connection to cross frames at intervals not greater than $\frac{10 \cdot C_1 \cdot C_2 \cdot D}{\sqrt{(C_1^2 - D^2)}}$

where C_1 and C_2 are as follows:

a) *For angles to BS 4848-4*

$$C_1 = \frac{d + 0.55d_1}{t_0} \quad C_2 = 0.375d - 0.08d_1$$

b) *For tees to BS 4-1*

$$C_1 = C' \times \frac{d}{t_O}$$

where C' is obtained from Figure 19 according to the ratios t_1/t_O and d_1/d

$$C_2 = 0.24\ d_1$$

The connections between angle and tee stiffeners and cross frames are to be made in accordance with **E.2.3**.

E.2.3 *Connections between angle and tee stiffeners and cross frames*. The connections between cross frames and the legs of stiffeners that lie parallel to the skin plate should be capable of preventing rotations of the legs of the stiffeners in their own plane. The cross frames should also be capable of restraining such rotations. Under these end conditions a stiffener may be considered as encastered against movements parallel to the skin plating.

Where either of these conditions cannot be met, the maximum cross frame spacing given in **E.2.2.2.2** is to be halved.

E.3 Outline of checking procedures

E.3.1 *Methods*. Simple and detailed methods of carrying out design checks are given in **E.5** and **E.6** respectively, both of which would lead to adequate designs. The detailed method incorporates a number of refinements that require additional calculations but that also tend to allow higher stresses than those permitted by the simple method. In some cases, therefore, the detailed method provides a more economical design.

The design checks apply to the stiffened compression flanges of box members or to those parts of stiffened flanges that are in compression. The compression may be due to axial loading on the member as a whole, to bending about one or both principal planes, or to a combination of axial loading and bending. Torsion might also be present.

The stresses taken into account consist of longitudinal compressive stresses and shear stresses the latter including those due to any torsion present. A simplified representation of the variation of stresses in a stiffened compression panel is shown in Figure 20.

E.3.2 ***Application of checks***

E.3.2.1 *General*. In order to guard against the various modes of failure that could occur there are three collapse checks and, where the stiffened compression panel has more than two stiffeners, a check against local bulging is also required.

E.3.2.2 *Bending in one principal plane*. Where a symmetrical box section is subjected to bending in one principal plane only, in the absence of shear lag, longitudinal stresses may be assumed constant across the width of the compression flange. This flange only is subjected to the collapse and serviceability checks described in **E.2.3**. Where axial compression is also present, the checks are applied to the flange subject to the highest (uniform) compressive stress. The checks are not applied to the webs of the box that are stressed primarily in shear.

E.3.2.3 *Bending in both principal planes*. When bending occurs in both principal planes, whether or not axial compression of the box girder is also present, there is a variation of compressive stress across the width of all faces of the box. The checks described in **E.3.3** are then applied to the two faces adjacent to the corner carrying the greatest compressive stress.

E.3.2.4 *Axial compression only*. When axial compression only is present the checks are applied to two adjacent faces of the box.

E.3.3 *Checks against collapse and local bulging*. The purpose of the checks and the modes of failure that they cover are as follows:

collapse check 1: to ensure that local failure of the complete cross section of the stiffened panel at the end carrying the higher compressive stress does not occur;

collapse check 2: to ensure that inward failure of a plate-stiffened unit considered as a column between supports at the cross frames does not occur due to failure of the skin plating in compression;

collapse check 3: to ensure that outward failure of a plate-stiffened unit considered as a column between support at the cross frames does not occur due to failure of the stiffener tip in compression;

check 4, local bulging: to ensure that no plate element develops a local bulge.

E.4 Factored stresses

The paper, referred to in **E.1.1**, upon which this appendix is based, is written in terms of limit-state design. In order to make the method compatible with the design approach used in this standard, in certain cases it is necessary for the stresses obtained from an elastic analysis of the structure to be multiplied by a factor K. Wherever they occur, stresses thus multiplied are termed “factored” stresses.

The value of K, which is applied where indicated to both compressive and shear stresses, depends upon the load combination under consideration (as defined in **3.1**) and the duty factor for the type and class of crane (see **4.3**).

The value of K used for the load combinations covered by this appendix is as given in Table 32.

Table 32 — Value of factor *K*

Load combination from 3.1	Value of factor K
Crane in use without wind	$K = \dfrac{1.70}{\text{duty factor}}$
Crane in use with "in-service" wind (**3.1.2.2**)	$K = \dfrac{1.50}{\text{duty factor}}$
Crane in "out-of-service" condition (**3.1.2.4**) Crane being erected or dismantled (**3.1.2.5**) Crane in collision with resilient buffers (**3.1.2.6**)	$K = 1.25$

The general symbols used in this appendix for stresses derived from an elastic analysis of the structure are:

f_c longitudinal compressive stress;

f_q shear stress.

The equivalent factored stresses are indicated as:

f'_c $(= f_c \times K)$ factored longitudinal compressive stress;

f'_q $(= f_q \times K)$ factored shear stress.

Other symbols are defined at their first occurrence in the text.

E.5 Simple method for checks against collapse and local bulging

E.5.1 *General*. This method uses the design curves given in Figure 20 and Figure 21. The same data is also presented in Table 28 and Table 32 respectively. The checks against collapse and local bulging and the associated calculations which are required are described in **E.5.2** to **E.5.7**.

E.5.2 *Allowance for shear stress*. When calculating the strength of plate elements in compression, allowance is made for the effects of any coexistent shear stresses by replacing the yield stress Y_s of the skin plating material by a modified yield stress $Y_{s.mod}$ determined as follows.

A representative factored shear stress $f_{q.rep}$ is obtained from the maximum factored shear stress in a panel $f'_{q.max}$ and the factored shear stress, $f'_{q.2}$ on the central line of the second stiffener from the edge of the panel at which $f'_{q.max}$ occurs.

$f'_{q.rep}$ is the greater of 2/3 $f'_{q.max}$ and $f'_{q.2}$

Where $f'_{q.rep} \leqslant 0.175\ Y_s$, the effect of shear is ignored and $Y_{s.mod} = Y_s$.

Where $f'_{q.rep} > 0.175\ Y_s$, then $Y_{s.mod} = 1.05\ \sqrt{[Y_s^2 - 3\,(f'_{q.rep})^2)}$

The nomogram on the left of Figure 21 enables $Y_{s.mod}$ to be determined directly from Y_s and $f'_{q.rep}$.

E.5.3 *Strength of plate elements*. The maximum compressive stress $F_{cp.max}$ that can be carried by the skin plating is obtained from Figure 21. The relevant curve is that which intersects the vertical (stress) axis at $Y_{s.mod}$, determined in **E.5.2**.

$F_{cp.max}$ is read from the vertical (stress) scale at the level at which the selected curve crosses the vertical line through the appropriate b/t ratio, where

b is taken as dimension b in Figure 17 for open stiffeners and is the greater of dimensions b_1 and b_2 in Figure 17 for closed stiffeners;

t is the skin plate thickness also shown in Figure 17.

E.5.4 *Collapse check 1*. The greatest factored compressive stress $f'_{cp.max}$ in the skin plating is not to exceed $F_{cf.mav}$, the maximum stress that can be carried, averaged over the whole width of the flange. $F_{cf.mav}$ is obtained by summing the maximum loads that can be carried by each plate and stiffener element in the panel and dividing by the total cross-sectional area. The maximum loads are obtained as follows.

a) *Skin plating*. For the skin plating, the maximum load $W_{p.max}$ is given by

$$W_{p.max} = F_{cp.max} W t$$

where

$F_{cp.max}$ is the maximum compressive stress that can be carried by the skin plating, obtained from **E.5.3**;

W and t are as shown in Figure 17.

b) *Open stiffeners*. For open stiffeners, the maximum load $W_{s.max}$ is given by

$W_{s.max} = A\ Y_s$

where

A is the cross-sectional area of the stiffener;

Y_s is the unmodified yield stress of the stiffener material.

c) *Closed stiffeners*. For closed stiffeners the maximum load $W_{s.max}$ is given by

$W_{s.max} = (2F_{cw.max}\ b_3 + Y_s b_4)t_O$

where

$F_{cw.max}$ is the maximum stress that can be carried by the web of the closed stiffener, obtained from Figure 21 using the unmodified yield stress Y_s of the stiffener material and the ratio b_3/t_O;

b_3, b_4 and t_O are the dimensions shown in Figure 18.

The area A of a closed-cross-section stiffener is given by

$A = (2b_3 + b_4)t_O$

d) *Stiffened compression flange*. For any stiffened compression flange, the maximum stress that can be carried, averaged over the whole compression flange, is given by

$$F_{cf.mav} = \frac{W_{p.max} + NW_{s.max}}{Wt + NA}$$

where

$W_{p.max}$ and $W_{s.max}$ are as given above;

W and t are the dimensions shown in Figure 17;

A is the cross-sectional area of one stiffener;

N is the number of stiffeners.

E.5.5 *Strength of a plate stiffener unit as a column*. A plate stiffener unit comprises one stiffener attached to a width of plating equal to the stiffener spacing. Its slenderness ratio S is given by the ratio L/r where the column length L is taken as the cross frame spacing a in Figure 17 and r is the radius of gyration of the plate stiffener unit about a centroidal axis parallel to the skin plating.

The maximum stress $F_{cu.max}$ that can be carried by a plate-stiffener unit acting as a column is obtained from Figure 22, selecting the curve that intersects the vertical (stress) axis at the value of $F_{cf.mav}$ calculated according to **E.5.4**. The value of $F_{cu.max}$ is read from the vertical scale at the point where the selected curve is crossed by the vertical line through the slenderness ratio s of the plate-stiffener unit.

E.5.6 *Collapse checks 2 and 3*. The representative factored compressive stress $f'_{cp.rep}$ in the skin plating within a panel is not to exceed $F_{cu.max}$ calculated in accordance with **E.5.5**, where $f'_{cp.rep}$ is obtained as follows.

$f'_{cp.max}$ is the greatest factored compressive stress in the skin plating within a panel, and $f'_{cp.opp}$ is the greatest factored compressive stress at the opposite end of the same panel. $f'_{cp.rep}$ is the factored compressive stress at a distance $0.4a$ from the end of the panel at which $f'_{cp.max}$ occurs; it is given by

$f'_{cp.rep} = 0.6f'_{cp.max} + 0.4f'_{cp.opp}$

The derivation of $f'_{cp.rep}$ is shown diagrammatically in Figure 20. In the simple method, check 3 is automatically satisfied by compliance with check 2.

E.5.7 *Check 4, against local bulging*, Where there are more than two stiffeners in a panel, a check is required to ensure that permanent bulging does not occur in the most highly stressed skin plate element under maximum working stress. This element will be in a position such as X in Figure 20 and is assumed to be of width b, where b is the greater of the widths b_1 and b_2 in Figure 17.

In carrying out this check, unfactored stresses (i.e. those determined from an elastic analysis of the structure and not multiplied by the factor K) are used. The unfactored compressive and shear stresses midway across the most highly stressed element at a distance of $b/2$ from the end carrying the higher compressive stress are denoted by $f_{cp.mid}$ and $f_{q.mid}$ respectively.

$f_{cp.mid}$ is not to exceed $F_{cp.mid}$,

where $F_{cp.mid}$ is the maximum compressive stress that can be carried to satisfy this check.

It is determined in the same way as $F_{cp.max}$ in **E.5.2** and **E.5.3** except that $f_{q.mid}$ is used in place of $f'_{q.rep}$ when determining $Y_{s.mod}$, the modified yield stress.

E.6 Detailed method for checks against collapse and local bulging

E.6.1 *General*. The detailed method, which in some cases will lead to more economical designs than the simple method, makes use of the design curves in Figure 24, Figure 25 and Figure 26.

The same data are given in Table 33 to Table 38 inclusive. The check against collapse and local bulging and the associated calculations required are as given in **E.6.2** to **E.6.9**.

E.6.2 *Allowance for shear stress*. As in the simple method, allowance has to be made for the effects of coexistent shear stresses when calculating the strength of plate elements in compression by replacing the yield stress Y_s of the skin plating material by a modified yield stress $Y_{s.mod}$.

Where the compressive stress is constant across the width of the stiffened panel, $Y_{s.mod}$ is determined by the method given in **E.5.2**.

Where the compressive stress varies across the width of the stiffened panel, the method given in **E.5.2** is also used but $f'_{q.mid}$, the factored shear stress midway across the most highly stressed skin element at a distance $b/2$ from the end carrying the higher stress (see **E.5.7**), is used in place of $f'_{q.rep}$ to determine $Y_{s.mod}$.

$Y_{s.mod}$ can be determined directly from Y_s and $f'_{q.mid}$ by means of the nomograms to the left of each of the graphs in Figure 24 (see **E.6.3**).

E.6.3 *Strength of plate elements*. Two classes of plate element are recognized in the detailed method as shown in Figure 23 and Figure 24.

Class P comprises "web" plate elements of closed-cross-section stiffeners and "lightly welded" skin plate elements. "Lightly welded" in this context is taken as having intermittent plate-stiffener welds. Class Q comprises skin plate elements bounded by continuous welds.

The maximum compressive stresses, $F_{cp.max}$, that can be carried by a skin plate element, and $F_{cw.max}$, the maximum compressive stress that can be carried by the web of a closed stiffener, are obtained from the appropriate graph of Figure 24 according to the class of plate element under consideration, selecting the curve that intersects the vertical (stress) axis at $Y_{s.mod}$ as determined from **E.6.2**. $F_{cp.max}$ is read off the vertical (stress) axis at the point where the selected curve crosses the vertical line through the appropriate b/t ratio, where b and t are as defined in **E.5.3**.

The curves in Figure 21 used in the simple method correspond to the class Q curves in Figure 24. Class P figures are given in Table 33 and class Q in Table 34.

E.6.4 *Collapse check 1*. Where the compressive stress is constant across the width of the stiffened panel, the maximum factored compressive stress $f'_{cp.max}$ in the skin plating is not to exceed $F_{cf.mav}$ calculated as in **E.5.4** using the values of $F_{cp.max}$ and $F_{cw.max}$ as appropriate, determined from **E.6.3**.

Where the compressive stress varies across the width of the stiffened panel, two checks are required.

a) The maximum factored compressive stress $f'_{cp.max}$ is not to exceed $Y_{s.mod}$ calculated as in **E.5.2** but substituting $f'_{q.max}$ for $f'_{q.rep}$.

b) The factored compressive stress, $f'_{cp.mid}$, which occurs in the same position as $f_{cp.mid}$ in **E.5.7**, is not to exceed $F_{cu.mav}$, the maximum stress that can be carried by a plate-stiffener unit comprising one stiffener plus a width of skin plating equal to b for stiffeners of open cross section and $b_1 + b_2$ for stiffeners of closed cross section.

$F_{cu.mav}$ is calculated by summing the maximum loads that can be carried by the plate and stiffener elements comprising one plate-stiffener unit and dividing by its cross-sectional area, using the values of $F_{cp.max}$ and $F_{cw.max}$ determined as in **E.6.3**.

E.6.5 *Slenderness ratio of a plate-stiffener unit*. The slenderness ratio S' of a plate-stiffener unit when using the detailed method is given by

$$S' = \frac{K_1 \cdot a_1}{r}$$

where

a_1 is the lesser value of a and $2w\sqrt{r/t}$, where r is the radius of gyration of the plate-stiffener unit as defined in **E.5.5** and a, t and w the dimensions shown in Figure 17;

K_1 is read from Figure 25 according to the values of r/t and a_1/w.

E.6.6 *Strength of a plate-stiffener unit as a column.* The strength of a plate-stiffener unit as a column is determined from Figure 26, which gives higher predicted strengths at high slenderness ratios than those obtained from the simple method.

The maximum compressive stress $F_{cu.max}$ that can be carried by the plate-stiffener unit as a column is the greater of the stresses $F_{cu.max}$ (a) and $F_{cu.max}$ (b) obtained from two column strength curves in Figure 26, selected as follows:

One curve (a) is taken from graph A1 or A2 and another (b) from graph B1 or B2, each according to the ratio $\bar{y}/r$, where $\bar{y}$ is the distance from the centre of the skin plate thickness to the centroid of the plate-stiffener unit, and r is the radius of gyration of the plate-stiffener unit about a centroidal axis parallel to the skin plating.

Curve (a)

Where $\bar{y}/r \leqslant 0.7$, $\alpha = 3.5$ and graph A1 is used.

Where $\bar{y}/r > 0.7$, $\alpha = 5.5$ and graph A2 is used.

The appropriate curve for determining $F_{cu.max}$ (a) is that which intersects the vertical (stress) axis of graph A1 or A2 at a stress equal to $F_{cf.mav}$ where the compressive stress is constant across the flange, and to $F_{cu.mav}$ where the compressive stress varies across the flange, $F_{cf.mav}$ and $F_{cu.mav}$ being calculated as in **E.6.4**. The value of $F_{cu.max}$ (a) is that at which the curve is crossed by the vertical line through the appropriate slenderness ratio S'.

Curve (b)

Where $\bar{y}/r \leqslant 0.7$, $\alpha = 1.0$ and graph B1 is used.

Where $\bar{y}/r > 0.7$, $\alpha = 2.0$ and graph B2 is used.

The appropriate curve for determining $F_{cu.max}$ (b) is that which intersects the vertical (stress) axis at the least of the stresses $F_{cp.max}$, $F_{cw.max}$, F_E and F_F

where

$F_{cp.max}$ and $F_{cw.max}$ are calculated in accordance with **E.6.3**.

$$F_E = Y_{s.mod} - C$$

$$F_F = \frac{3.63E}{(b/t)^2} - C$$

where

$Y_{s.mod}$ is the modified yield stress calculated in accordance with **E.6.2**;

C is 25 N/mm^2 for class P elements and 75 N/mm^2 for class Q elements as defined in **E.6.3**;

b is taken as dimension b in Figure 17 for open stiffeners and as the greater of dimensions b_1 and b_2 in Figure 17 for closed stiffeners;

t is the skin plate thickness;

E is Young's modulus for steel, taken as 205 000 N/mm^2.

The value of $F_{cu.max}$ (b) is that at which the curve is crossed by the vertical line through the appropriate slenderness ratio, $S' = L/r$ as defined in **E.6.5**.

In Figure 26 the column strength curves in graph B2 ($\alpha = 2.0$), graph A1 ($\alpha = 3.5$) and graph A2 ($\alpha = 5.5$) are identical with the column curves referred to in **5.1.3** except that in **5.1.3** the vertical (stress) axis indicates the yield stress of the steel under consideration. Also, values of the curves for $\alpha = 1.0$ are given in Table 35, for $\alpha = 2.0$ in Table 36, for $\alpha = 3.5$ in Table 37, and for $\alpha = 5.5$ in Table 38.

E.6.7 *Collapse check 2.* Where the compressive stress is constant across the width of the stiffened panel, the representative factored compressive stress, $f'_{cp.rep}$ as defined in **E.5.6** is not to exceed the maximum compressive stress for a plate stiffener unit $F_{cu.max}$ calculated in accordance with **E.6.6**.

Where the compressive stress varies across the width of the stiffened panel the same conditions as those given above apply, except that the representative factored compressive stress $f'_{cp.rep}$ is calculated on the longitudinal centre line of the most highly stressed plate element. This element would be in a position such as X in Figure 20.

E.6.8 *Collapse check 3*. Where the compressive stress is constant across the width of a panel, the modified representative factored compressive stress, $f'_{cp.rep}$ at the level of the centroid of the stiffened panel is not to exceed a stress F_{mod}.

where

$$f''_{cp.rep} = f'_{cp.rep} \left(1 - \frac{\bar{y}}{\bar{h}}\right);$$

$f'_{cp.rep}$ is as defined in **E.5.6**;

$\bar{y}$ is the distance defined in **E.6.6**;

h is the distance from the neutral axis of the complete box girder to the centre of the skin plate thickness;

F_{mod} is read from graph A1 (α = 3.5) of Figure 26, selecting the curve which intersects the vertical (stress) axis at the appropriate yield stress Y_s, and reading F_{mod} off the vertical (stress) scale where the selected curve crosses the vertical line through the appropriate slenderness ratio, $S' = L/r$ as defined in **E.6.5**.

Where the compressive stress varies across the width of a panel, the same check as that given above is carried out except that the value of $f'_{cp.rep}$ used in the formula for determining $f''_{cp.rep}$ is calculated as in **E.5.6** but on the basis of the compressive stresses that occur at the longitudinal centre-line of the more heavily stressed outside stiffener in the panel.

E.6.9 *Check 4, against local bulging*. This is carried out as in the simple method given in **E.5.7**, except that $F_{cp.mid}$ is determined on the basis of the unfactored stress $f_{q.mid}$ by the procedure set out in **E.6.2** and **E.6.3** for obtaining $F_{cp.max}$.

Table 33 — Plate strength in newtons per square millimetre: class P

b/t	$Y_{s.mod}$ (in N/mm²)																									
	200	**210**	**220**	**230**	**240**	**250**	**260**	**270**	**280**	**290**	**300**	**310**	**320**	**330**	**340**	**350**	**360**	**370**	**380**	**390**	**400**	**410**	**420**	**430**	**440**	**450**
20	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	430	440	450
22	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	430	440	450
24	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	429	438	446
26	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	349	358	367	376	385	394	402	411	420	428	437
28	200	210	220	230	240	250	260	270	280	290	299	308	316	325	334	342	351	360	368	376	385	393	401	410	418	426
30	200	210	220	230	240	249	258	267	275	284	293	301	310	318	326	335	343	351	359	367	375	383	391	398	406	413
32	200	210	218	227	236	244	253	261	270	278	286	294	302	310	318	326	334	342	349	357	364	371	378	385	392	399
34	197	206	214	223	231	239	247	255	263	271	279	287	295	302	309	317	324	331	338	345	351	358	365	371	377	383
36	193	201	210	218	226	234	241	249	257	264	272	279	286	293	300	306	313	319	326	332	338	344	349	355	360	366
38	189	197	205	213	220	228	235	242	249	256	263	270	276	283	289	295	301	307	312	318	323	328	333	338	343	347
40	185	192	200	207	214	221	228	235	242	248	254	260	266	272	278	283	288	293	298	303	307	312	316	320	324	328
42	180	188	195	201	208	215	221	227	233	239	245	250	256	261	266	270	275	279	283	287	291	295	299	304	308	313
44	176	182	189	195	202	208	214	219	225	230	235	240	244	249	253	257	261	265	269	273	278	283	287	292	297	301
46	171	177	183	189	195	200	206	211	216	220	225	229	233	237	241	244	249	254	259	263	268	272	277	281	286	290
48	165	171	177	182	188	193	197	202	206	211	214	218	222	226	231	236	241	245	250	254	259	263	268	272	276	281
50	160	166	171	176	180	185	189	193	197	201	205	210	214	219	224	228	233	237	242	246	250	255	259	263	268	272
52	155	160	164	169	173	177	181	185	189	194	198	203	208	212	217	221	226	230	234	239	243	247	252	256	260	264
54	149	154	158	162	166	169	174	179	183	188	192	197	201	206	210	215	219	223	228	232	236	241	245	249	253	257
56	144	148	152	155	159	164	169	173	178	183	187	191	196	200	205	209	213	218	222	226	230	234	239	243	247	251
58	138	142	145	150	155	160	164	169	173	178	182	186	191	195	199	204	208	212	216	220	225	229	233	237	241	245
60	133	137	142	146	151	155	160	164	169	173	178	182	186	190	195	199	203	207	211	215	219	223	227	231	235	239
62	129	133	138	143	147	152	156	160	165	169	173	178	182	186	190	194	199	203	207	211	215	218	222	226	230	234
64	126	130	135	139	144	148	152	157	161	165	170	174	178	182	186	190	194	198	202	206	210	214	218	221	225	229
66	123	127	132	136	140	145	149	153	158	162	166	170	174	178	182	186	190	194	198	202	206	209	213	217	220	224
68	120	124	129	133	138	142	146	150	155	159	163	167	171	175	179	183	187	190	194	198	202	205	209	212	216	219
70	117	122	126	131	135	139	143	147	152	156	160	164	168	172	175	179	183	187	190	194	198	201	205	208	212	215
72	115	119	124	128	132	136	141	145	149	153	157	161	165	168	172	176	180	183	187	190	194	197	201	204	207	211
74	113	117	121	126	130	134	138	142	146	150	154	158	162	165	169	173	176	180	183	187	190	194	197	200	204	207
76	111	115	119	124	128	132	136	140	144	148	151	155	159	162	166	170	173	177	180	183	187	190	193	197	200	203
78	109	113	117	121	126	130	134	137	141	145	149	152	156	160	163	167	170	174	177	180	183	187	190	193	196	199
80	107	111	115	120	124	127	131	135	139	143	146	150	153	157	160	164	167	170	174	177	180	183	186	189	192	195

Table 34 — Plate strength in newtons per square millimetre: class Q

b/t	$Y_{s,mod}$ (in N/mm^2)																								
	200	**210**	**220**	**230**	**240**	**250**	**260**	**270**	**280**	**290**	**300**	**310**	**320**	**330**	**340**	**350**	**360**	**370**	**380**	**390**	**400**	**410**	**420**	**430**	**440**	**450**
20	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	430	440	450
22	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	429	438	447
24	200	210	220	230	240	250	260	270	280	290	300	310	320	330	339	348	357	365	374	383	391	400	408	417	425	433
26	200	210	220	230	240	250	260	269	278	287	296	304	313	321	330	338	347	355	363	371	379	387	396	403	411	419
28	200	210	219	228	236	245	254	262	271	279	287	296	304	312	320	328	336	344	351	359	367	374	382	389	397	404
30	196	205	213	222	230	238	247	255	263	271	279	286	294	302	309	317	324	332	339	346	353	360	367	374	381	388
32	191	199	207	215	223	231	239	247	255	262	270	277	284	291	299	306	312	319	326	333	339	346	352	358	364	370
34	186	194	201	209	217	224	232	239	246	253	260	267	274	281	287	294	300	306	312	318	324	330	336	341	347	352
36	180	188	195	203	210	217	224	231	237	244	251	257	263	269	275	281	287	293	298	304	309	314	319	324	329	334
38	175	182	189	196	203	209	216	222	229	235	241	247	252	258	263	269	274	279	284	289	293	298	302	307	311	315
40	169	176	183	189	196	202	208	214	220	225	231	236	241	246	251	256	261	265	270	274	278	282	286	290	293	297
42	164	170	176	182	188	194	200	205	211	216	221	226	230	235	239	243	248	252	255	259	263	266	270	275	279	284
44	158	164	170	176	181	186	192	197	201	206	211	215	219	223	227	231	235	238	242	246	251	256	260	265	269	274
46	153	158	164	169	174	179	184	188	193	197	201	205	209	212	216	219	224	229	233	238	242	247	252	256	261	265
48	147	152	157	162	167	171	176	180	184	188	191	195	198	203	207	212	217	221	226	230	235	239	244	248	253	257
50	142	146	151	156	160	164	168	172	175	179	183	187	192	197	201	206	210	215	219	224	228	233	237	241	246	250
52	136	141	145	149	153	157	160	164	168	173	177	182	186	191	196	200	204	209	213	218	222	226	231	235	239	244
54	131	135	139	143	146	150	154	159	163	168	172	177	181	186	190	195	199	204	208	212	217	221	225	229	234	238
56	126	130	133	137	141	145	150	154	159	164	168	173	177	181	186	190	195	199	203	207	212	216	220	224	228	232
58	121	124	128	132	137	142	146	151	155	160	164	169	173	177	182	186	190	194	199	203	207	211	215	219	223	227
60	116	120	125	129	134	138	143	147	152	156	160	165	169	173	178	182	186	190	195	199	203	207	211	215	219	223
62	113	117	122	126	131	135	140	144	148	153	157	161	166	170	174	178	183	187	191	195	199	203	207	211	214	218
64	110	115	119	124	128	132	137	141	146	150	154	158	163	167	171	175	179	183	187	191	195	199	203	207	210	214
66	108	112	117	121	126	130	134	139	143	147	151	155	160	164	168	172	176	180	184	188	191	195	199	203	206	210
68	106	110	114	119	123	128	132	136	140	144	149	153	136	161	165	169	173	177	180	184	188	192	195	199	203	206
70	104	108	112	117	121	125	130	134	138	142	146	150	154	158	162	166	170	173	177	181	185	188	192	195	199	202
72	102	106	111	115	119	123	127	132	136	140	144	148	152	155	159	163	167	171	174	178	181	185	188	192	195	199
74	100	104	109	113	117	121	125	130	134	138	141	145	149	153	157	160	164	168	171	175	178	182	185	189	192	195
76	99	103	107	111	115	119	124	128	131	135	139	143	147	151	154	158	161	165	168	172	175	179	182	185	189	192
78	97	101	105	110	114	118	122	126	130	133	137	141	145	148	152	155	159	162	166	169	172	176	179	182	185	188
80	96	100	104	108	112	116	120	124	128	131	135	139	142	146	149	153	156	160	163	166	170	173	176	179	182	185

Table 35 — Column strength in newtons per square millimetre: α = 1.0

L/r	Stress $F_{cf.mav}$ or $F_{cu.mav}$ (in N/mm²)																														
	150	**160**	**170**	**180**	**190**	**200**	**210**	**220**	**230**	**240**	**250**	**260**	**270**	**280**	**290**	**300**	**310**	**320**	**330**	**340**	**350**	**360**	**370**	**380**	**390**	**400**	**410**	**420**	**430**	**440**	**450**
15	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	400	410	420	429	439	449
20	150	160	170	180	190	200	210	220	230	240	249	259	269	279	289	299	309	319	328	338	348	358	368	378	388	398	407	417	427	437	447
25	150	160	169	179	189	199	209	219	228	238	248	258	268	278	287	297	307	317	327	336	346	356	366	376	385	395	405	415	424	434	444
30	149	159	169	178	188	198	208	217	227	237	247	256	266	276	286	295	305	315	325	334	344	354	363	373	383	392	402	412	422	431	441
35	148	158	168	177	187	197	206	216	226	235	245	255	264	274	284	293	303	313	322	332	341	351	361	370	380	389	399	409	418	428	437
40	147	157	167	176	186	195	205	215	224	234	243	253	262	272	282	291	301	310	320	329	339	348	357	367	376	386	395	404	414	423	433
45	146	156	165	175	184	194	204	213	222	232	241	251	260	270	279	288	298	307	316	326	335	344	353	363	372	381	390	399	408	417	426
50	145	155	164	174	183	192	202	211	221	230	239	248	258	267	276	285	294	303	313	322	331	340	348	357	366	375	384	392	401	410	418
55	144	154	163	172	181	191	200	209	218	227	236	246	255	263	272	281	290	299	308	316	325	333	342	350	359	367	375	383	391	399	407
60	143	152	161	171	180	189	198	207	216	224	233	242	251	259	268	276	285	293	301	309	317	325	333	341	348	356	363	370	377	384	390
65	142	151	160	169	177	186	195	204	212	221	229	238	246	254	262	270	278	285	293	330	307	314	321	328	334	340	346	352	358	363	368
70	140	149	158	166	175	184	192	200	208	216	224	232	240	247	255	262	269	275	282	288	294	300	305	311	316	320	325	329	333	337	340
75	138	147	155	164	172	180	188	196	204	211	218	225	232	239	245	251	257	263	268	273	277	282	286	290	293	297	300	303	306	308	310
80	136	145	153	161	169	176	184	191	198	204	211	217	223	228	233	238	243	247	251	255	258	262	263	267	270	272	274	276	278	280	281
85	134	142	150	157	164	171	178	184	191	196	202	207	212	216	220	224	227	230	233	236	238	241	244	244	246	248	249	251	252	253	254
90	131	139	146	153	159	166	171	177	182	187	191	195	199	202	205	208	211	213	215	217	219	220	222	223	224	225	226	227	228	229	230
95	128	135	142	148	154	159	164	169	173	176	180	183	186	188	191	193	194	196	198	199	200	201	202	203	204	205	206	207	207	208	208
100	125	131	137	142	147	152	156	159	163	166	168	171	173	175	176	178	179	180	181	182	183	184	185	186	186	187	188	188	189	189	190
105	121	126	131	136	140	144	147	150	152	155	157	158	160	162	163	164	165	166	167	168	168	169	170	170	171	171	172	172	172	173	173
110	117	121	126	129	133	136	138	140	142	144	146	147	148	149	150	151	152	153	153	154	155	155	156	156	156	157	157	158	158	158	158
115	112	116	119	122	125	127	130	131	133	134	135	137	137	138	139	140	140	141	142	142	143	143	143	144	144	144	145	145	145	145	146
120	107	110	113	116	118	120	121	123	124	125	126	127	128	128	129	129	130	130	131	131	132	132	132	133	133	133	133	134	134	134	134

Table 36 — Column strength in newtons per square millimetre: α = 2.0

L/*r*	Stress $F_{cf.mav}$ or $F_{cu.mav}$ (in N/mm²)																														
	150	**160**	**170**	**180**	**190**	**200**	**210**	**220**	**230**	**240**	**250**	**260**	**270**	**280**	**290**	**300**	**310**	**320**	**330**	**340**	**350**	**360**	**370**	**380**	**390**	**400**	**410**	**420**	**430**	**440**	**450**
15	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	380	390	399	409	419	429	439	449
20	150	160	170	180	190	200	210	220	229	239	249	259	268	278	288	298	307	317	327	337	346	356	366	376	385	395	405	414	424	434	444
25	149	159	169	179	188	198	208	217	227	237	246	256	266	275	285	294	304	314	323	333	342	352	362	371	381	390	400	410	419	429	438
30	148	157	167	177	186	196	205	215	224	234	243	253	262	272	281	291	300	310	319	329	338	348	357	367	376	385	395	404	414	423	432
35	146	156	165	175	184	193	203	212	222	231	240	250	259	268	278	287	296	306	315	324	333	343	352	361	370	380	389	398	407	416	425
40	145	154	163	172	182	191	200	210	219	228	237	246	255	265	274	283	292	301	310	319	328	337	346	355	364	373	382	390	399	408	417
45	143	152	161	170	179	188	198	207	216	225	234	242	251	260	269	278	287	296	304	313	322	330	339	348	356	365	373	382	390	398	407
50	141	150	159	168	177	186	194	203	212	221	229	238	247	255	264	272	281	289	298	306	314	322	331	339	347	355	363	371	378	386	394
55	139	148	156	165	174	182	191	200	208	217	225	233	242	250	258	266	274	282	290	298	305	313	321	328	335	343	350	357	364	371	378
60	137	145	154	162	171	179	187	196	204	212	220	228	235	243	251	258	266	273	280	288	295	301	308	315	321	328	334	340	346	352	358
65	134	143	151	159	167	175	183	191	199	206	214	221	228	236	243	249	256	263	269	276	282	288	294	299	305	310	316	321	326	330	335
70	132	140	148	156	163	171	178	186	193	200	207	214	220	227	233	239	245	251	256	262	267	272	277	282	286	291	295	299	303	306	310
75	129	137	144	152	159	166	173	180	186	193	199	205	211	217	222	227	233	237	242	247	251	255	259	263	266	269	273	276	279	281	284
80	126	133	141	147	154	161	167	173	179	185	191	196	201	206	210	215	219	223	227	230	234	237	240	243	246	248	251	253	255	257	259
85	123	130	136	143	149	155	161	166	171	176	181	186	190	194	198	201	205	208	211	214	217	219	221	224	226	228	230	231	233	234	236
90	119	126	132	138	143	149	154	159	163	167	171	175	179	182	185	188	191	193	196	198	200	202	204	205	207	209	210	211	213	214	215
95	116	122	127	132	137	142	146	151	154	158	161	164	167	170	173	175	177	179	181	183	184	186	187	189	190	191	192	193	194	195	196
100	112	117	122	127	131	135	139	142	146	149	151	154	156	159	161	162	164	166	167	169	170	171	172	173	174	175	176	177	178	178	179
105	108	112	117	121	125	128	131	134	137	139	142	144	146	148	149	151	152	153	155	156	157	158	159	160	160	161	162	162	163	164	164
110	103	107	111	115	118	121	124	126	129	131	133	134	136	137	139	140	141	142	143	144	145	146	146	147	148	148	149	149	150	150	151
115	99	103	106	109	112	114	117	119	121	122	124	125	127	128	129	130	131	132	133	133	134	135	135	136	136	137	137	138	138	139	139
120	84	88	101	103	106	108	110	111	113	114	116	117	118	119	120	121	122	122	123	124	124	125	125	126	126	127	127	128	128	128	129

Table 37 — Column strength in newtons per square millimetre: α = 3.5

L/*r*	Stress $F_{cf.mav}$ or $F_{cu.mav}$ (in N/mm²)																													
	150	**160**	**170**	**180**	**190**	**200**	**210**	**220**	**230**	**240**	**250**	**260**	**270**	**280**	**290**	**300**	**310**	**320**	**330**	**340**	**350**	**360**	**370**	**380**	**390**	**400**	**410**	**420**	**430**	**440**	**450**
15	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	379	389	399	409	418	428	438	447
20	150	160	170	180	190	200	210	219	229	239	248	258	267	277	286	296	306	315	325	334	344	353	363	372	382	391	401	410	420	429	439
25	149	159	168	178	187	196	206	215	225	234	244	253	262	272	281	290	300	309	318	328	337	346	356	365	374	384	393	402	411	421	430
30	146	156	165	174	183	193	202	211	220	230	239	248	257	266	275	285	294	303	312	321	330	339	348	357	366	375	384	393	402	411	420
35	144	153	162	171	180	189	198	207	216	225	234	243	252	261	269	278	287	296	305	313	322	331	340	348	357	366	374	383	392	400	409
40	141	150	158	167	176	185	194	202	211	220	229	237	246	254	263	271	280	288	297	305	314	322	331	339	347	355	364	372	380	388	396
45	138	146	155	164	172	181	189	198	206	215	223	231	239	248	256	264	272	280	288	296	304	312	320	328	336	344	351	359	367	374	382
50	135	143	152	160	168	176	185	193	201	209	217	225	233	240	248	256	264	271	279	286	294	301	309	316	323	330	337	344	351	358	365
55	132	140	148	156	164	172	180	187	195	203	210	218	225	233	240	247	254	261	268	275	282	289	295	302	309	315	321	328	334	340	346
60	129	136	144	152	159	167	174	182	189	196	203	210	217	224	231	237	244	250	257	263	269	275	281	287	293	298	304	309	314	320	325
65	125	133	140	147	155	162	169	176	182	189	196	202	208	215	221	227	233	238	244	250	255	260	265	270	275	280	285	289	294	298	302
70	122	129	136	143	150	156	163	169	175	181	187	193	199	205	210	215	221	226	231	235	240	245	249	253	257	261	265	269	272	276	279
75	116	125	131	138	144	150	156	162	168	174	179	184	189	194	199	204	208	213	217	221	225	228	232	236	239	242	245	248	251	254	257
80	114	121	127	133	139	144	150	155	160	165	170	175	179	184	188	192	196	199	203	206	209	213	215	218	221	224	226	229	231	233	235
85	111	116	122	128	133	138	143	148	152	157	161	165	169	173	176	180	183	186	189	192	194	197	200	202	204	206	208	210	212	214	216
90	107	112	117	122	127	132	136	140	145	148	152	156	159	162	165	168	171	173	176	178	180	182	184	186	188	190	192	193	195	196	197
95	103	108	112	117	121	125	129	133	137	140	143	146	149	152	154	157	159	161	163	165	167	169	170	172	174	175	176	178	179	180	181
100	98	103	107	112	115	119	123	126	129	132	135	137	140	142	144	146	148	150	152	153	155	156	158	159	160	161	162	163	164	165	166
105	94	99	102	106	110	113	116	119	121	124	126	129	131	133	134	136	138	139	141	142	143	145	146	147	148	149	150	151	152	152	153
110	90	94	98	101	104	107	110	112	114	117	119	121	122	124	126	127	128	130	131	132	133	134	135	136	137	138	139	139	140	141	141
115	86	90	93	96	99	101	103	106	108	110	111	113	114	116	117	119	120	121	122	123	124	125	125	126	127	128	128	129	130	130	131
120	82	85	88	91	93	96	98	100	101	103	104	106	107	108	110	111	112	113	114	114	115	116	117	117	118	119	119	120	120	121	121

Table 38 — Column strength in newtons per square millimetre: α = 5.5

L/r	Stress $F_{cf.mav}$ or $F_{cu.mav}$ (in N/mm²)																													
	150	**160**	**170**	**180**	**190**	**200**	**210**	**220**	**230**	**240**	**250**	**260**	**270**	**280**	**290**	**300**	**310**	**320**	**330**	**340**	**350**	**360**	**370**	**380**	**390**	**400**	**410**	**420**	**430**	**440**	**450**
15	150	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310	320	330	340	350	360	370	379	389	398	408	417	427	436	446
20	150	160	170	180	190	200	210	219	228	238	247	256	266	275	285	294	303	312	322	331	340	350	359	368	377	387	396	405	414	424	433
25	148	158	167	176	185	194	204	213	222	231	240	249	258	267	276	285	294	303	312	321	330	339	348	357	366	375	384	393	402	410	419
30	144	153	162	171	180	189	198	206	215	224	233	242	250	259	268	276	285	294	302	311	320	328	337	345	354	363	371	380	388	396	405
35	140	149	157	166	175	183	192	200	209	217	226	234	242	251	259	267	276	284	292	300	309	317	325	333	341	349	357	365	374	382	389
40	136	144	153	161	169	177	186	194	202	210	218	226	234	242	250	258	266	274	282	289	297	305	312	320	328	335	343	350	358	365	373
45	132	140	148	156	164	172	179	187	195	203	210	218	226	233	241	248	256	263	270	278	285	292	299	306	313	320	327	334	341	348	355
50	128	135	143	151	158	166	173	181	188	195	202	210	217	224	231	238	245	252	258	265	272	278	285	291	298	304	310	317	323	329	335
55	124	131	138	145	153	160	167	174	181	188	194	201	208	214	221	227	233	240	246	252	258	264	270	276	281	287	293	298	304	309	314
60	119	126	133	140	147	154	160	167	173	180	186	192	198	204	210	216	222	227	233	238	244	249	254	260	265	270	274	279	284	289	293
65	115	122	128	135	141	147	154	160	166	171	177	183	188	194	199	205	210	215	220	225	229	234	239	243	247	252	256	260	264	268	272
70	111	117	123	129	135	141	147	152	158	163	169	174	179	184	188	193	198	202	207	211	215	219	223	227	230	234	238	241	244	248	251
75	107	113	118	124	129	135	140	145	150	155	160	164	169	173	178	182	186	190	193	197	201	204	208	211	214	217	220	223	226	228	231
80	103	108	113	119	124	129	133	138	143	147	151	155	159	163	167	171	174	177	181	184	187	190	193	196	198	201	203	206	208	210	213
85	98	104	108	113	118	122	127	131	135	139	143	146	150	153	157	160	163	166	168	171	174	176	179	181	183	186	188	190	192	194	195
90	94	99	104	108	112	116	120	124	128	131	135	138	141	144	147	149	152	155	157	159	162	164	166	168	170	172	173	175	177	179	180
95	90	95	99	103	107	110	114	117	121	124	127	130	132	135	137	140	142	144	146	148	150	152	154	156	157	159	160	162	163	164	166
100	86	90	94	98	101	105	108	111	114	117	119	122	124	126	129	131	133	134	136	138	140	141	143	144	146	147	148	149	151	152	153
105	83	76	90	93	96	99	102	105	107	110	112	114	116	118	120	122	124	125	127	129	130	131	133	134	135	136	137	138	139	140	141
110	79	82	85	88	91	94	97	99	101	103	105	107	119	111	113	114	116	117	119	120	121	122	123	124	125	126	127	128	129	130	131
115	75	78	81	84	87	89	91	93	95	97	99	101	103	104	106	107	108	110	111	112	113	114	116	116	117	118	118	119	120	121	121
120	72	75	77	80	82	84	86	88	90	92	93	95	96	98	99	100	101	103	104	105	106	106	107	108	109	110	110	111	112	112	113

Table 39 — Effective length factor K_1

a_1/w	r/t						
	2	**3**	**4**	**5**	**6**	**8**	**10**
0.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.5	0.996	0.998	0.999	0.999	1.000	1.000	1.000
1.0	0.980	0.991	0.995	0.997	0.998	0.999	0.999
1.5	0.940	0.972	0.984	0.990	0.993	0.996	0.997
2.0	0.867	0.934	0.961	0.975	0.982	0.990	0.994
2.5	0.765	0.872	0.992	0.948	0.963	0.979	0.986
3.0	0.651	0.789	0.884	0.906	0.932	0.960	0.974
3.5	0.542	0.696	0.791	0.850	0.889	0.933	0.955
4.0	0.449	0.602	0.709	0.782	0.833	0.895	0.929
4.5	0.737	0.516	0.626	0.709	0.770	0.849	0.895
5.0	0.312	0.442	0.549	0.634	0.702	0.796	0.854
5.5	0.263	0.379	0.479	0.584	0.634	0.738	0.807
6.0	0.225	0.327	0.419	0.499	0.569	0.678	0.755

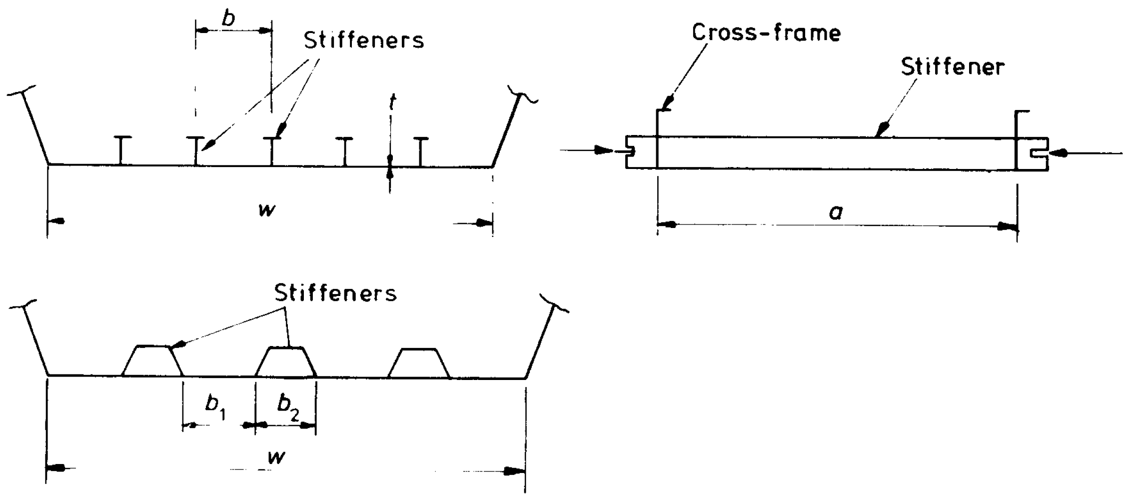
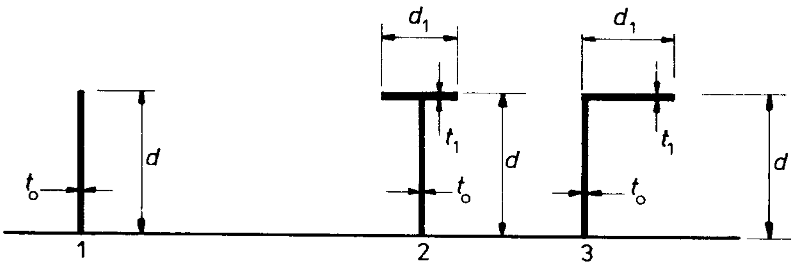
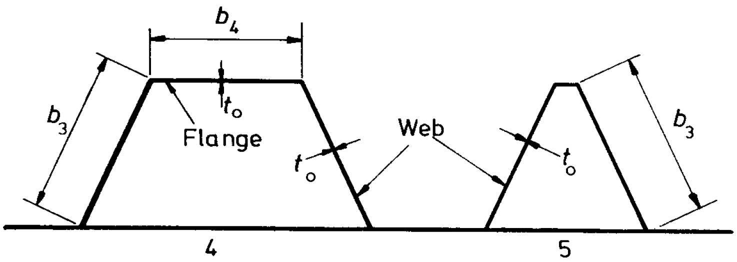


Figure 17 — Box girder compression flange: typical construction



For rolled angles and tees, dimensions d, d_1, t_o and t_1 are those given in the relevant British Standard or section tables.

(a) Stiffeners of open cross section



For stiffeners of closed cross section, dimensions b_3 and b_4 are measured between the centre lines of the plate elements.

(b) Stiffeners of closed cross section

Figure 18 — Types of stiffener

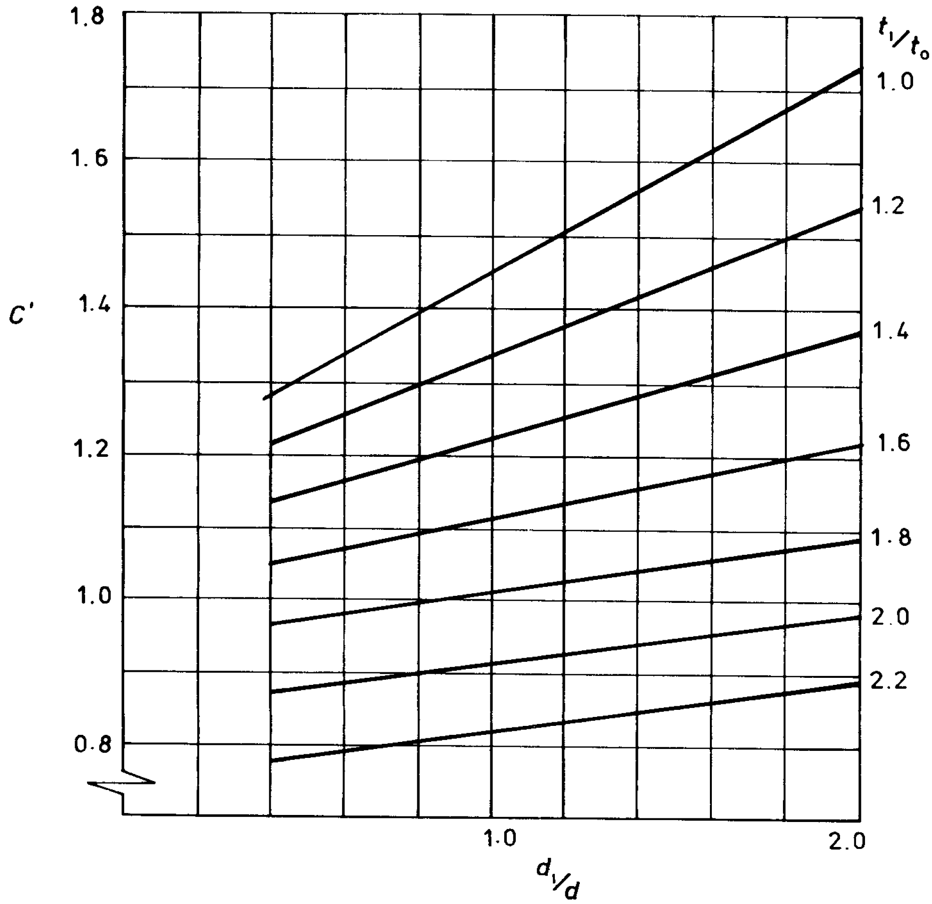


Figure 19 — Factor *C′* for tees to BS 4-1

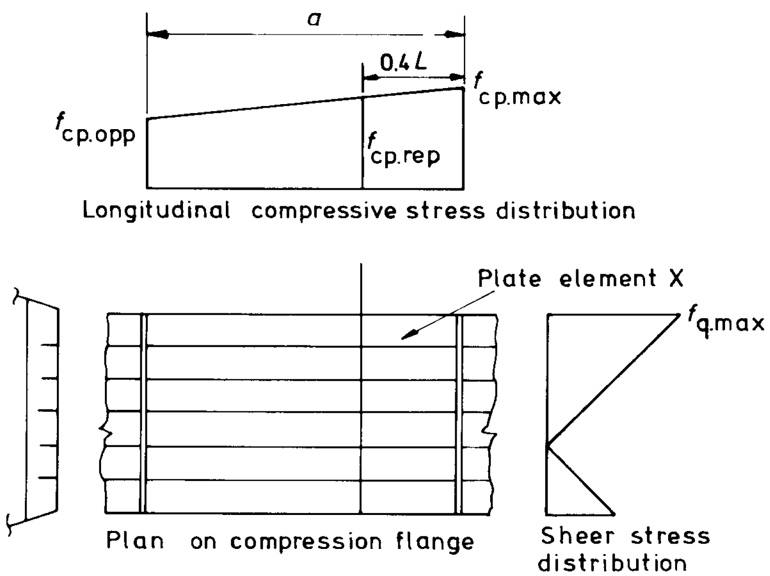
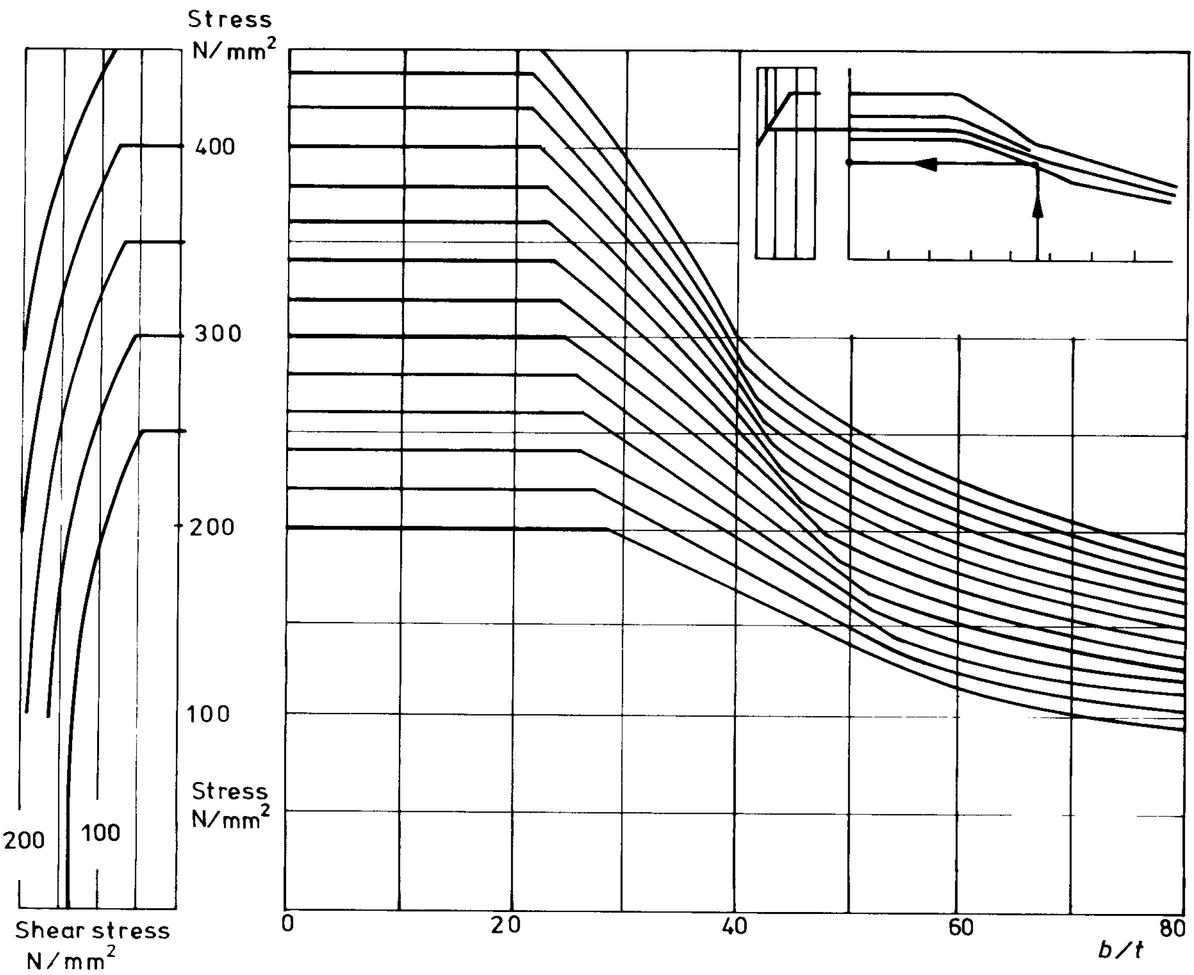


Figure 20 — Variation of stresses in a stiffened compression panel



The modified yield stress, used in cases where longitudinal compressive stress and shear stress are both present, is determined by the nomogram to the left of the graph. The curve corresponding to the yield stress of the steel is selected; it is that crossing the vertical (stress) axis at the yield stress. The modified yield is read from the vertical (stress) axis at the level at which the selected curve intersects the vertical line drawn through the appropriate shear stress, whence it may be transferred directly onto the graph on the right.

Figure 21 — Plate strength curves for simple method

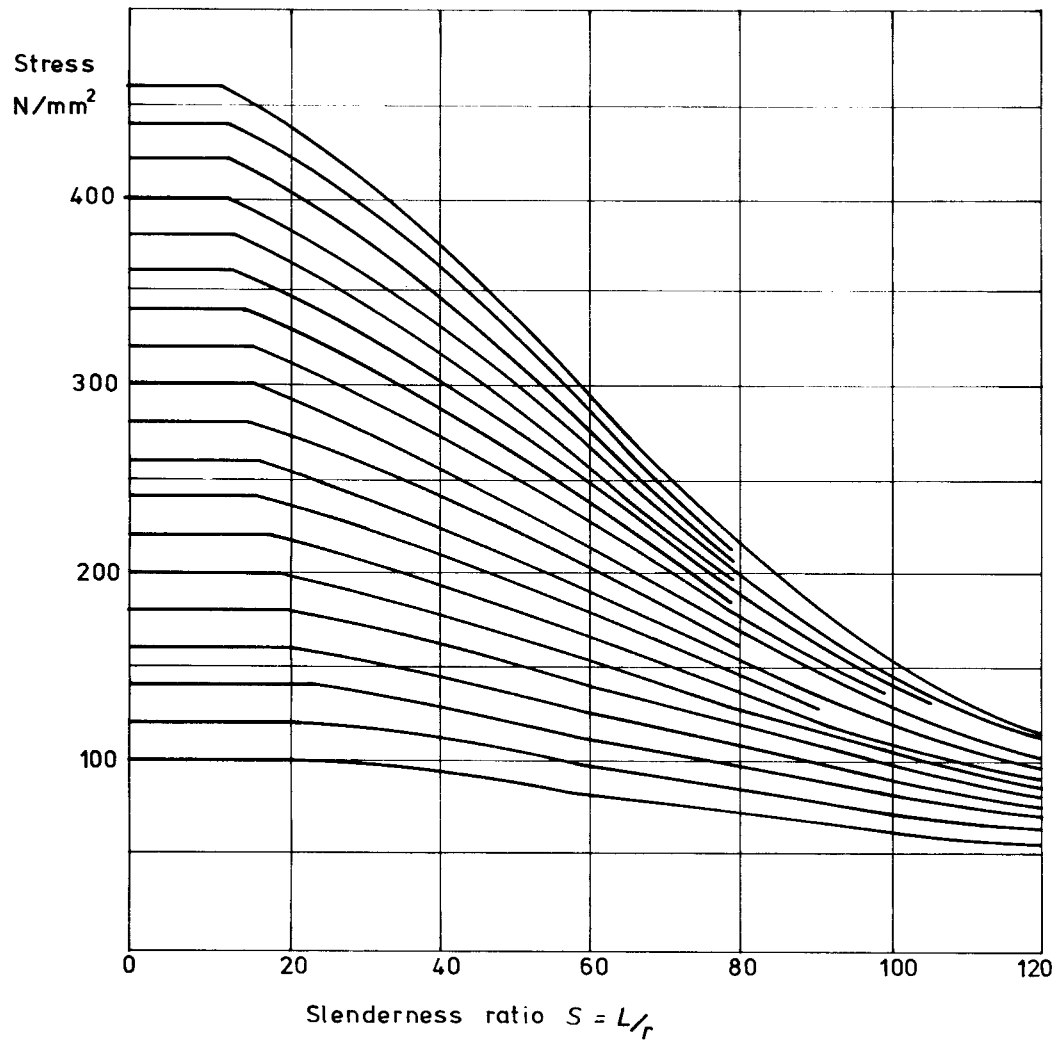


Figure 22 — Column strength curves for simple method

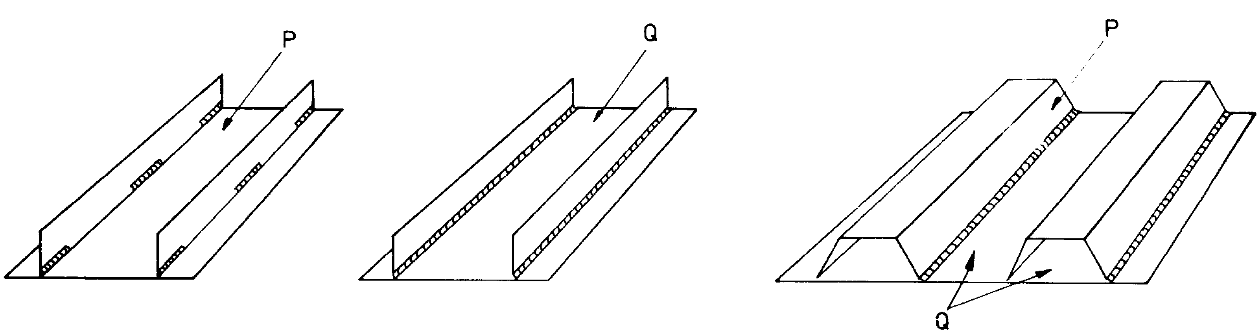
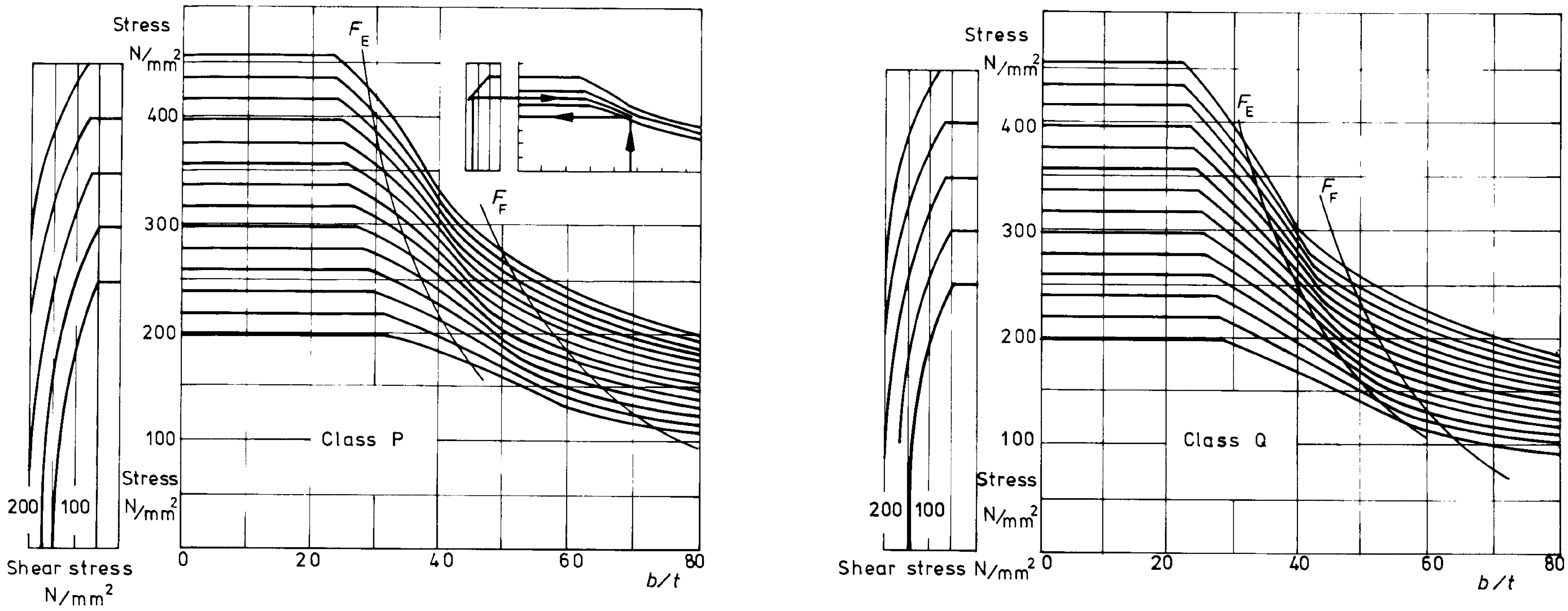


Figure 23 — Classification of plate elements for detailed method



For each class of plate F_E depends only on the modified yield stress $Y_{s.mod}$ and F_F only on the *b*/*t* ratio. The use of the nomograph to the left of each graph to determine $Y_{s.mod}$ is described in Figure 21.

Figure 24 — Plate strength curves for detailed method

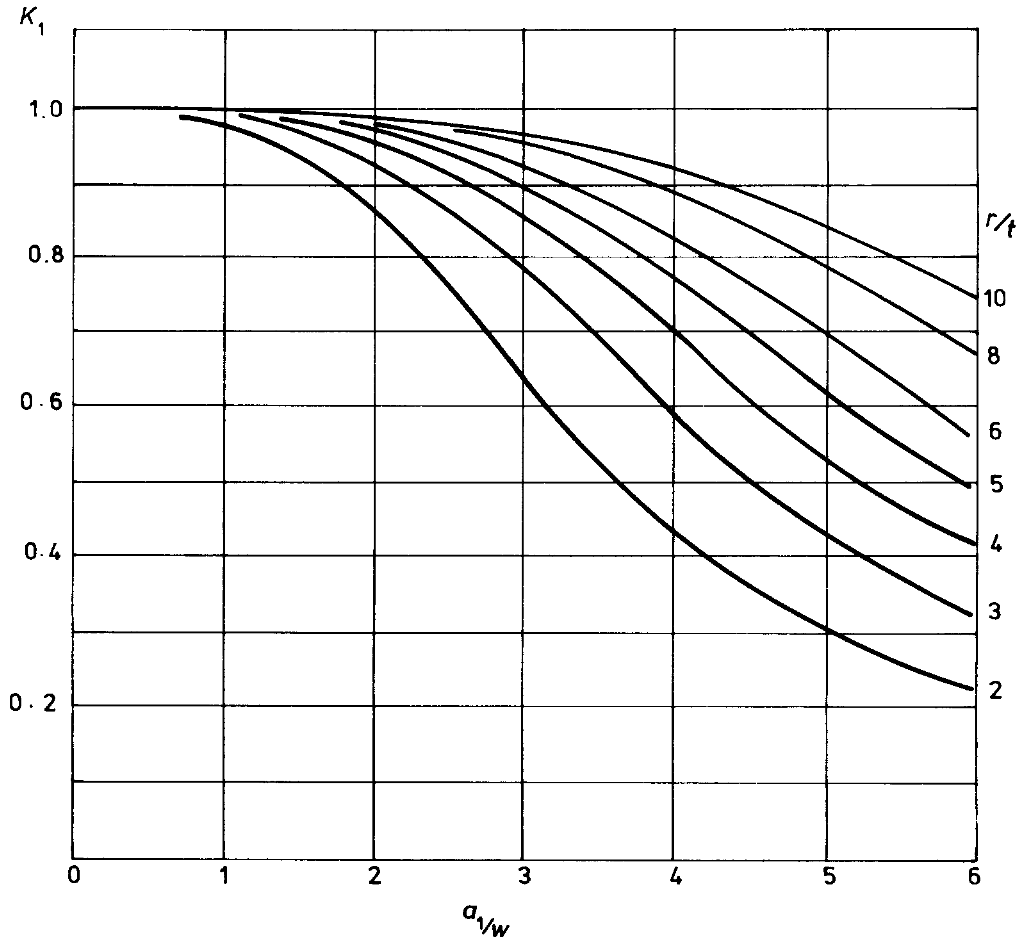


Figure 25 — Factor K_1 for detailed method

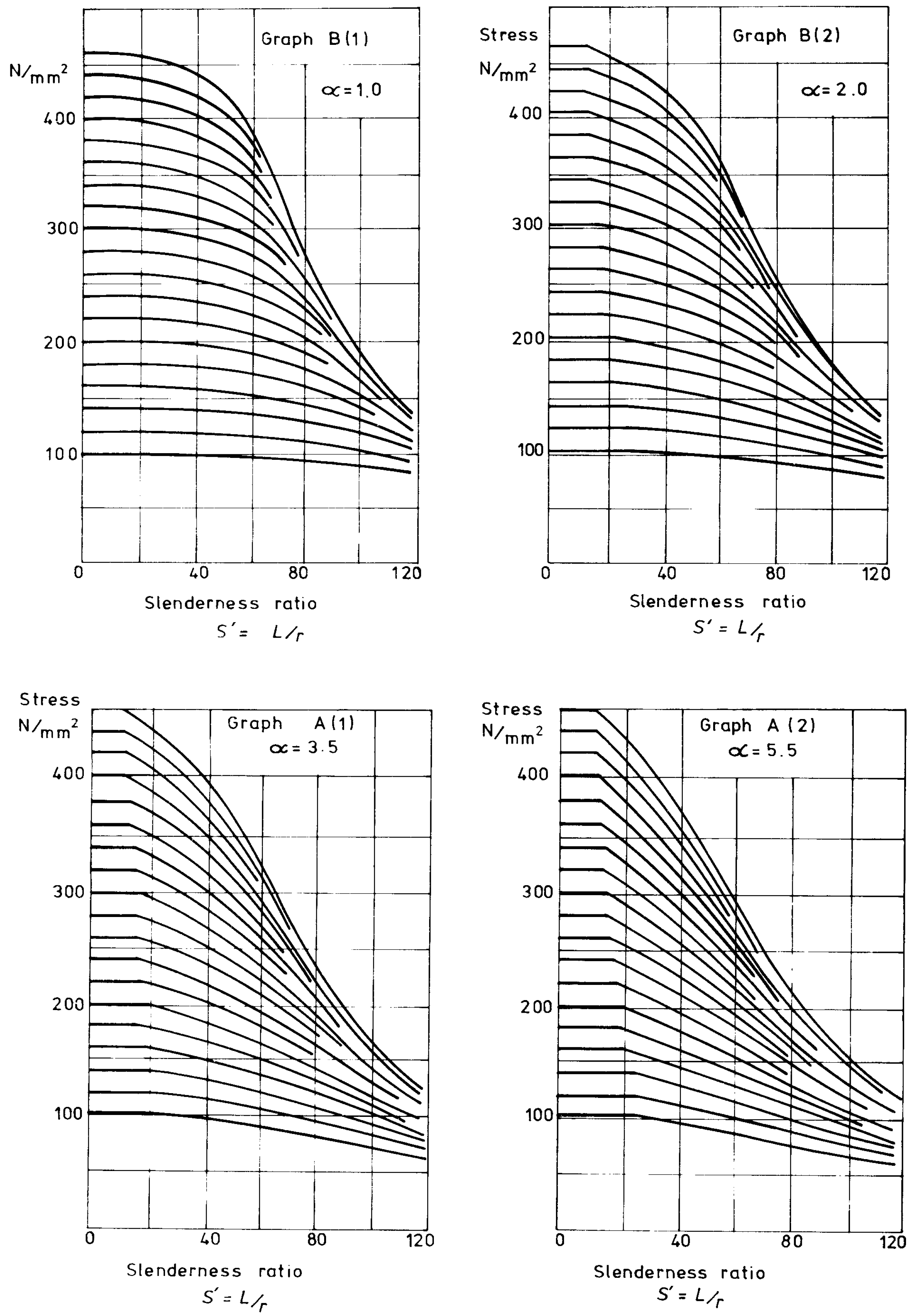


Figure 26 — Column strength curves for detailed method

Appendix F Fatigue strength

F.1 Structural components and details

During crane operation, structural components and details are subjected to stress cycles that vary in magnitude and number. This appendix sets out a method of assessing fatigue life according to the conditions of stress to which components and details are subjected as a result of operation of the crane throughout its useful life. The use of this method of making fatigue checks requires realistic assessments of the intensity of service which the component or detail by undergoes.

A measure of the cumulative service intensity for a structural component or detail is given by the service intensity factor, K_8, which is derived from a detailed knowledge of the load and motion spectra of the crane and its mechanisms. If significant aspects of these spectra are omitted, inadequately proportioned structures will result.

A component or detail should be so designed that

$K_8 \leqslant 1.0$

where $K_8 = \sum \frac{n_i}{N_i} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \ldots + \frac{n_n}{N_n}$.

and n_i is the number of stress cycles in the component or detail associated with a specific combination of f_{max} and ratio f_{min}/f_{max}.

N_i is the permissible number of stress cycles to which the component or detail may be subjected at that specific combination of f_{max} and f_{min}/f_{max}.

N_i is found from Table 34 as follows.

The value of P_{ft} or P_{fc}, as appropriate, which is equal to the calculated value of f_{max}, is found in Table 34 for the class of constructional detail and ratio f_{min}/f_{max} under consideration. N_i is the corresponding number of stress cycles from the head of the column in which that value of P_{ft} or P_{fc} appears. For intermediate values of P_{ft} or P_{fc}, the permissible number of cycles, N_i, can be obtained by interpolation from the straight line that results when P_f and N are plotted on a log/log basis.

F.2 Number of stress cycles for structural components

In many cases the number of stress cycles for a structural component will be related to the number of operating cycles of the crane as a whole. However, structural members may be subjected to several cycles of stress during one operating cycle or the number of stress cycles may be independent of the main operation and therefore different from the corresponding number of operating cycles.

F.3 Spectra of load and stress

F.3.1 *Load spectrum*. The load spectrum defines the state of loading of a crane throughout its working life and indicates the number of times it lifts loads of various magnitudes in relation to its maximum capacity.

F.3.2 *Stress spectrum*. The spectrum of stress induced in a structural component is derived from the load spectrum and motion data, taking into account all conditions of loading, except wind, to which the component may be subjected during operation.

Publications referred to

BS 4, *Structural steel sections.*
BS 4-1, *Specification for hot-rolled sections.*
BS 153, *Steel girder bridges.*
BS 449, *The use of structural steel in building.*
BS 449-2, *Metric units.*
BS 639, *Covered electrodes for the manual metal-arc welding of carbon and carbon manganese steel.*
BS 709, *Methods of testing fusion welded joints and weld metal in steel.*
BS 2573, *Specification for permissible stresses in cranes and design rules*[1].
BS 2573-2, *Mechanisms.*
BS 2853, *The design and testing of steel overhead runway beams.*
BS 4360, *Weldable structural steels.*
BS 4395, *High strength friction grip bolts and associated nuts and washers for structural engineering.*
BS 4395-1, *General grade.*
BS 4395-2, *Higher grade bolts and nuts and general grade washers.*
BS 4395-3, *Higher grade bolts (waisted shank), nuts and general grade washers.*
BS 4604, *The use of high strength friction grip bolts in structural steelwork. Metric series.*
BS 4604-1, *General grade.*
BS 4604-2, *Higher grade (parallel shank).*
BS 4604-3, *Higher grade (waisted shank).*
BS 4848, *Hot-rolled structural steel sections.*
BS 4848-4, *Equal and unequal angles.*
BS 5135, *Metal-arc welding of carbon and carbon manganese steels.*
ISO 4301, *Lifting appliances — Classification.*

[1] Referred to in the foreword only.

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