BRITISH STANDARD

BS 5400 : Part 10 : 1980

Incorporating Amendment No. 1

Steel, concrete and composite bridges —

Part 10: Code of practice for fatigue

ICS 93.040

Summary of pages

The following table identifies the current issue of each page. Issue 1 indicates that a page has been introduced for the first time by amendment. Subsequent issue numbers indicate an updated page. Vertical sidelining on replacement pages indicates the most recent changes (amendment, addition, deletion).

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Foreword

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

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

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.

1. Scope

1.1 General. This Part of this British Standard recommends methods for the fatigue assessment of parts of bridges which are subject to repeated fluctuations of stress.

1.2 Loading. Standard load spectra are given for both highway and railway bridges.

1.3 Assessment procedures. The following alternative methods of fatigue assessment are described for both highway and railway bridges:

(a) simplified methods that are applicable to parts of bridges with classified details and which are subjected to standard loadings;

(b) methods using first principles that can be applied in all circumstances.

1.4 Other sources of fatigue damage. The following topics are not specifically covered by this Part of this British Standard but their effects on the fatigue life of a structure may need to be considered:

(a) aerodynamically induced oscillations;

(b) fluctuations of stress in parts of a structure immersed in water, which are due to wave action and/or eddy induced vibrations;

(c) reduction of fatigue life in a corrosive atmosphere (corrosion fatigue).

1.5 Limitations

1.5.1 ***Steel decks.*** Highway loading is included in this Part and is applicable to the fatigue design of welded orthotropic steel decks. However, the stress analysis and classification of details in such a deck is very complex and is beyond the scope of this Part of this British Standard.

1.5.2 ***Reinforcement.*** The fatigue assessment of certain details associated with reinforcing bars is included in this Part but interim criteria for unwelded bars are given in Part 4.

NOTE. These criteria are at present under review and revised criteria may be issued later as an amendment.

1.5.3 ***Shear connectors.*** The fatigue assessment of shear connectors between concrete slabs and steel girders acting compositely in flexure is covered in this Part, but the assessment of the effects of local wheel loads on shear connectors between concrete slabs and steel plates is beyond the scope of this Part of this British Standard. This effect may, however, be ignored if the concrete slab alone is designed for the entire local loading.

2. References

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

3. Definitions and symbols

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

3.1.1 ***fatigue.*** The damage, by gradual cracking of a structural part, caused by repeated applications of a stress which is insufficient to induce failure by a single application.

3.1.2 ***loading event.*** The approach, passage and departure of either one train or, for short lengths, a bogie or axle, over a railway bridge or one vehicle over a highway bridge.

3.1.3 ***load spectrum.*** A tabulation showing the relative frequencies of loading events of different intensities experienced by the structure.

NOTE. A convenient mode of expressing a load spectrum is to denote each load intensity as a proportion (K_w) of a standard load and the number of occurrences of each load as a proportion (K_n) of the total number of loading events.

3.1.4 ***standard load spectrum.*** The load spectrum that has been adopted in this Part of this British Standard, derived from the analysis of actual traffic on typical roads or rail routes.

3.1.5 ***stress history.*** A record showing how the stress at a point varies during a loading event.

3.1.6 ***combined stress history.*** A stress history resulting from two consecutive loading events, i.e. a single loading event in one lane followed by a single loading event in another lane.

3.1.7 ***stress cycle (or cycle of stress).*** A pattern of variation of stress at a point which is in the form of two opposing half-waves, or, if this does not exist, a single half-wave.

3.1.8 ***stress range (or range of stress)*** (σ_r). Either

(a) in a plate or element, the greatest algebraic difference between the principal stresses occurring on principal planes not more than 45° apart in any one stress cycle; or

(b) in a weld, the algebraic or vector difference between the greatest and least vector sum of stresses in any one stress cycle.

3.1.9 ***stress spectrum.*** A tabulation of the numbers of occurrences of all the stress ranges of different magnitudes during a loading event.

3.1.10 ***design spectrum.*** A tabulation of the numbers of occurrences of all the stress ranges caused by all the loading events in the load spectrum, which is to be used in fatigue assessment of the structural part.

3.1.11 ***detail class.*** A rating given to a detail which indicates its level of fatigue resistance. It is denoted by the following: A, B, C, D, E, F, F2, G, S, or W.

NOTE. The maximum permitted class is the highest recommended class, that can be achieved with the highest workmanship specified in Part 6 (see table 17). The minimum required class to be specified for fabrication purposes relates to the lowest $\sigma_r - N$ curve in figure 14, which results in a life exceeding the design life.

3.1.12 ***$\sigma_r - N$ relationship or $\sigma_r - N$ curve.*** The quantitative relationship between σ_r and N for a detail which is derived from test data on a probability basis.

3.1.13 ***design $\sigma_r - N$ curve.*** The $\sigma_r - N$ relationship adopted in this Part of this British Standard for design on the basis of 2.3 % probability of failure.

3.1.14 ***design life.*** The period in which a bridge is required to perform safely with an acceptable probability that it will not require repair.

3.1.15 ***standard design life.*** 120 years, adopted in this Part of this British Standard.

3.1.16 ***Miner's summation.*** A cumulative damage summation based on the rule devised by Palmgren and Miner.

3.2 Symbols. The symbols in this Part of this British Standard are as follows.

A	Net area of cross section
A_1	Effective weld throat area for the particular type of connector
d	Number of standard deviations below the mean line $\sigma_r - N$ curve
d_{120}	Life time damage factor (Miner's summation for 120 million repetitions of a stress range σ_v in a highway bridge)
F	Design stress parameter for bolts
K_0	Parameter defining the mean line $\sigma_r - N$ relationship
K_2	Parameter defining the $\sigma_r - N$ relationship for two standard deviations below the mean line
K_B	Value of ratio $\sigma_{v1B}/\sigma_{v1A}$ (highway bridges)
K_F	Miner's summation adjustment factor (highway bridges)
K_n	Proportion factor for occurrences of vehicles of a specified gross weight (320 K_w kN) in any one lane of a highway bridge
K_{RC}	Fatigue stress concentration factor for re-entrant corners
K_{UA}	Fatigue stress concentration factor for unreinforced apertures
K_w	Ratio of actual : standard gross weights of vehicles, trains, bogies or axles in a load spectrum
$k_1 - k_6$	Coefficients in the simplified assessment procedure for a railway bridge
L	Base length of that portion of the point load influence line which contains the greatest ordinate (see figure 12) measured in the direction of travel
M, M_1	Applied bending moments
m	Inverse slope of log σ_r/log N curve
N	Number of repetitions to failure of stress range σ_r
N_1, N_2	Number of repetitions to failure of stress ranges σ_{r1}, σ_{r2} ... etc., corresponding to n_1, n_2 ... etc., repetitions of applied cycles
n_1, n_2 .. etc.	Number of applied repetitions of damaging stress ranges σ_{r1}, σ_{r2} ... etc., in a design spectrum
n_c	Number of vehicles (in millions per year) traversing any lane of a highway bridge
$\bar{n}_c$	Effective value of n_c
n_R	Total number of live load cycles (in millions) for each load proportion K_w in a railway bridge
P, P_1	Applied axial forces
P_u	Basic static strength of the stud
z	Elastic modulus of section
γ_f	Partial safety factor for load (the product $\gamma_{f1} \cdot \gamma_{f2} \cdot \gamma_{f3}$, see Part 1)
γ_{fL}	Product of $\gamma_{f1} \cdot \gamma_{f2}$
γ_m	Partial safety factor for strength
Δ	Reciprocal of the antilog of the standard deviation of log N
$\Sigma \frac{n}{N}$	Miner's summation
σ_B	Stress on the core area of a bolt, determined on the basis of the minor diameter
σ_H	Limiting stress range under loading from the standard fatigue vehicle on a highway bridge
σ_N	Stress on net section
σ_o	Constant amplitude non-propagating stress range (σ_r at $N = 10^7$)
σ_p	Algebraic value of stress in a stress history
$\sigma_{p\,max}$, $\sigma_{p\,min}$	Maximum and minimum values of σ_p from all stress histories produced by standard loading
σ_r	Range of stress (stress range) in any one cycle
σ_{r1}, σ_{r2} ... etc	Individual stress ranges (σ_r) in a design spectrum
$\sigma_{R\,max}$	($\sigma_{p\,max} - \sigma_{p\,min}$) for a railway bridge
σ_{R1}, σ_{R2} ... etc	Stress ranges (in descending order of magnitude) in a stress history of a railway bridge under unit uniformly distributed loading
σ_T	Limiting stress range under standard railway loading
σ_u	Nominal ultimate tensile strength, to be taken as $1.1\sigma_Y$ unless otherwise specified
σ_v	Value of σ_r under loading from the standard fatigue vehicle (highway bridges)
$\sigma_{v\,max}$	($\sigma_{p\,max} - \sigma_{p\,min}$) for a highway bridge
σ_{v1}, σ_{v2} ... etc	Values of σ_v (in descending order of magnitude) in any one stress history for one lane of a highway bridge
σ_{v1A}	The largest value of σ_{v1} from all stress histories (highway bridges)
σ_{v1B}	The second largest value of σ_{v1} from all stress histories (highway bridges)
σ_x, σ_y	Coexistent orthogonal direct stresses
σ_Y	Nominal yield strength
τ	Shear stress coexistent with σ_x and σ_y

4. General guidance

4.1 Design life. The design life is that period in which a bridge is required to perform safely with an acceptable probability that it will not require repair (see appendix A).

The standard design life for the purposes of this Part of this British Standard should be taken as 120 years unless otherwise specified.

4.2 Classification and workmanship. Each structural steel detail is classified in accordance with table 17 (see **5.1.2**). This shows the maximum permitted class for different types of structural detail. The class denoted in table 17 determines the design of $\sigma_r - N$ curve in figure 14 that may be safely used with the highest workmanship standards specified in Part 6 for the detail under consideration.

In **5.3.1** is defined the information to be provided to the fabricator, to ensure that the appropriate quality standards for Part 6 are invoked.

4.3 Stresses. Stresses should generally be calculated in accordance with Part 1 of this British Standard but clause **6** of this Part supplements the information given in Part 1.

4.4 Methods of assessment. All methods of assessment described in this Part of this British Standard are based on the Palmgren-Miner rule for damage calculation (see clause **11**). The basic methods given respectively in **8.4** and **9.3** for highway and railway bridges may be used at all times. The simplified procedures given in **8.2** and **8.3** for highway bridges and in **9.2** for railway bridges may be used when the conditions stipulated in **8.2.1**, **8.3.1** and **9.2.1** are satisfied.

4.5 Factors influencing fatigue behaviour. The best fatigue behaviour of joints is achieved by ensuring that the structure is so detailed that the elements may deform in their intended ways without introducing secondary deformations and stresses due to local restraints. Stresses may also be reduced, and hence fatigue life increased, by increased thickness of parent metal or weld metal.

The best joint performance is achieved by avoiding joint eccentricity and welds near free edges and by other controls over the quality of the joints. Performance is adversely affected by concentrations of stress at holes, openings and re-entrant corners. Guidance in these aspects is given in table 17 and appendix H. The effect of residual stresses is taken into account in the classification tables.

5. Classification of details

5.1. Classification

5.1.1 ***General***

5.1.1.1 For the purpose of fatigue assessment, each part of a constructional detail subject to fluctuating stress should, where possible, have a particular class designated in accordance with the criteria given in table 17. Otherwise the detail may be dealt with in accordance with **5.2.**

5.1.1.2 The classification of each part of a detail depends upon the following:

(a) the direction of the fluctuating stress relative to the detail;

NOTE. Propagation of cracks takes place in a direction perpendicular to the direction of stress.

(b) the location of possible crack initiation at the detail;

(c) the geometrical arrangement and proportions of the detail;

(d) the methods and standards of manufacture and inspection.

5.1.1.3 In welded details there are several locations at which potential fatigue cracks may initiate; these are as follows:

(a) in the parent metal of either part joined adjacent to;

(1) the end of the weld,

(2) a weld toe,

(3) a change of direction of the weld,

(b) in the throat of the weld.

In the case of members or elements connected at their ends by fillet welds or partial penetration butt welds and flanges with shear connectors, the crack initiation may occur either in the parent metals or in the weld throat: both possibilities should be checked by taking into account the appropriate classification and stress range. For other details, the classifications given in table 17 cover crack initiation at any possible location in the detail. Notes on the potential modes of failure for each detail are given in appendix H.

5.1.2 ***Classification of details in table 17***

5.1.2.1 Table 17 is divided into three parts which correspond to the three basic types into which details may be classified. These are as follows:

(a) type 1, non-welded details, table 17 (a);

(b) type 2, welded details on surface, table 17 (b);

(c) type 3, welded details at end connections of members, table 17 (c).

5.1.2.2 Each classified detail is illustrated and given a type number. Table 17 also gives various associated criteria and the diagrams illustrate the geometrical features and potential crack locations which determine the class of each detail and are intended to assist with initial selection of the appropriate type number. (For important features that change significantly from one type to another see the footnote to table 17.)

5.1.2.3 A detail should only be designated a particular classification if it complies in every respect with the tabulated criteria appropriate to its type number.

5.1.2.4 Class A is generally inappropriate for bridge work and the special inspection standards relevant to classes B and C cannot normally be achieved in the vicinity of welds in bridge work. (For these and other classifications that should be used only when special workmanship is specified see the footnote to table 17.)

5.1.2.5 The classifications of table 17 are valid for the qualities of steel products and welds which meet the requirements of Part 6, except where otherwise noted. For certain details the maximum permitted class depends on acceptance criteria given in Part 6.

5.2 Unclassified details

5.2.1 ***General.*** Details not fully covered in table 17 should be treated as class G, or class W for load carrying weld metal, unless a superior resistance to fatigue is proved by special tests. Such tests should be sufficiently extensive to allow the design $\sigma_r - N$ curve to be determined in the manner used for the standard classes (see appendix A).

5.2.2 ***Post-welding treatments.*** Where the classification of table 17 does not give adequate fatigue resistance, the performance of weld details may be improved by post-welding treatments such as controlled machining, grinding or peening. When this is required the detail should be classified by tests as given in **5.2.1.**

5.3 Workmanship and inspection

5.3.1 ***General.*** Where the classification of a detail is dependent upon particular manufacturing or inspection requirements, which are not generally specified in Part 6 of this British Standard, the necessary standards of workmanship and inspection should be indicated on the relevant drawings.

All areas of the structure where welded details classified as class F or higher are necessary should be shown on the drawings together with the minimum required class and an arrow indicating the direction of stress fluctuation (see figure 1). For inspection purposes this information should be incorporated onto the fabricator's shop instructions.

Note that a joint may have more than one class requirement if it experiences significant stress fluctuations in two or more directions.

NOTE. The level of manufacturing quality can affect the fatigue life of all stuctural details. The manufacturing quality determines the degree to which discontinuities, that may act as stress raisers, may be introduced during the fabrication process. Such discontinuities can act as fatigue points, which may reduce the fatigue life to an unacceptable level for the detail under consideration. Details with a high permitted class are more seriously affected by such discontinuities because of the restrictions already placed by table 17 on stress raisers inherent in the form of the detail itself.

In order to dertermine which level of quality and inspection is required in accordance with Part 6, the minimum required class has to be derived. If a class higher than F2 is required this has to be specified on the drawings, otherwise the required fatigue life may not be achieved. If a class higher than F2 is specified, but not required, an uneconomical fabrication would result.

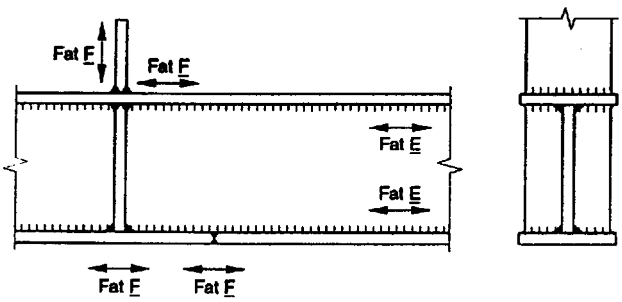


Figure 1. Method of indicating minimum class requirements on drawings

5.3.2 ***Detrimental effects.*** The following occurrences can result in a detail exhibiting a lower performance than its classification would indicate:

(a) weld spatter;

(b) accidental arc strikes;

(c) unauthorized attachments;

(d) corrosion pitting.

5.4 Steel decks. The classifications given in table 17 should not be applied to welded joints in orthotropic steel decks of highway bridges; complex stress patterns usually occur in such situations and specialist advice should be sought for identifying the stress range and joint classification.

6. Stress calculations

6.1 General

6.1.1 ***Stress range for welded details.*** The stress range in a plate or element to be used for fatigue assessment is the greatest algebraic difference between principal stresses occurring on principal planes not more than 45° apart in any one stress cycle.

6.1.2 ***Stress range for welds.*** The stress range in a weld is the algebraic or vector difference between the greatest and least vector sum of stresses in any one stress cycle.

6.1.3 ***Effective stress range for non-welded details***
For non-welded details, where the stress range is entirely in the compression zone, the effects of fatigue loading may be ignored.

For non-welded details subject to stress reversals, the stress range should be determined as in **6.1.1**. The effective stress range to be used in the fatigue assessment should be obtained by adding 60 % of the range from zero stress to maximum compressive stress to that part of the range from zero stress to maximum tensile stress.

6.1.4 ***Calculation of stresses***

6.1.4.1 Stresses should be calculated in accordance with Part 1 of this British Standard using elastic theory and taking account of all axial, bending and shearing stresses occurring under the design loadings given in clause **7**. No redistribution of loads or stresses, such as is allowed for checking static strength at ultimate limit state or for plastic design procedures, should be made. For stresses in composite beams the modulus of elasticity of the concrete should be derived from the short term stress/strain relationship (see Part 4). The stresses so calculated should be used with a material factor $\gamma_m = 1$.

6.1.4.2 The bending stresses in various parts of a steel orthotropic bridge deck may be significantly reduced as the result of composite action with the road surfacing. However, this effect should only be taken into account on the evidence of special tests or specialist advice.

6.1.5 ***Effects to be included.*** Where appropriate, the effects of the following should be included in stress calculations:

(a) shear lag, restrained torsion and distortion, transverse stresses and flange curvature (see Parts 3 and 5);

(b) effective width of steel plates (see Part 3);

(c) cracking of concrete in composite elements (see Part 5);

(d) stresses in triangulated skeletal structures due to load applications away from joints, member eccentricities at joints and rigidity of joints (see Part 3).

6.1.6 ***Effects to be ignored.*** The effects of the following need not be included in stress calculations:

(a) residual stresses;

(b) eccentricities necessarily arising in a standard detail;

(c) stress concentrations, except as required by table 17;

(d) plate buckling.

6.2 Stress in parent metal

6.2.1 The reference stress for fatigue assessment should be the principal stress in the parent metal adjacent to the potential crack location, as shown in figure 2a. Unless otherwise noted in table 17, the stress should be based on the net section. Where indicated in table 17, stress concentrations should be taken into account either by special analysis or by the factors given in figure 22 (see also H.1.2).

6.2.2 Shear stress may be neglected where it is numerically less than 15 % of a coexistent direct stress.

6.2.3 The peak and trough values of principal stress should be those on principal planes which are not more than 45° apart. This will be achieved if either

(a) $\sigma_x - \sigma_y$ is at least double the corresponding shear stress τ at both peak and trough, or

(b) the signs of $\sigma_x - \sigma_y$ and τ both reverse or both remain the same at the peak and the trough,

where

σ_x, σ_y and τ are the coexistent values with appropriate signs of the two orthogonal direct stresses and the shear stresses at the point under consideration.

In either (a) or (b), provided that $\sigma_x^2 \geqslant \sigma_y^2$ at both peak and trough, the required stress range will be the algebraic difference between the numerically greater peak principal stress and the numerically greater trough principal stress.

6.3 Stress in weld throats other than those attaching shear connectors. The reference stress for fatigue of a weld throat should be the vector sum of the shear stresses in the weld metal based on an effective throat dimension as defined in Part 3, and on the assumption that none of the load is carried in bearing between parent metals. This is illustrated in figure 2b. When calculating the stress range, the vector difference of the greatest and the least vector sum stress may be used instead of the algebraic difference.

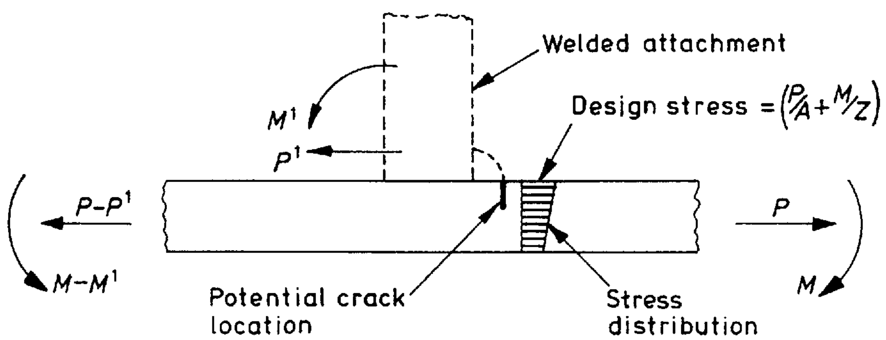


Figure 2a. Reference stress in parent metal

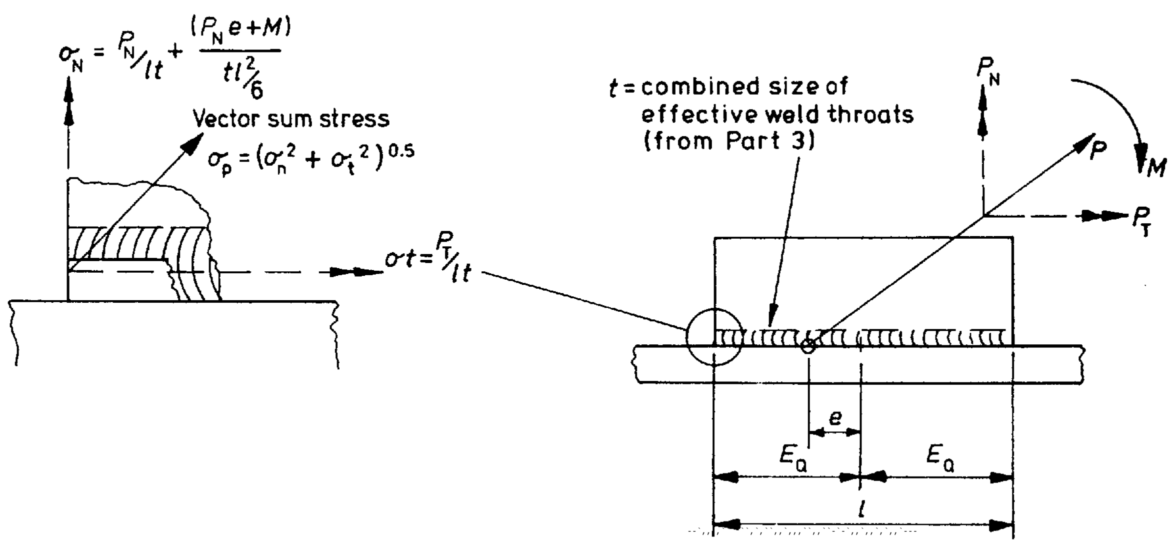


Figure 2b. Reference stress in weld throat

6.4 Stresses in welds attaching shear connectors

6.4.1 ***General.*** For shear connectors in accordance with the dimensional recommendations of Part 5, the design stresses for fatigue in the weld metal should be calculated in accordance with **6.4.2** and **6.4.3.** Where the dimensions of the shear connectors and/or the concrete haunches are not in accordance with Part 5, the fatigue strength should be determined in accordance with appendix G of this Part.

6.4.2 ***Stud connectors.*** The stresses in the weld metal attaching stud shear connectors should be calculated from the following expression :

stress in weld =

$$\frac{\text{longitudinal shear load on stud}}{\text{appropriate nominal static strength (from Part 5)}} \times 425\ \text{N/mm}^2$$

6.4.3 ***Channel and bar connectors***

6.4.3.1 The stresses in the weld metal attaching channel and bar shear connectors should be calculated from the effective throat area of weld, transverse to the shear flow, when the concrete is of normal density and from 0.85 × throat area when lightweight concrete is used. For the purposes of this clause the throat area should be based on a weld leg length which is the least of the dimensions tabulated below.

Channel connector	Bar connector
Actual leg length	0.6 × actual leg length
Thickness of channel web	0.125 × (height + breadth of bar)
Half the thickness of beam flange	half the thickness of beam flange

6.4.3.2 It may assist calculation to note that in normal density concrete, where the thickness of the beam flange is at least twice the actual weld leg length and the weld dimensions comply with Part 5, the effective weld areas are :

50 × 40 bar connectors × 200 mm long, 1697 mm²
25 × 25 bar connectors × 200 mm long, 1018 mm²
127 and 102 channel connectors × 150 mm long, 1272 mm²
76 channel connectors × 150 mm long, 1081 mm²

6.5 Axial stress in bolts. The design stress for fatigue in bolts complying with the requirements of BS 4395 and bolts to dimensional tolerances complying with the requirements of BS 3692 should be calculated from the following expression :

$$\text{stress in bolt} = \frac{F}{\sigma_u} \times \sigma_B$$

where

$F = 1.7\ \text{kN/mm}^2$ for threads of nominal diameter up to 25 mm

or

$F = 2.1\ \text{kN/mm}^2$ for threads of nominal diameter over 25 mm

σ_B is the stress range on the core area of the bolt determined on the basis of the minor diameter

σ_u is the nominal ultimate tensile strength of the bolt material in kN/mm²

When subjected to fluctuating stresses, black bolts complying with the requirements of BS 4190 may only be used if they are faced under the head and turned on shank in accordance with the requirements of BS 4190.

7. Loadings for fatigue assessment

7.1 Design loadings. Highway and railway design loadings appropriate for bridges in the UK are given in **7.2** and **7.3** respectively.

The load factors γ_{fL} and γ_{f3} should be taken as equalling 1.0 (see Part 2).

7.2 Highway loading

7.2.1 ***General.*** In determining the maximum range of fluctuating stress, generally, only the vertical effects of vehicular live load as given in clause 7 should be considered, modified where appropriate to allow for impact as given in **7.2.4.** In welded members the dead load stress need not be considered. In unwelded members the dead load stress will have to be considered in determining the effective stress range when compression stresses occur (see **6.1.3**).

Centrifugal effects need only be considered for substructures (see **7.2.5**).

7.2.2 ***Standard loading***

7.2.2.1 *Standard load spectrum.* The standard load spectrum should be as shown in table 11 which gives the weight intensities and relative frequencies of commercial traffic on typical trunk roads in the UK. The minimum weight taken for a commercial vehicle is 30 kN. All vehicles less than 30 kN are ignored when considering fatigue.

7.2.2.2 *Standard fatigue vehicle.* The standard fatigue vehicle is a device used to represent the effects of the standard load spectrum ; for highway bridges this is a single vehicle with a weight of 320 kN. It consists of four standard axles with the dimensions as shown in figures 3 and 4.

NOTE. See appendix C for the derivation of the standard fatigue vehicle.

7.2.2.3 *Number of vehicles.* The numbers of commercial vehicles that are assumed to travel along each lane of a bridge per year should be taken from table 1. If for any reason vehicle numbers other than these are adopted, suitable adjustments may be made to the fatigue analysis in accordance with **8.2.3** or **8.3.2.1** (e).

7.2.3 ***Application of loading***

7.2.3.1 *Demarcation of lanes.* For the purposes of this Part of this British Standard the lanes should be the actual traffic lanes marked on the carriageway. They should be designated in accordance with figure 5 and the loading should be applied to the slow and the adjacent lanes only. Where a crawler lane is provided it should be treated as an additional slow lane.

7.2.3.2 *Path of vehicles.* The mean centre line of travel of all vehicles in any lane should be along a path parallel to, and within 300 mm of, the centre line of the lane as shown in figure 6. The transverse position of the centre line of the vehicle should be selected so as to cause the maximum stress range in the detail being considered. In some instances it may be found that the use of multiple paths results in significantly less calculated damage and guidance on this is given in **C.1.4.**

7.2.3.3 *Standard loading.* The passage of one standard fatigue vehicle along the entire length of one lane should be taken as one loading event.

7.2.3.4 *Non-standard load spectrum.* If a load spectrum is used, which differs in any way from the standard load spectrum, the passage of each vehicle forming the load spectrum should be considered to provide a separate loading event.

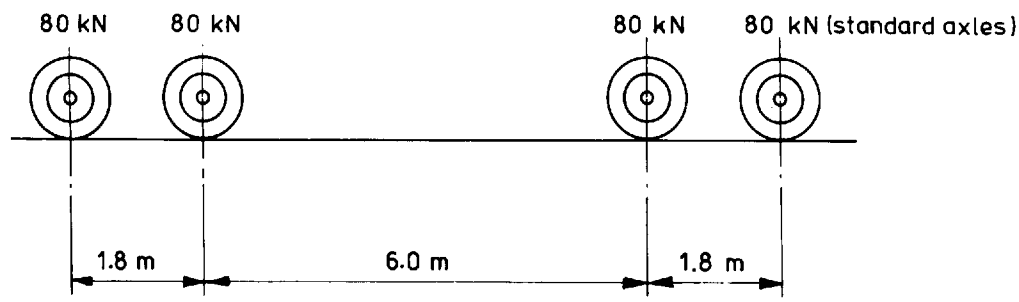


Figure 3. Axle arrangement of standard fatigue vehicle

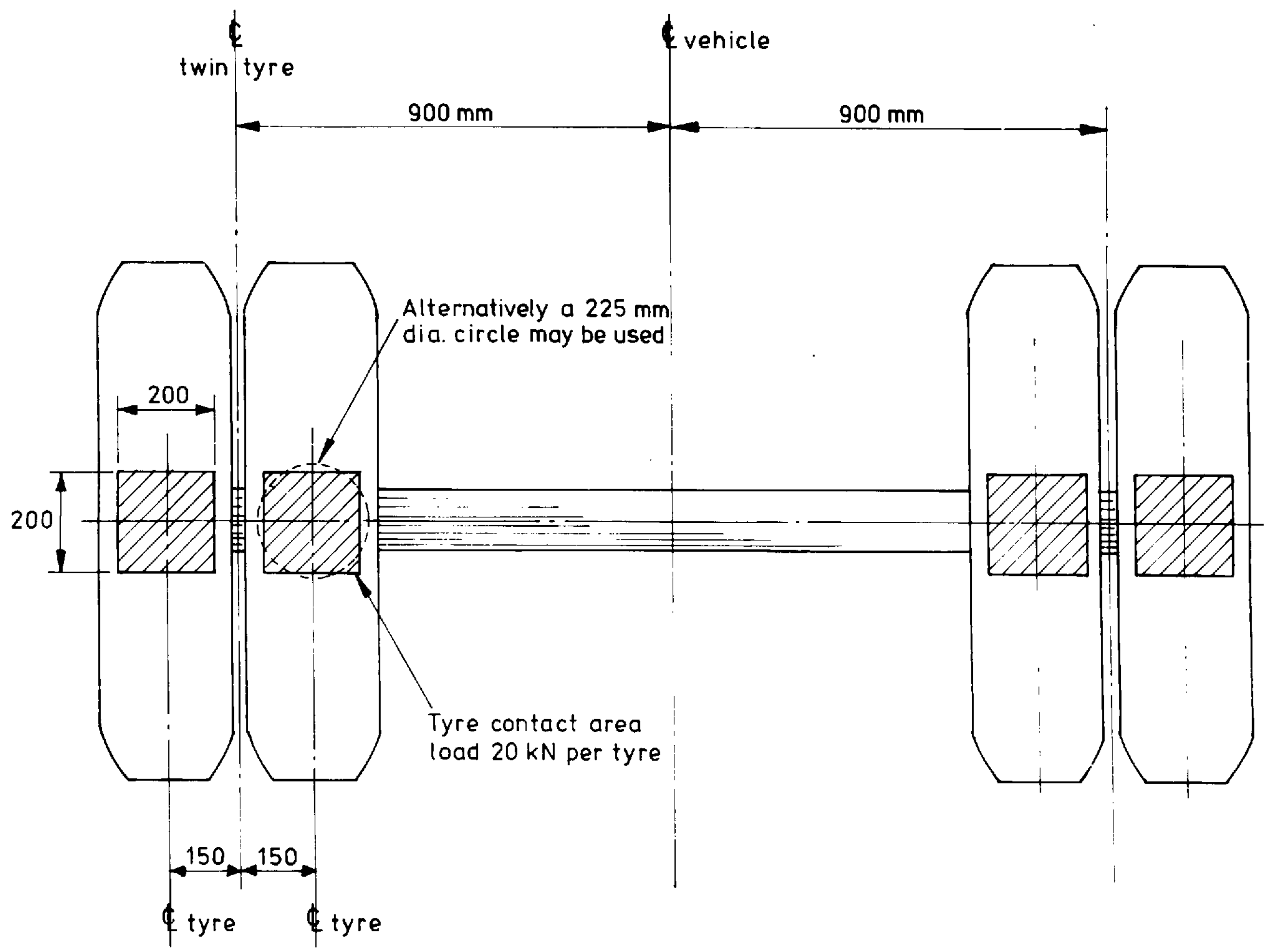


Figure 4. Plan of standard axle

Table 1. Annual flow of commercial vehicles ($n_c \times 10^6$)

Category of road			Number of millions of vehicles per lane, per year (n_c)	
Type	Carriageway layout	Number of lanes per carriageway	Each slow lane	Each adjacent lane
Motorway	Dual	3	2.0	1.5
Motorway All purpose All purpose Slip road	Dual Dual Dual Single	2 3 2 2	1.5	1.0
All purpose All purpose Slip road	Single Single (10 m*) Single	3 2 1	1.0	Not applicable
All purpose	Single (7.3 m*)	2	0.5	Not applicable

*The number of vehicles in each lane of a single carriageway between 7.3 m and 10 m wide should be obtained by linear interpolation.

7.2.3.5 *Method of loading.* Only one vehicle should be assumed to be on the structure at any one time and each lane should be traversed separately. The effects of combinations of vehicles are allowed for in clause **8**.

7.2.4 ***Allowance for impact.*** Where a discontinuity occurs in the road surface, e.g. at an expansion joint, the static stress at every point affected by a wheel, at or within 5 m of the discontinuity, should be increased by magnifying the relevant influence line, as shown in figure 7.

7.2.5 ***Centrifugal forces.*** The effects of any centrifugal force associated with the fatigue loading defined in **7.2.2** need only be considered for substructures; the force should be taken as acting at and parallel to the road surface. The magnitude of the force should be calculated at the appropriate design speed of the particular road, for the individual vehicles of the standard load spectrum shown in table 11 as follows:

the centrifugal force per axle $= \frac{W_v{}^2}{127_r}$ (kN)

where

W is the axle load of the vehicle (kN)
v is the design speed of the road (km/h)
r is the radius of curvature at the particular lane on which the vehicles are assumed to travel (m)

The force assumed for any vehicle should not exceed

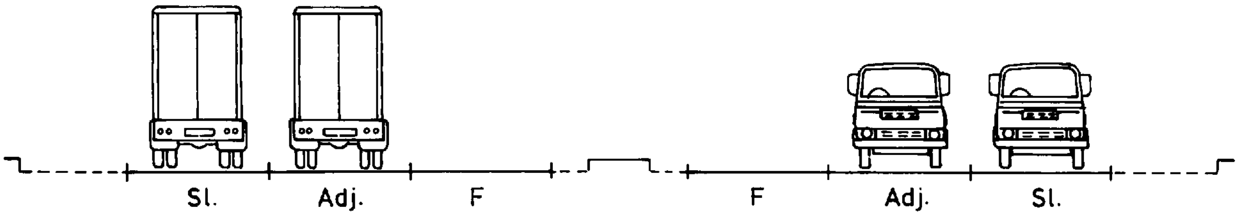
$$\frac{30\,000}{r + 150}\ \text{kN}$$

7.3 Railway loading

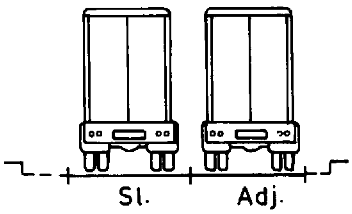
7.3.1 ***General.*** The loads to be considered should be the appropriate combination of the nominal live load, impact, lurching and centrifugal force, as specified in Part 2 of this British Standard.

In welded members the dead load stress need not be considered. In unwelded members the dead load stress will have to be considered in determining the effective stress range when compression stresses occur (see **6.1.3**).

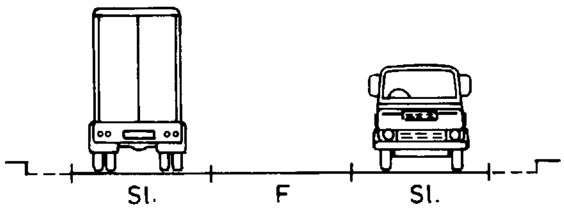
7.3.2 ***Application of loading.*** The loads should be applied to the appropriate lengths of the point load influence lines of not more than two tracks, so as to produce the algebraic maximum and minimum values of stress at the detail under consideration.



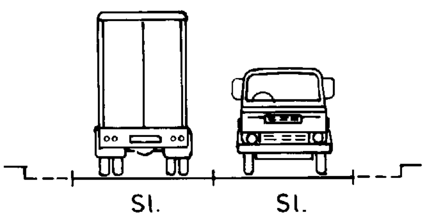
Two and three lane dual carriageways



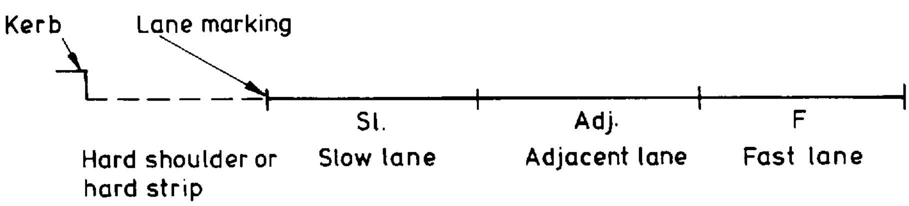
Two lane slip road



Three lane single carriageway

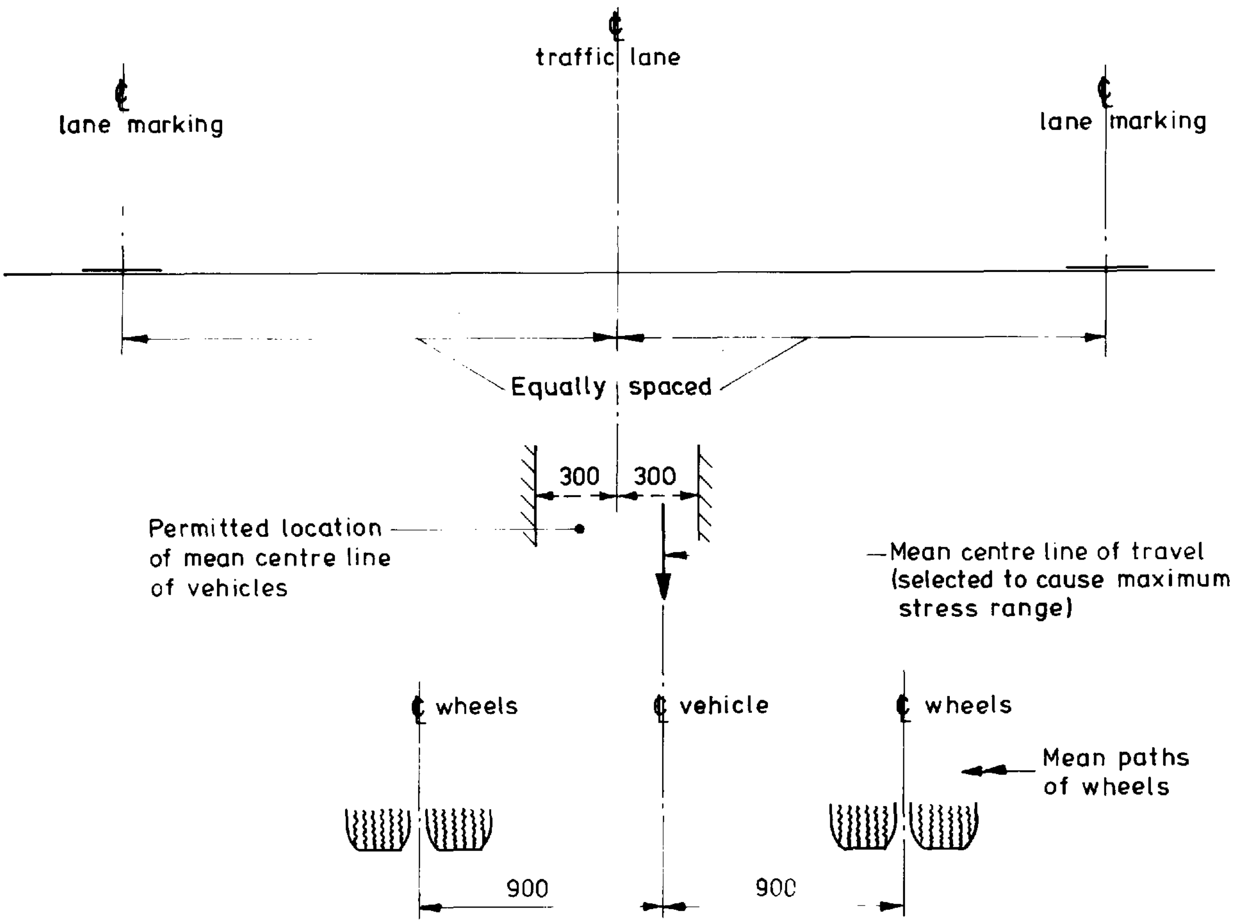


Two lane single carriageway



NOTE. For two lane dual omit fast lane.

Figure 5. Designation of lanes for fatigue purposes



All dimensions are in millimetres

Figure 6. Transverse location of vehicles

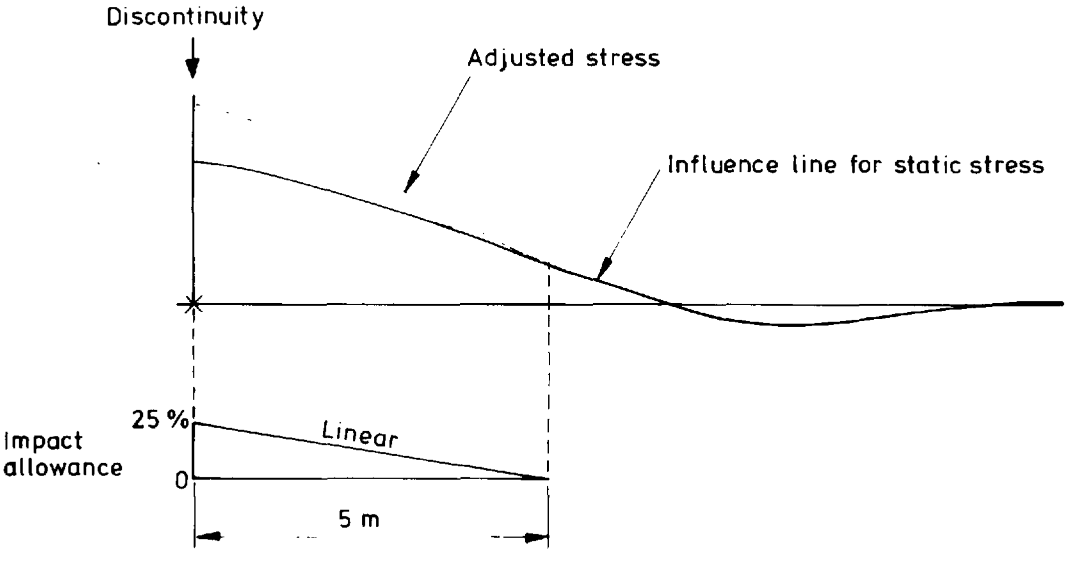


Figure 7. Impact allowance at discontinuities

7.3.3 ***Standard load spectra.*** The load spectrum for a permanent railway bridge subjected to standard loading should be taken from either table 2 for RU loading or table 3 for RL loading. These tables relate proportions of the standard loading K_W to the total number of applied cycles $n_R \times 10^6$ occurring in a design life of 120 years and for a traffic volume of 27×10^6 tonnes per annum. They also allow for variations in the loading events with influence line length. However, reference to tables 2 and 3 is not necessary when the assessment procedure given in **9.2** is used. Where the volumes of traffic differ from the 27×10^6 tonnes per annum, which are assumed in tables 2 and 3, or where a design life other than 120 years is specified, the appropriate values of n_R may be obtained by direct proportion.

NOTE. For the derivation of load spectra see appendix E.

Table 2. Standard load spectra for RU loading

	Heavy traffic								Medium traffic								Light traffic							
Group number	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8	1	2	3	4	5	6	7	8
Load proportion, K_w	0.75	0.65	0.55	0.45	0.35	0.25	0.15	0.05	0.75	0.65	0.55	0.45	0.35	0.25	0.15	0.05	0.75	0.65	0.55	0.45	0.35	0.25	0.15	0.05
Range	0.7 to 0.8	0.6 to 0.7	0.5 to 0.6	0.4 to 0.5	0.3 to 0.4	0.2 to 0.3	0.1 to 0.2	0 to 0.1	0.7 to 0.8	0.6 to 0.7	0.5 to 0.6	0.4 to 0.5	0.3 to 0.4	0.2 to 0.3	0.1 to 0.2	0 to 0.1	0.7 to 0.8	0.6 to 0.7	0.5 to 0.6	0.4 to 0.5	0.3 to 0.4	0.2 to 0.3	0.1 to 0.2	0 to 0.1
Length, L (m)	Total number of live load cycles ($n_R \cdot 10^5$) for various loading groups and types																							
2	58	0	65	0	0	71	0	0	41	18	14	0	130	16	0	0	1	4	17	45	217	5	0	0
3	23	23	26	0	0	40	82	0	0	40	3	8	65	87	0	5	0	2	0.2	3	95	132	45	2
4	0	13	12	46	0	1.5	71	28	0	4	18	27	0	33	32	81	0	2	1	3	27	53	89	111
5	0	3	18	20	5	18	34	43	0	4	3	22	8	3	47	116	0	2	0.1	2	2	42	30	202
7	0	3.5	11	9	6	19	34	62	0	0.3	11	4	1	18	35	52	0	0.1	1	2	7	44	30	60
10	0	3	10	3	8	0	22	40	0	0.3	6	0.3	4	34	17	40	0	0.1	0.5	0.3	10	69	3	60
15	0	1	3	9	8	2	1.5	74	0	0.6	3	3	0	0.3	40	13	0	0.1	0.5	0.3	6	24	51	3
20	0	1	3	4	5	4	6	99	0	0.3	4	0	0	3	30	54	0	0.1	0.5	0.3	5	7	46	27
30	0	2	1	4	0	0	4	79	0	0.5	4	0	0	0	0	138	0	0.2	0.5	0.3	0	10	2	174
⩾50	0	3	0	0	4	0	0	60	0	1.7	0	3	0	0	0	134	0	0.3	0	1	2	8	0	136

NOTE 1. L is the base length of the point load influence line (see figure 12). For intermediate values of L, permissible stress ranges may be derived from the spectra for the two adjacent lengths shown in the table and the values interpolated. n_R values apply to one track.

NOTE 2. The values are based on a traffic volume of 27×10^6 tonnes per annum.

Table 3. Standard load spectra for RL loading

Group number	1	2	3	4	5	6
Load proportion, K_w	0.55	0.45	0.35	0.25	0.15	0.05
Range	0.5 to 0.6	0.4 to 0.5	0.3 to 0.4	0.2 to 0.3	0.1 to 0.2	0 to 0.1
Length, L (m)	Total number of live load cycles ($n_R \times 10^6$) for various loading groups and types					
2	9	120	189	42	0	0
3	1	112	68	10	170	0
4	0	29	75	3	74	180
5	0	6	110	0	2	75
7	0	38	65	0	0	77
10	1	10	56	37	0	77
15	1	13	0	49	30	15
20	1	13	0	0	50	80
30	0	8	6	0	0	265
$\geqslant$50	1	13	0	0	0	80

NOTE 1. L is the base length of the point load influence line (see figure 12). For intermediate values of L, permissible stress ranges may be derived from the spectra for the two adjacent lengths shown in the table and the values interpolated. n_R values apply to one track.

NOTE 2. The values are based on a traffic volume of 27 $\times$ 10^6 tonnes per annum.

8. Fatigue assessment of highway bridges

8.1 Methods of assessment

8.1.1 ***General.*** Three procedures for the fatigue assessment of details in highway bridges are given in **8.2**, **8.3** and **8.4**. The selection of the appropriate procedure depends upon the detail classification, the design life, the load spectrum and the assumed annual flow of commercial vehicles.

8.1.2 ***Simplified procedures.*** As an alternative to the more rigorous procedure of **8.4**, the simplified procedures of **8.2** and **8.3** may be used provided the conditions stated are satisfied.

NOTE. Appendix C gives the derivation of standard highway bridge fatigue loading.

8.2 Assessment without damage calculation

8.2.1 ***General.*** This method determines the limiting value of the maximum range of stress for a 120 year design life and is generally simpler but more conservative than the more exact methods of **8.3** and **8.4**. It should only be used where all the following conditions are satisfied :

(a) the detail class is in accordance with table 17 ;

(b) the design life is 120 years ;

(c) the fatigue loading is the standard load spectrum (see **7.2.2.1**) ;

(d) the annual flows of commercial vehicles are in accordance with table 1.

NOTE. For class S detail only, **8.2.3** provides factors by which non-standard design life, different traffic flow and design HB loading of less than 45 units may be taken into account.

8.2.2 ***Procedure***

8.2.2.1 The following procedure should be used (see appendix D) :

(a) apply the standard fatigue vehicle to each slow and each adjacent lane in turn, in accordance with **7.2.3** ;

(b) apply the impact allowance of **7.2.4**, if appropriate, and determine the maximum and minimum values of principal stress or vector sum stress for weld throat, $\sigma_{p\ max}$ and $\sigma_{p\ min}$ occurring at the detail being assessed, whether resulting from the fatigue vehicle in the same lane or not ;

(c) determine the maximum range of stress $\sigma_{v\ max}$ equal to the numerical value of $\sigma_{p\ max} - \sigma_{p\ min}$. For non-welded details the stress range should be modified as given in **6.1.3** ;

(d) obtain the appropriate limiting stress range σ_H from figure 8.

NOTE. The sign convention used for σ_p is immaterial provided it is applied consistently. Where stress reversal does not occur, the value of either $\sigma_{p\ max}$ or $\sigma_{p\ min}$ should be taken as zero.

8.2.2.2 For class S details the values of σ_H may be adjusted by the factors given in **8.2.3**, when appropriate.

8.2.2.3 Where $\sigma_{v\ max}$ does not exceed σ_H the detail may be considered to have a fatigue life in excess of the specified design life.

8.2.2.4 Where $\sigma_{v\ max}$ exceeds σ_H either of the following options may be adopted.

(a) The detail may be assessed by the alternative procedure given in **8.3** if it is not a class S detail, or by the procedure given in **8.4** if it is a class S detail. However, if $\sigma_{v\ max} > 1.30\ \sigma_H$ for class S details or $> 1.55\ \sigma_H$ for the other classes this option will not satisfy the recommendations of **8.3** and **8.4**.

(b) The detail may be strengthened in order to reduce the value of $\sigma_{v\ max}$ or it may be redesigned to a higher class.

8.2.3 ***Adjustment factors for σ_H, class S details only.*** The values of σ_H obtained from figure 8 may be adjusted by multiplying successively by the following factors where appropriate.

(a) Non-standard design life :

$$\text{factor} = \left(\frac{120}{\text{design life in years}}\right)^{0.125}$$

(b) Non-standard annual flows :

$$\text{factor} = \left(\frac{n_c \text{ (from table 1}}{n_c \text{ (assumed)}}\right)^{0.125}$$

where

n_c is the annual flow in the lane loaded to produce $\sigma_{v\ max} = \sigma_{p\ max} - \sigma_{p\ min}$

NOTE. In the case where $\sigma_{p\ max}$ and $\sigma_{p\ min}$ are produced by loading in two lanes, n_c should be taken as the sum of the flows in those two lanes.

(c) Reduced values of abnormal load capacity (see **C.4.4.2**) :

factor = 1.3 for bridges designed for 37.5 units HB
factor = 1.7 for bridges designed for 25 units HB

8.3 Damage calculation, single vehicle method

8.3.1 ***General.*** This method determines the fatigue life of the detail in question and may be used where a more precise assessment than that provided by the method of **8.2** is required or where the standard design life and/or the annual flows given in table 1 are not applicable. It should only be used where the following conditions are satisfied :

(a) the detail class is in accordance with table 17 but is not class S,

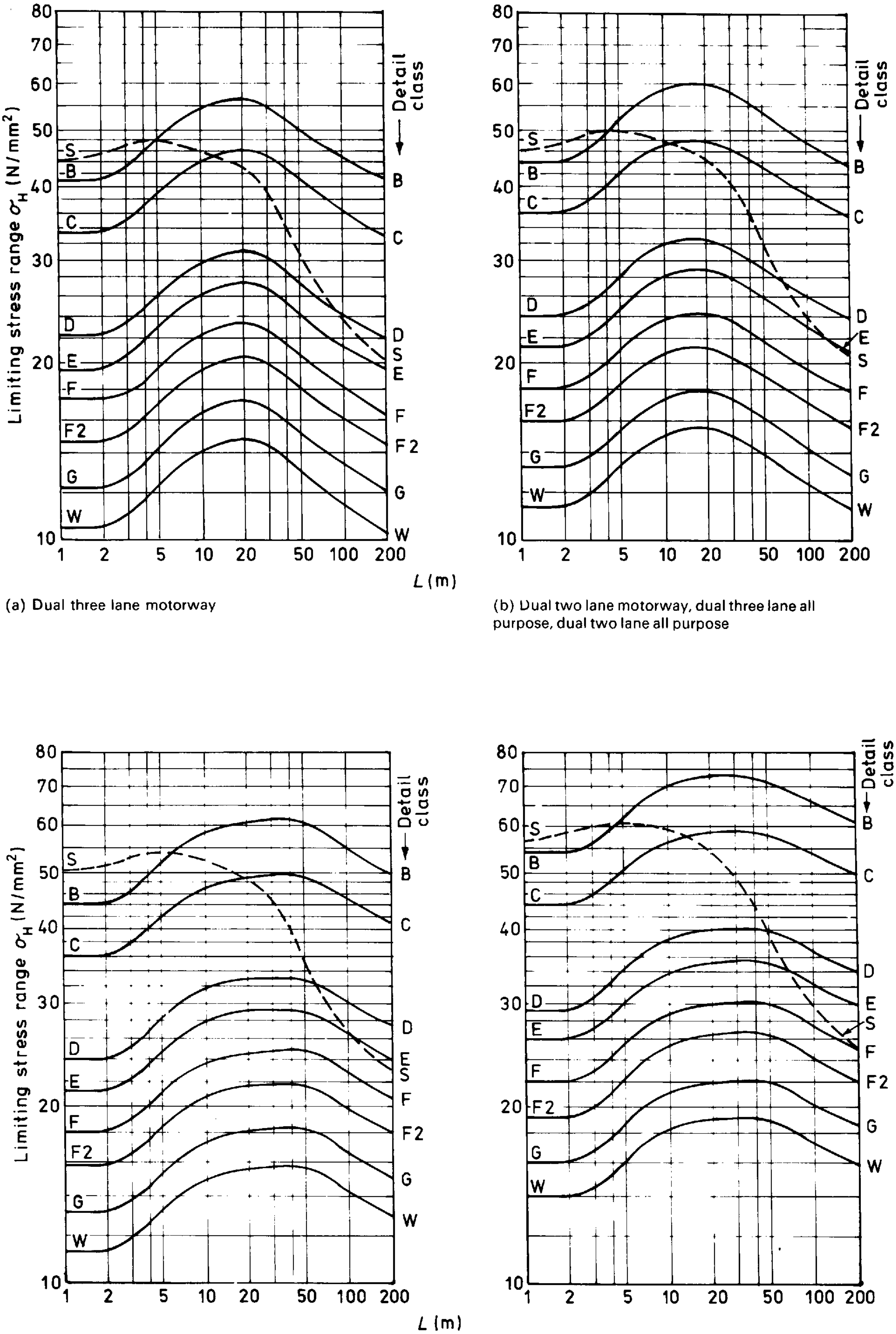
(b) the fatigue loading is the standard load spectrum (see **7.2.2.1**).

8.3.2 ***Procedure***

8.3.2.1 The following procedure should be used (see appendix D).

(a) Apply the standard fatigue vehicle to each slow lane and each adjacent lane in turn, in accordance with **7.2.3**.

(b) Apply the impact allowance of **7.2.4**, if appropriate, and determine the algebraic value of principal stress, or for weld throat, the vector sum stress at the detail being



(a) Dual three lane motorway

(b) Dual two lane motorway, dual three lane all purpose, dual two lane all purpose

(c) three lane all purpose, two lane all purpose (10 m), two lane slip road

(d) two lane all purpose (7.3 m) ; single lane slip road

Figure 8. Values of σ_H for different road categories

assessed for each peak and each trough in the stress history of each lane in turn (see figure 9).

NOTE. It is sufficiently accurate to calculate each peak or trough value of the direct stress and to obtain the principal stress by combining these with the coincident shear stress, or vice-versa where this is more severe.

(c) When the maximum and the minimum algebraic values of stress $\sigma_{p\ max}$, $\sigma_{p\ min}$, result from vehicle positions in the same lane (referred to as case 1 in figure 9) the damage should be calculated for the stress histories for each lane separately.

When $\sigma_{p\ max}$ and $\sigma_{p\ min}$ result from vehicle positions in different lanes (referred to as case 2 in figure 9) an additional combined stress history should be derived, which allows for the increased maximum stress range produced by a proportion of the vehicles travelling in alternating sequence in the two lanes. In this case the damage should be calculated for the combined stress history as well as for the separate lane stress histories.

(d) Derive the stress spectrum σ_{v1}, σ_{v2} etc., from each stress history determined from (c).

Where a stress history contains only one peak and/or only one trough, only one cycle results, as shown in figure 9 for lanes C and D, and the stress range can be determined directly.

Where a stress history contains two or more peaks and/or two or more troughs, as shown in figure 9 for lanes A and B, more than one cycle results and the individual stress ranges should be determined by the reservoir method given in appendix B.

(e) Determine the effective annual flow of commercial vehicles, n_c million, appropriate to each stress spectrum as follows:

(1) where case 1 of figure 9 applies, $\bar{n}_c = n_c$ and may be derived directly from table 1 unless different vehicle flows are adopted;

(2) where case 2 of figure 9 applies the effective annual flow $\bar{n}_c$ should be obtained as indicated in figure 9 for case 2.

(f) For each stress range σ_v of each stress spectrum, determine the appropriate lifetime damage factor d_{120} from the damage chart of figure 10 and multiply each of these values by the appropriate value of $\bar{n}_c$. For non-welded details the stress range should be modified as in **6.1.3**.

(g) Determine the value of the adjustment factor K_F from figure 11 according to the base length L of the point load influence line (see figure 12) and the stress range ratio K_B defined in figure 11.

For a combined stress history from two lanes (see (c) above and case 2 in figure 9) K_B should be taken as zero for determining K_F.

NOTE. For the derivation of K_F see appendix C.

(h) Determine the predicted fatigue life of the detail from the following expression:

$$\text{fatigue life (in years)} = \frac{120}{\Sigma K_F \bar{n}_c d_{120}}$$

where the summation includes all the separate lane stress histories as well as the combined stress history, where appropriate.

8.3.2.2 Where the predicted fatigue life of the detail is less than the specified design life, the detail should either be strengthened to reduce the value of $\sigma_{v\ max}$ or redesigned to a higher class and then re-checked as in **8.3.2.1**.

As a guide, an approximate stress range for the same class of detail can be obtained by multiplying the original value by:

$$\left(\frac{\text{predicted life}}{\text{design life}}\right)^{1/(m+1)}$$

where

m is the inverse slope of the appropriate log σ_r/log N curve given in table 8.

If the detail is to be redesigned to a higher class the procedure given in **11.5**(b) may be used as a guide to assess the adequacy of the proposed detail.

8.4 Damage calculation, vehicle spectrum method

8.4.1 ***General.*** This method involves an explicit calculation of Miner's summation and may be used for any detail for which the σ_r–N relationship is known and for any known load or stress spectrum.

8.4.2 ***Design spectrum***

8.4.2.1 The individual stress spectra for the detail being assessed should be derived by traversing each vehicle in the load spectrum along the various lanes. Account should also be taken of the possibility of higher stress ranges due to some of the vehicles occurring simultaneously in one or more lanes and/or in alternating sequence in two lanes. For non-welded details the stress range should be modified as given in **6.1.3**.

8.4.2.2 In the absence of other evidence, allowance for impact should be made in accordance with **7.2.4**. The design spectrum should then be determined by combining the stress spectra with the specified numbers of vehicles in the respective lanes.

8.4.2.3 In assessing an existing structure, a design spectrum may be compiled from strain readings or traffic records obtained from continuous monitoring.

8.4.3 ***Simplification of design spectrum.*** The design spectrum may be divided into any convenient number of intervals, as shown in figure 13, with all the stress ranges in any one interval being treated as the maximum range in that interval but low stress ranges should be treated in accordance with **11.3**. It should be noted that the use of small intervals will reduce the conservatism in fatigue assessment.

8.4.4 ***Calculation of damage.*** Using the design spectrum, the value of Miner's summation $\sum \frac{n}{N}$ should be calculated in accordance with clause **11**. This value should not exceed 1.0 for the fatigue life of the detail to be acceptable.

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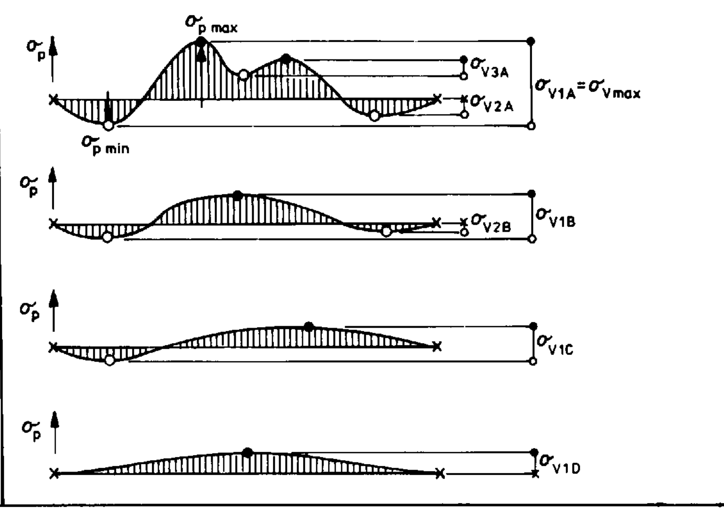
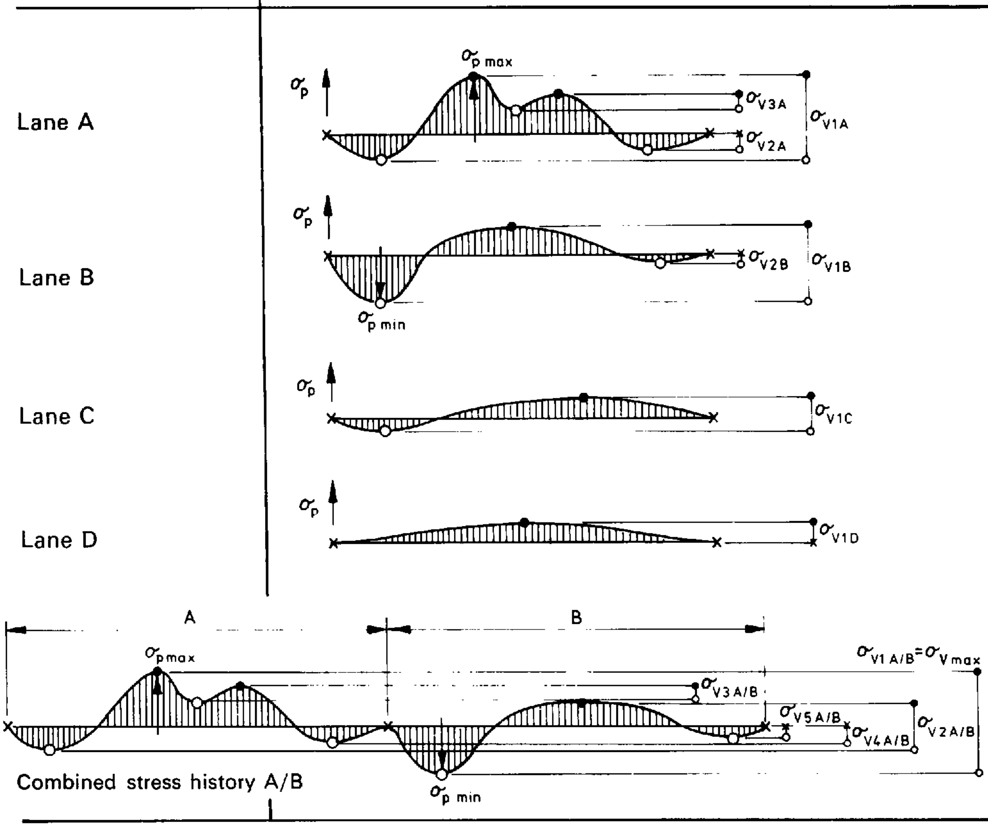


Lane reference*	Lane stress history	Lane stress spectra†	Number of cycles per loading event	Effective lane flow, $\bar{n}_c$
Lane A			3	n_{CA}
Lane B			2	n_{CB}
Lane C			1	n_{CC}
Lane D			1	n_{CD}

Case 1. Highest peak and lowest trough with vehicles in same lane



Lane reference*	Lane stress history	Lane stress spectra†	Number of cycles per loading event	Effective lane flow, $\bar{n}_c$
Lane A			3	$\frac{n^2_{CA}}{n_{CA} + n_{CB}}$
Lane B			2	$\frac{n^2_{CB}}{n_{CA} - n_{CB}}$
Lane C			1	n_{CC}
Lane D			1	n_{CD}
Combined stress history A/B			5	$\frac{n_{CA} \times n_{CB}}{n_{CA} + n_{CB}}$

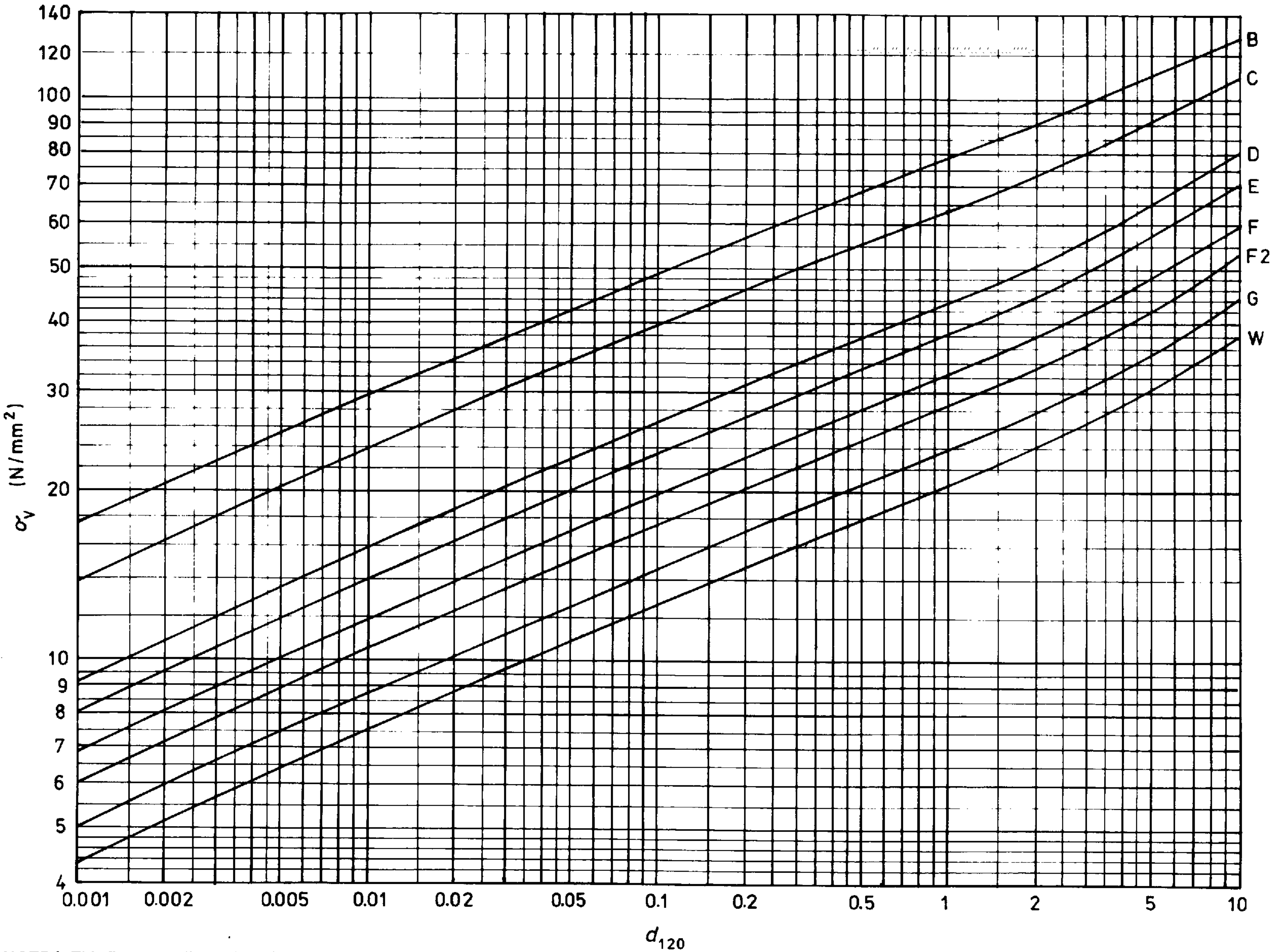
Case 2. Highest peak and lowest trough with vehicles in different lanes

Key
● Peak stress
○ Trough stress
X Datum stress

Stress range σ_v for cycle shown

*The lane reference letters A, B etc. should be allocated in descending order of magnitude of stress ranges σ_{V1A}, σ_{V1B} etc.
† Values of σ_v should be obtained by the method given in appendix B.

Figure 9. Derivation of σ_v and $\bar{n}_c$ for damage calculation

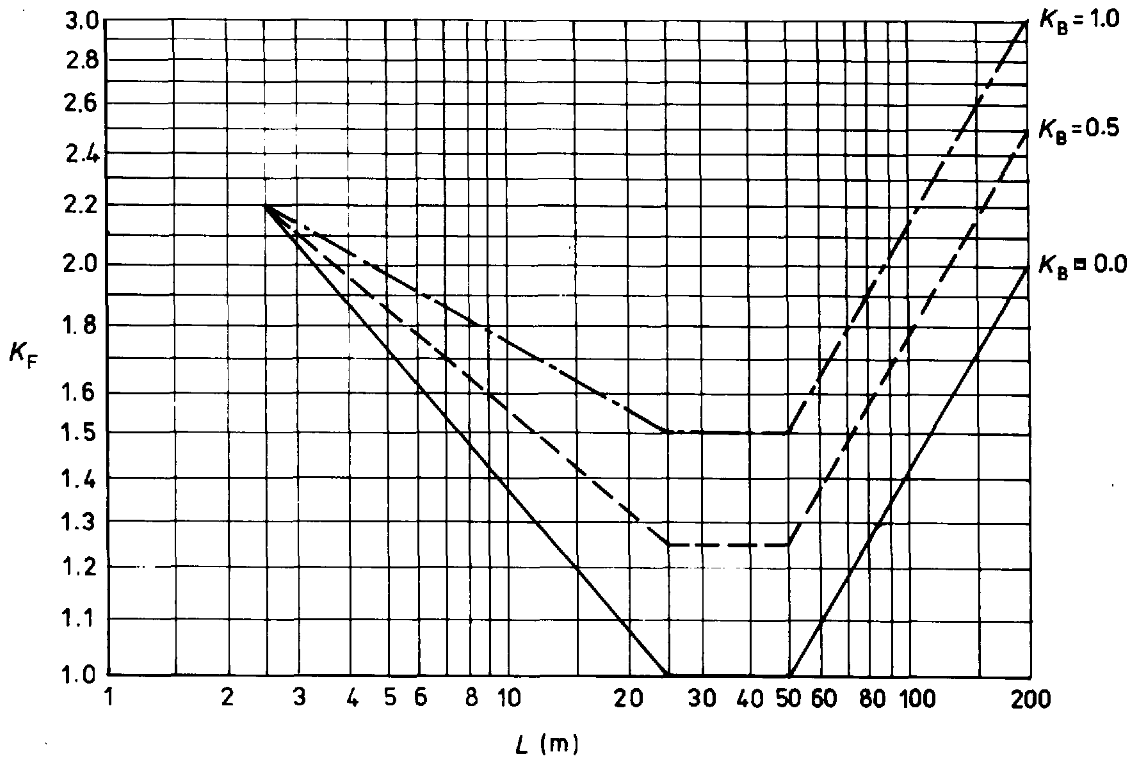


NOTE 1. This figure applies only to fatigue assessment which uses the standard fatigue vehicle. It is based on the standard load spectrum given in table 11 and is derived from the design σ_r—N curves of **11.2**.

NOTE 2. For detail classes see **5.1** and table 17.

NOTE 3. σ_v is the stress range under the standard fatigue vehicle (N/mm^2)
d_{120} is the lifetime damage factor for 1 million cycles per annum for 120 years (1.2×10^8 cycles).

Figure 10. Damage chart for highway bridges (values of d_{120})



NOTE 1. L is the base length of the point load influence line (see figure 12).

K_B is the ratio $\frac{\sigma_{V1B}}{\sigma_{V1A}}$

where

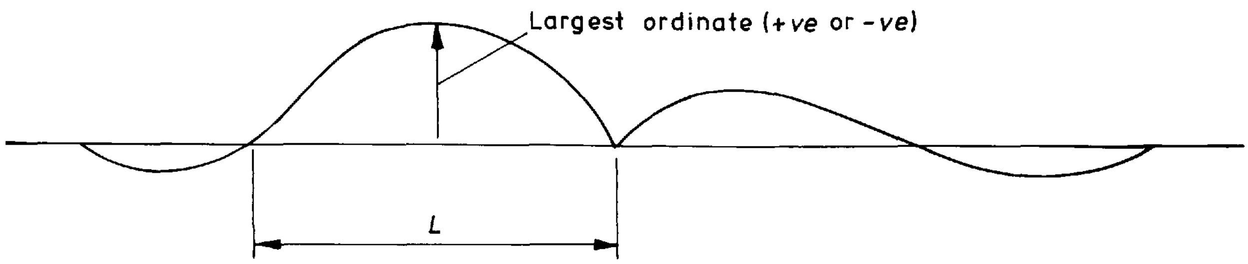
σ_{V1A} is the largest stress range produced by loading in any one lane

σ_{V1B} is the next largest stress range produced by loading in any other lane ($\sigma_{V1B} < \sigma_{V1A}$).

For non-welded details the stress range should be modified as given in **6.1.3**.
(K_B may be taken as zero for the combined history of figure 9, case 2).

NOTE 2. This figure is applicable only to detail classes B to G, F2 and W.

Figure 11. Miner's summation adjustment factor K_F for highway bridges



NOTE. L is the base length of loop containing the largest ordinate measured in direction of travel. For an element of a highway bridge loaded by more than one lane, L should be determined from the influence line for the lane producing the largest value of σ_{V1} ($= \sigma_{V1A}$) (see figure 9).

Figure 12. Typical point load influence line

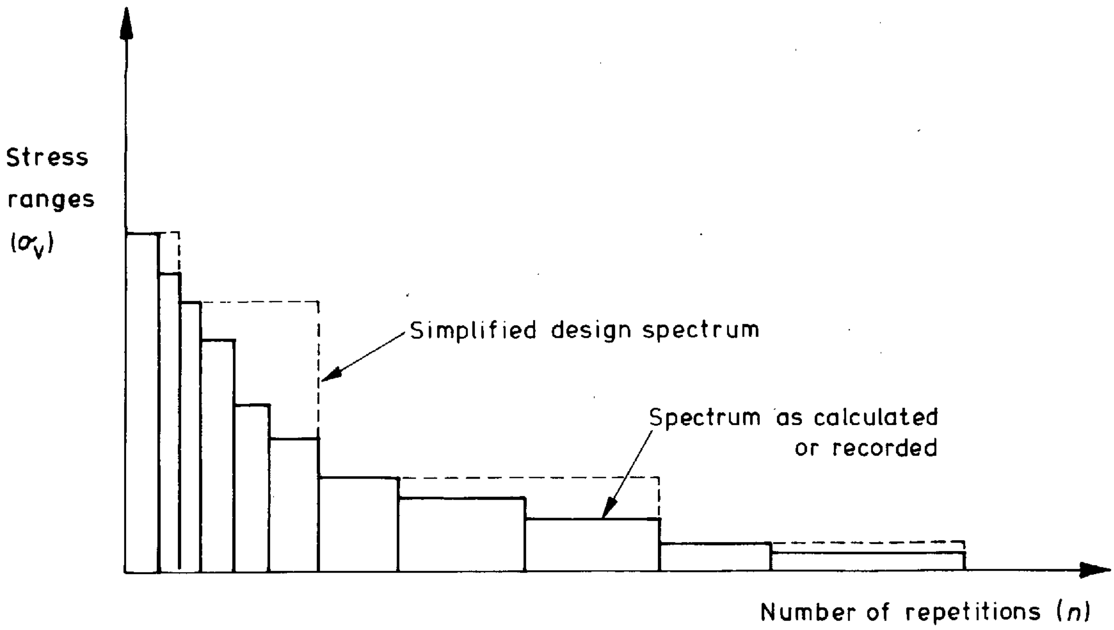


Figure 13. Simplification of a spectrum

9. Fatigue assessment of railway bridges

9.1 Methods of assessment

9.1.1 *General.* Two methods for the fatigue assessment of details in railway bridges are given in **9.2** and **9.3**. The choice of the appropriate method depends upon the detail classification and the nature of the loading.

9.1.2 *Simplified procedure.* As an alternative to the more rigorous method of **9.3** the simplified procedure of **9.2** may be used provided the conditions stated therein are satisfied.

9.2 Assessment without damage calculation

9.2.1 *General.* This method determines the limiting value of the maximum range of stress for the specified design life. It should only be used where the following conditions are satisfied :

(a) the detail class is in accordance with table 17 ;

(b) the loading is the standard railway bridge loading in accordance with **7.3**.

The simplified procedure produces the same results as the method given in **9.3** when the coefficients k_1, k_2, k_4 and k_5 (see **9.2.2**) are equal to unity. In other cases the method may be more conservative than the method given in **9.3**.

9.2.2 ***Procedure***

9.2.2.1 The following procedure should be used.

(a) Apply the standard railway loading in accordance with **7.3.1** and **7.3.2**.

(b) Determine the maximum and minimum values of principal stress or vector sum stress for weld throat $\sigma_{p\ max}$ and $\sigma_{p\ min}$, occurring at the detail being assessed, by loading the appropriate loops of the point load influence line, as shown in example 4 of appendix F, whether resulting from railway loading on the same track or not.

(c) Determine the maximum range of stress $\sigma_{R\ max}$ equal to the numerical value of $\sigma_{p\ max}-\sigma_{p\ min}$. For non-welded details the stress range should be modified as given in **6.1.3**.

(d) Obtain the appropriate limiting stress range σ_T from the following expressions :

$\sigma_T = k_1 \times k_2 \times k_3 \times k_4 \times k_5 \times \sigma_0$ for RU loading

$\sigma_T = k_1 \times k_2 \times k_4 \times k_5 \times k_6 \times \sigma_0$ for RL loading

where

k_1 = 1.0 if the design life is 120 years, otherwise it is obtained from **9.2.3**

k_2 = 1.0 if the loading event produces only one cycle of stress, otherwise it is obtained from **9.2.4**

k_3 is obtained from table 4

k_4 is obtained from table 5

k_5 is obtained from table 6

k_6 is obtained from table 7

σ_0 is the constant amplitude non-propagating stress range for the appropriate class of detail and is obtained from table 8

NOTE. The sign convention used for σ_p is immaterial providing it is consistently applied. Where stress reversal does not occur under the loading described, either $\sigma_{p\ max}$ or $\sigma_{p\ min}$ should be be taken as zero.

9.2.2.2 Where $\sigma_{R\ max}$ does not exceed σ_T the detail may be considered to have a fatigue life in excess of the specified design life.

9.2.2.3 Where $\sigma_{R\ max}$ is found to exceed σ_T either of the following options may be adopted :

(a) the detail may be assessed by the more precise procedure given in **9.3** ;

(b) the detail may be strengthened in order to reduce the value of $\sigma_{R\ max}$ or it may be redesigned to a higher class.

9.2.3 ***Non-standard design life.*** Where the specified design life is other than 120 years, the value of k_1 should be taken as the lesser of either :

(a) $\left(\dfrac{120}{\text{design life in years}}\right)^{\frac{1}{m}}$

or

(b) $\left(\dfrac{120}{\text{design life in years}}\right)^{\frac{1}{m+2}}$

where

m is the inverse slope of the $\sigma_r - N$ curve appropriate to the detail class and is obtained from table 8.

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Table 4. Values of k_3 for RU loading of railway bridges

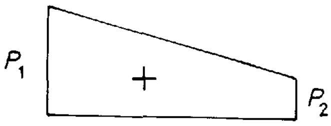
	Heavy traffic				Medium traffic				Light traffic			
Detail class	D E F F2 G W	C	B	S	D E F F2 G W	C	B	S	D E F F2 G W	C	B	S
Length, L (m)	**Values of k_3**											
<3.4	1.00	1.00	1.01	1.14	1.09	1.09	1.13	1.28	1.37	1.60	1.60	1.71
3.4 to 4.0	1.09	1.09	1.13	1.28	1.23	1.22	1.30	1.46	1.53	1.79	1.80	1.71
4.0 to 4.6	1.23	1.22	1.30	1.46	1.37	1.36	1.46	1.46	1.71	1.79	1.80	1.71
4.6 to 7.0	1.37	1.36	1.46	1.65	1.53	1.56	1.62	1.65	1.92	2.05	2.00	1.95
7.0 to 10.0	1.53	1.56	1.62	1.65	1.71	1.75	1.81	1.83	2.19	2.31	2.24	2.20
10.0 to 14.0	1.71	1.75	1.62	1.65	1.92	1.95	2.03	1.83	2.46	2.31	2.50	2.20
14.0 to 28.0	1.92	1.95	2.03	1.83	2.19	2.18	2.03	1.83	2.74	2.56	2.50	2.20
>28.0	2.19	1.95	2.03	1.83	2.46	2.18	2.03	1.83	3.06	2.87	2.50	2.20

NOTE. L is the base length of the point load influence line (see figure 12).

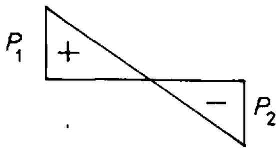
Table 5. Values of k_4 for railway bridges

Annual traffic tonnage on one track (millions of tonnes)					
42 to 27	27 to 18	18 to 12	12 to 7	7 to 5	<5
0.89	1.0	1.13	1.27	1.42	1.6

Table 6. Values of k_5 for railway bridges



Ratio $\frac{P_1}{P_1 + P_2}$



Ratio $\frac{P_2}{P_1}$

0. 5 to 0.6	0.6 to 0.75	0.75 to 0.9	0.9 to 1.0	0.0 to 0.7	0.7 to 1.0
1.42	1.27	1.125	1.0	1.0	0.89

NOTE. P_1 is the numerical value of stress due to track 1.
P_2 is the numerical value of stress due to track 2 (where track 1 is the track causing the greater stress at the detail under consideration).
Trains on two tracks should be considered in the same longitudinal position.

Table 7. Values of k_6 for RL loading of railway bridges

Detail class	D E F F2 G W	C	B	S
Length, L (m)	**Values of k_6**			
<3.0	1.23	1.28	1.35	1.65
3.0 to 3.4	1.34	1.37	1.45	1.71
3.4 to 4.0	1.43	1.49	1.55	1.80
4.0 to 10.0	1.57	1.62	1.68	1.91
10.0 to 15.0	1.77	1.79	1.90	2.10
15.0 to 20.0	1.98	1.99	2.00	2.10
>20.0	2.08	2.05	2.09	2.10

NOTE. L is the base length of the point load influence line (see figure 12).

9.2.4 ***Multiple cycles.*** Where the loading event produces more than one cycle of stress the value of k_2 should be taken as:

$$\left(1 + \left(\frac{\sigma_{R2}}{\sigma_{R1}}\right)^m + \left(\frac{\sigma_{R3}}{\sigma_{R1}}\right)^m + \ldots\right)^{-\frac{1}{m}}$$

where

m is defined in **9.2.3**

σ_{R1}, σ_{R2}, σ_{R3} etc. are the stress ranges, in descending order of magnitude, at the individual cycles produced by the approach, passage and departure of a unit uniformly distributed load.

NOTE. Such cycles should be counted and the individual stress ranges determined by the reservoir method given in appendix B. An illustration of the multiple cycle stress history is given in example 4 of appendix F.

9.3 Damage calculation

9.3.1 ***General.*** This method involves a calculation of Miner's summation and may be used for any detail for which the $\sigma_r - N$ relationship is known and for any known load or stress spectra. It may also be used as a more precise alternative to the simplified procedure of **9.2**.

9.3.2 ***Design spectrum for standard loading***

9.3.2.1 Applying the standard railway loading as given in **7.3.1** and **7.3.2** the value of $\sigma_{R\,max}$ should be derived in accordance with the procedure set out in **9.2.2.1** (a) to (c). The design spectrum should then be determined by the use of either table 2 for RU loading or table 3 for RL loading (amended where appropriate in accordance with **7.3.3**). These tables indicate, for simply supported members, the equivalent frequency of occurrence of stress ranges of varying magnitudes resulting from the passage of the individual trains forming various standard traffic types, where the stress ranges are expressed as proportions of the maximum stress range.

9.3.2.2 In the case of loading from more than one track, account should be taken of the possibility of stress fluctuations arising from the passage of trains on not more than two tracks, both separately and in combination. As an approximation, the effects of two track loading may be obtained by dividing $\sigma_{R\,max}$ (see **9.3.2.1**) by the coefficient k_5 which can be obtained from table 6.

9.3.2.3 Where the approach, passage and departure of a unit uniformly distributed load produces more than one cycle of stress, as for instance in multi-span longitudinal or cross members or in continuous deck slabs, all the cycles should be taken into account. The appropriate standard trains of figure 19 or figure 20 should be traversed across the relevant point load influence lines and the resulting stress histories should be analysed by the reservoir method, given in appendix B, to derive the respective stress spectra. These should then be combined with the appropriate annual occurrences obtained from table 15 or table 16 proportioned for the required traffic volume and multiplied by the specified design life to produce the overall design spectrum. As an approximation, the effect of the additional cycles may be obtained by dividing either $\sigma_{R\,max}$ (see **9.3.2.1**) or $\sigma_{R\,max}/k_5$ (see **9.3.2.2**) by the coefficient k_2 which should be obtained from **9.2.4**.

9.3.3 ***Design spectrum for non-standard loading***

9.3.3.1 Where the loading does not comply with **7.3.1** the appropriate train should be traversed across the relevant point load influence lines and the resulting stress histories should be analysed by the rainflow method* to derive the respective stress spectra. These should then be combined with the appropriate total occurrences in the design life of the bridge to compile the overall design spectrum. For non-welded details the stress range should be modified as given in **6.1.3**.

9.3.3.2 In assessing an existing structure, a design spectrum may be compiled from strain readings or traffic records obtained from continuous monitoring.

9.3.4 ***Simplification of spectrum.*** Where a non-standard loading is used in accordance with **7.1**, or the stress ranges are obtained from strain gauge readings, the design spectrum should be divided into at least 10 equal intervals of stress. All the stress ranges in any one interval should be treated as the mean range in that interval and low stress ranges should be treated in accordance with **11.3**.

9.3.5 ***Calculation of damage.*** Using the design spectrum, the value of Miner's summation $\sum \frac{n}{N}$ should be calculated in accordance with clause **11** and should not exceed 1.0 for the fatigue life of the detail to be acceptable.

10. Fatigue assessment of bridges carrying highway and railway loading

In the case of bridges carrying both highway and railway loadings, the total damage (i.e. 120 years divided by the predicted life) should be determined for each loading condition separately, in accordance with **8.3** or **8.4** and **9.3**.

To obtain the total damage, the sum of the two damage values should be multiplied by a further adjustment factor which takes into account the probability of coexistence of the two types of loading. This factor should be determined for a given member after consideration of the fact that coincidence of highway traffic on multiple lanes and of railway traffic on multiple tracks has already been taken into account in assessing the separate damage values.

Except at very busy railway stations, where the probability of coincidence of rail and road traffic is higher than on the open track, the adjustment factor is not expected to exceed 1.2 where the stresses from highway and railway loading are of the same sign.

11. The Palmgren-Miner rule

11.1 General. The value of Miner's summation $\sum \frac{n}{N}$ for use in **8.4.4** and **9.3.5** should be determined from the following expression:

$$\sum \frac{n}{N} = \left(\frac{n_1}{N_1} + \frac{n_2}{N_2} + \ldots\ldots + \frac{n_n}{N_n}\right)$$

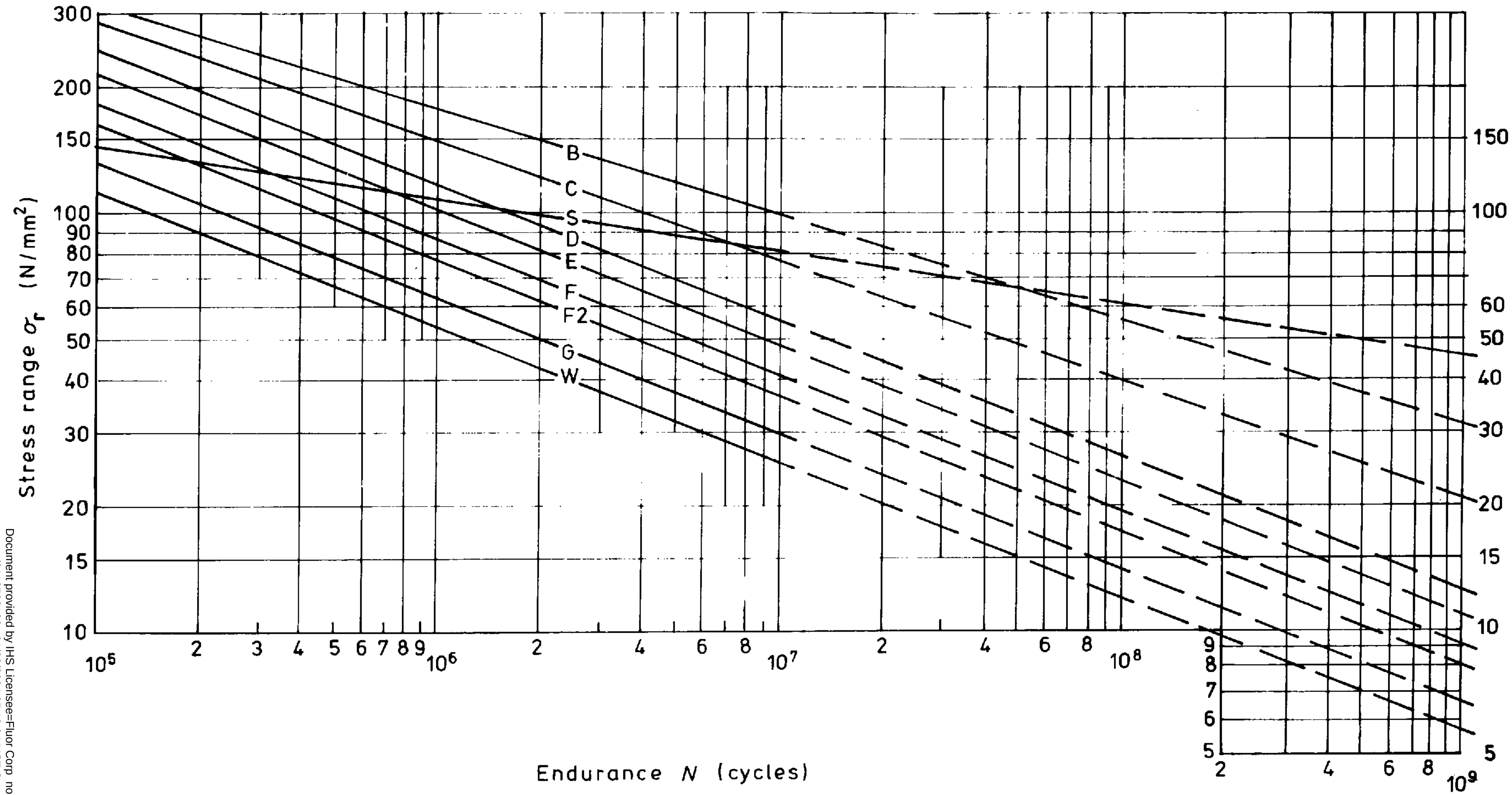
where

$n_1, n_2 \ldots n_n$ are the specified numbers of repetitions of the various stress ranges in the design spectrum, which occur in the design life of the structure.

NOTE. The number of repetitions may be modified in accordance with **11.3**, and for non-welded details the stress range should be modified as given in **6.1.3**.

$N_1, N_2 \ldots N_n$ are the corresponding numbers of repetitions to failure for the same stress ranges, obtained from **11.2**.

*The rainflow method is described in ORE D128 Report No. 5 'Bending moment spectra and predicted lives of railway bridges', published by the Office for Research and Experiments of the International Union of Railways. The reservoir method of cycle counting, described in appendix B for highway bridges, may be applied to stress histories for railway bridges (see example 4 of appendix F) and will produce the same results as the rainflow method for many repetitions of the loading event.



NOTE 1. The use of these curves for calculation purposes is not recommended.

NOTE 2. For endurances greater than 10^7 cycles adjustments should be made in accordance with **11.3**.

Figure 14. Summary design σ_r—N curves (mean minus two standard deviations)

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11.2 Design σ_r – N relationship. The number of repetitions to failure N of any one stress range σ_r should be obtained from either of the following equations, which have been plotted in figure 16:

$$N \times \sigma_r^m = K_2$$

$$\text{Log}_{10} N = \text{Log}_{10} K_2 - m \text{ Log}_{10} \sigma_r$$

where

K_2 and m have the values given in table 8 for the different detail classes.

NOTE. The values of K_2 correspond to a probablity of failure of 2.3 % within the design life. The basic equations and a mean-line plot, i.e. for 50 % probability of failure, are given in appendix A.

11.3 Treatment of low stress cycles. The number of repetitions of each stress range σ_r less than σ_0 should be reduced in the proportion $(\sigma_r/\sigma_0)^2$

where

σ_0 is the stress range given by the equation in **11.2** for $N = 10^7$ and tabulated in table 8.

It may assist in calculations to note that:

$$\frac{n}{N} = \frac{n\sigma_r^m}{K_2} = \frac{n}{10^7}\left(\frac{\sigma_r}{\sigma_0}\right)^m \text{ when } \sigma_r \geqslant \sigma_0$$

$$\frac{n}{N} = \frac{n\sigma_r^{m+2}}{K_2\sigma_0^2} = \frac{n}{10^7}\left(\frac{\sigma_r}{\sigma_0}\right)^{(m+2)} \text{ when } \sigma_r \leqslant \sigma_0$$

11.4 Procedure. The following procedure should be used in applying the Palmgren-Miner rule.

(a) Determine the class of each detail in accordance with table 17.

(b) Calculate the stresses and hence the stress ranges at each detail in accordance with clause **6** and determine the design spectrum in accordance with **8.4** and **9.3**. The number of low stress cycles should be modified in accordance with **11.3**.

(c) Determine the number of repetitions to failure N of each of the stress ranges in the design spectrum in accordance with **11.2**.

(d) Evaluate Miner's summation in accordance with **11.1**.

11.5 Miner's summation greater than unity. If the conditions in **8.4.4** and **9.3.5** for highway and railway bridges respectively are not met, i.e. if $\sum \frac{n}{N} > 1.0$, the following alternative actions should be considered.

Either

(a) strengthen the detail to reduce the values of σ_r. The strengthened detail should be satisfactory if the reduced values of stresses lie between the limits obtained by dividing the original values by the following factors:

$$\left(\sum \frac{n}{N}\right)^{1/m} \text{ and}$$

$$\left(\sum \frac{n}{N}\right)^{1/(m+2)}$$

where

m is obtained from table 8

or

(b) redesign the detail to a higher class. As a guide for upgrading to any class up to D, the value of σ_0 for the new class should be between

$$\left(\sum \frac{n}{N}\right)^{0.33} \text{and} \left(\sum \frac{n}{N}\right)^{0.2}$$

times the value of σ_0 of the original class of the detail.

Table 8. σ_r – N relationships and constant amplitude non-propagating stress range values

Detail class	m	K_2	σ_0 (N/mm²)
W	3.0	0.16×10^{12}	25
G	3.0	0.25×10^{12}	29
F2	3.0	0.43×10^{12}	35
F	3.0	0.63×10^{12}	40
E	3.0	1.04×10^{12}	47
D	3.0	1.52×10^{12}	53
C	3.5	4.23×10^{13}	78
B	4.0	1.01×10^{15}	100
S	8.0	2.08×10^{22}	82

NOTE. Values applicable to non-standard criteria may be obtained from appendix A.

Appendix A

Basis of $\sigma_r - N$ relationship

A.1 General. The $\sigma_r - N$ relationships have been established from statistical analyses of available experimental data (using linear regression analysis of log σ_r and log N) with minor empirical adjustments to ensure compatability of results between the various classes.

The equation given in **11.2** may be written in basic form as:

$$N \times \sigma_r^m = K_0 \times \Delta^d$$

where

N is the predicted number of cycles to failure of a stress range σ_r

K_0 is the constant term relating to the mean-line of the statistical analysis results

m is the inverse slope of the mean-line log σ_r – log N curve

Δ is the reciprocal of the anti-log of the standard deviation of log N

d is the number of standard deviations below the mean-line.

NOTE. This corresponds to a certain probability of failure, as shown in table 10.

The relevant values of these terms are given in tables 9 and 10 and the mean-line relationships are plotted in figure 15.

Table 9. Mean-line $\sigma_r - N$ relationships

Detail class	K_0	Δ	m
W	0.37×10^{12}	0.654	3.0
G	0.57×10^{12}	0.662	3.0
F2	1.23×10^{12}	0.592	3.0
F	1.73×10^{12}	0.605	3.0
E	3.29×10^{12}	0.561	3.0
D	3.99×10^{12}	0.617	3.0
C	1.08×10^{14}	0.625	3.5
B	2.34×10^{15}	0.657	4.0
S	2.13×10^{23}	0.313	8.0

Table 10. Probability factors

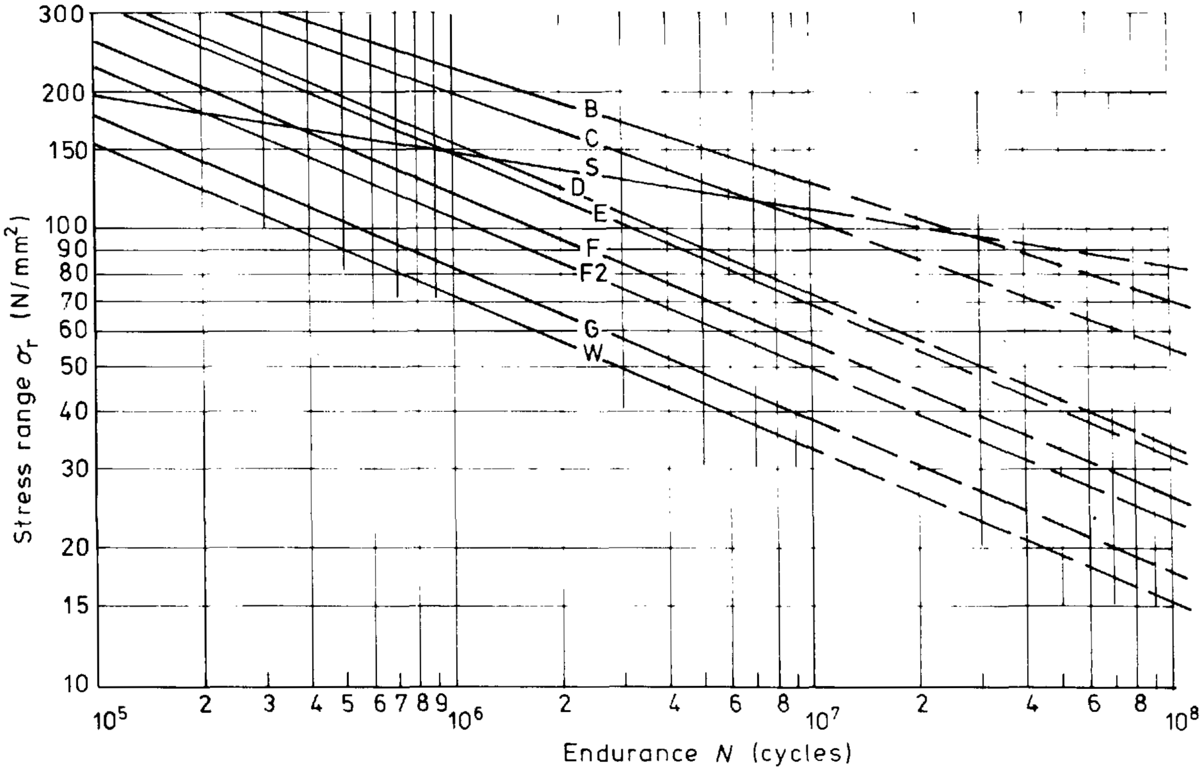
Probability of failure	d
50 %	0*
31 %	0.5
16 %	1.0
2.3 %	2.0†
0.14 %	3.0

*Mean-line curve.
†The standard design curve of **11.2**.

A.2 Treatment of low stress cycles. Under fluctuating stress of constant amplitude, there is a certain stress range below which an indefinitely large number of cycles can be sustained. The value of this 'non-propagating stress range' varies both with the environment and with the size of any initial defect in the stressed material. In clean air, a steel detail which complies with the requirements of Parts 6, 7 or 8 is considered to have a constant amplitude non-propagating range σ_0 equal to the value of σ_r obtained from the formula in **A.1** when $N = 10^7$.

When the applied fluctuating stress has varying amplitude, so that some of the stress ranges are greater and some less than σ_0, the larger stress ranges will cause enlargement of the initial defect. This gradual enlargement reduces the value of the non-propagating stress range below σ_0. Thus, as time goes on, an increasing number of stress ranges below σ_0 can themselves contribute to the further enlargement of the defect. The final result is an earlier fatigue failure than could be predicted by assuming that all stress ranges below σ_0 are ineffective.

This phenomenon has been studied on principles derived from fracture mechanics. It is found that an adequate approximation to the fatigue performance so predicted can be obtained by assuming that a certain fraction $(\sigma_r/\sigma_0)^2$ of stress ranges σ_r less than σ_0 cause damage in accordance with the formula in **A.1**.



NOTE. The use of these curves for calculation purposes is not recommended.

Figure 15. Summary of mean-line $\sigma_r - N$ curves

The same result can be obtained by using a notional log σ_r/log N curve, which has the inverse slope $m + 2$ where N is greater than 10^7.

These points are illustrated in figure 16, which shows a typical log σ_r/log N curve.

A.3 Fatigue life for various failure probabilities. The standard design $\sigma_r - N$ curves in figure 14 are based on two standard deviations below the mean-line with a probability of failure of 2.3 %. In certain cases, a higher probability of failure could be acceptable, for example, where fatigue cracking would not have serious consequences, or where a crack could be easily located and repaired.

The probabilities of failure associated with various numbers of standard deviations below the mean-line are given in table 10. The $\sigma_r - N$ curves appropriate to other numbers of standard deviations below the mean-line can be derived from the formula given in **A.1**.

Where the methods of clauses **8, 9** or **11**, which are based on two standard deviations below the mean-line, predict the fatigue life (or damage), the life (or damage) appropriate to other numbers of standard deviations below the mean-line can be obtained by multiplying the calculated life (or dividing the calculated damage) by the following factors:

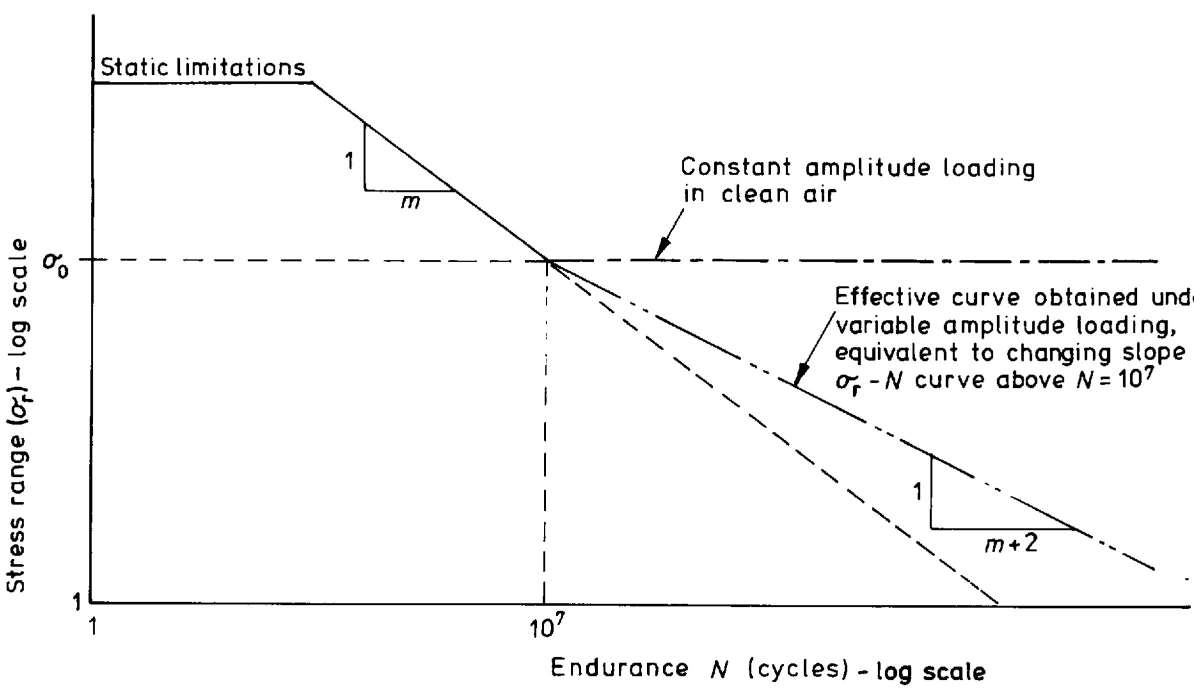
Detail class	Number of standard deviations below the mean-line			
	1.5	1.0	0.5	0.0
W	1.24	1.53	1.89	2.34
G	1.23	1.51	1.86	2.28
F2	1.30	1.69	2.20	2.85
F	1.29	1.65	2.12	2.73
E	1.34	1.78	2.38	3.18
D	1.27	1.62	2.06	2.63
C	1.27	1.60	2.02	2.56
B	1.23	1.52	1.88	2.32
S	1.79	3.20	5.71	10.21

The simplified assessment method of **8.2** gives limiting stress ranges. For numbers of standard deviations below the mean-line other than two (as used in the methods), these limiting stress ranges can be multiplied by the following factors:

Detail class	Number of standard deviations below the mean-line			
	1.5	1.0	0.5	0.0
W	1.07	1.15	1.24	1.32
G	1.07	1.15	1.23	1.32
F2	1.09	1.19	1.30	1.42
F	1.09	1.18	1.29	1.40
E	1.10	1.21	1.34	1.47
D	1.08	1.17	1.27	1.38
C	1.07	1.14	1.22	1.31
B	1.05	1.11	1.17	1.23
S	1.07	1.16	1.24	1.34

Both tables of factors given above apply only where all the stress ranges exceed the value of σ_0 calculated in accordance with **A.2**, from the formula given in **A.1**, with the appropriate value of d.

As the value of σ_0 will increase with a decrease in the number of standard deviations, these factors are conservative when applied to the simplified methods of **8.2** and **8.3**, with certain stress ranges below the values of σ_0 calculated as given above. Where fatigue damage is predicted by the methods of **8.4, 9.2** and **9.3**, with numbers of standard deviations other than two, the appropriate values of σ_0 should be determined and used.



NOTE. Only that portion of this figure shown as a full line is based on experimental evidence.

Figure 16. Typical $\sigma_r - N$ relationship

Appendix B
Cycle counting by the reservoir method

B.1 General. The purpose of cycle counting is to reduce an irregular series of stress fluctuations to a simple list of stress ranges. The method given in this appendix, and shown in the figure below, is suitable when dealing with short stress histories, such as those produced by individual loading events. It consists of imagining a plot of the graph of each individual stress history as a cross section of a reservoir, which is successively drained from each low point, counting one cycle for each draining operation. The result, after many repetitions of the loading event, will be the same as that obtainable by the rainflow method (see the footnote to **9.3.3**).

B.2 Method

B.2.1 Derive the peak and trough values of the stress history, due to one loading event, in accordance with **8.3.2.1** (c). Sketch the history due to two successive occurrences of this loading event. The calculated values of peak and trough stresses may be joined with straight lines if desired. Mark the highest peak of stress in each occurrence. If there are two or more equal highest peaks in one history, mark only the first such peak in each occurrence.

B.2.2 Join the two marked points and consider only that part of the plot which falls below this line, like the section of a full reservoir.

B.2.3 Drain the reservoir from the lowest point leaving the water that cannot escape. If there are two or more equal lowest points the drainage may be from any one of them. List one cycle having a stress range σ_{v1} equal to the vertical height of water drained.

B.2.4 Repeat **B.2.3** successively with each remaining body of water until the whole reservoir is emptied, listing one cycle at each draining operation.

B.2.5 Compile the final list which contains all the individual stress ranges in descending order of magnitude σ_{v1}, σ_{v2} etc. Where two or more cycles of equal stress range are recorded, list them separately.

B.2.6 For non-welded details only, a horizontal line representing zero stress should be plotted and those parts of the stress ranges in the compression zone modified as in **6.1.3**

Appendix C
Derivation of standard highway bridge fatigue; loading and methods of use

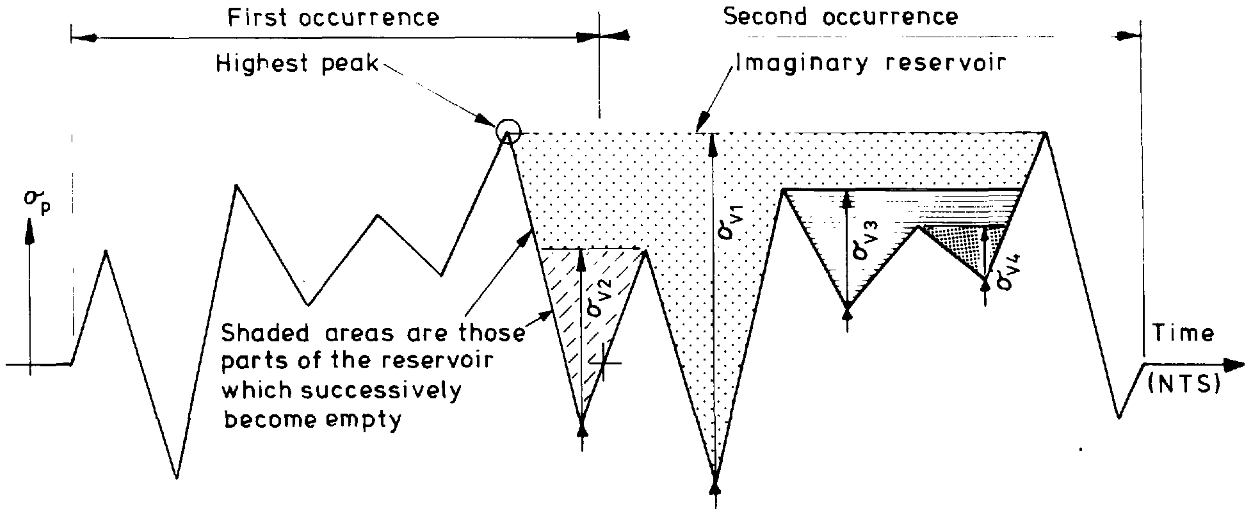
C.1 Standard loading

C.1.1 ***Standard load spectrum.*** (See **7.2.2.1**.) Table 11 shows a 25 band spectrum of commercial vehicle weights, axle arrangements and frequencies of occurrence, which is typical of the full range of commercial traffic on a trunk road in the UK. Other relatively uncommon vehicle types have been included in the types which are nearest to them on the basis of equivalent damage. Private cars and light vans below 15 kN unladen weight are not included as their contribution to fatigue damage is negligible. Table 11 includes vehicles operating under both the Motor Vehicles (Construction and Use) Regulations and the Motor Vehicle (Authorization of Special Types) General Order.

The proportions of the various types of vehicles of the spectrum have been taken from sample traffic counts. To allow for variations in the loads being carried by similar vehicles, the various types have been divided into heavy, medium and light loading groups (H, M and L). The axle loads have been averaged from weighbridge records of moving traffic taken between 1971 and 1974.

C.1.2. ***Standard fatigue vehicle.*** (See **7.2.2.2**.) The proportions of the damage caused by individual vehicle types, compared with the total damage by all vehicles, varies between the limits shown in table 12. The standard fatigue vehicle has been devised to represent the most damaging group which, for the majority of detail classes and influence line lengths, is group 4A-H.

The axle spacings of the standard fatigue vehicle are the same as those for the short HB vehicle (see Part 2) and the 80 kN axle weight is equivalent to the standard 18 000 lb axle, which has been used for some years as the datum axle in the fatigue design of road pavements. As the damage done by the single tyred wheels on vehicles listed in table 11 is normally less than 4 % of the damage done by the double tyred wheels, the standard fatigue vehicle with double tyred wheels gives an adequate representation of the wheel damage for all types of vehicles.



C.1.3 ***Standard lane flows.*** (See **7.2.2.3.**) The annual flows of commercial vehicles $n_c \times 10^6$ given in table 1 are *based on the design capacity of the particular road type,* as specified by the Department of Transport. The proportion of commercial traffic above 15 kN unladen weight has been taken to be 20 % of all traffic for all-purpose roads and 25 % for motorways.

C.1.4 ***Multiple paths.*** (See **7.2.3.2.**) A significant reduction in assessed damage can be achieved by the consideration of multiple paths when the transverse influence line profile departs rapidly from the value of the mean path ordinate. Figure 17 shows a histogram of occurrence of proportion factors for 100 mm wide intervals of carriageway width, derived from observations of traffic patterns.

The standard fatigue vehicle centre line should be traversed along the centre line of each 100 mm strip and a stress spectrum obtained for each strip in accordance with **8.3.2**, with the annual flow of vehicles in any strip being derived from the appropriate proportion (from figure 17) $\times n_c \times 10^6$ (taken from table 1). In cases where the transverse influence line changes sign across the 1300 mm histogram width, an alternating succession of vehicles along paths of opposing sign would produce a reduced number of cycles of enhanced stress range. However, trial calculations, which take into account the probability of the occurrence of alternating sequences, have shown that the increase in damage is not significant and may be neglected.

C.2 Derivation of load spectra based on the standard fatigue vehicle. The principle behind the assessment procedures of **8.2** and **8.3** is that each commercial vehicle in table 11 is represented by one vehicle of the same gross weight but with axle configurations identical to those of the standard fatigue vehicle. The resulting load spectrum shown in table 13, where the various vehicle group weights are expressed as a proportion of the standard fatigue vehicle gross weight, has been used in the derivation of the procedures of **8.2** and **8.3** instead of the complete vehicle spectrum of table 11. Table 13 has been derived using a datum influence line which is 25 m long and rectangular in shape. This has the effect of limiting the gross weight of the 18 GT group so that static design stresses are not exceeded.

For values of L less than 25 m the design spectrum becomes increasingly influenced by bogie and axle spacings and weights. The weighbridge records, from which table 11 is derived (see **C.1.1**), show a wider variation in axle loads than in gross vehicle loads, and an increase of 10 % (indicated by trial calculations) over the individual values of table 11 has been allowed in deriving the spectrum of axle weights of table 14. When L is greater than 25 m, the design spectrum for individual vehicles will be proportional to the gross vehicle weight spectrum. However, as L increases, account should be taken of the contribution to damage done by two or more vehicles acting simultaneously. All these effects have been catered for by means of a simple adjustment factor K_F (see figure 11) which is described in more detail in **C.4**.

C.3 Assessment charts

C.3.1 ***Limiting stress ranges*** σ_H. (See figure 8 and **8.2.**) The graphs of allowable stress ranges in figure 8, which are based on a 120 year design life and the appropriate traffic flows from table 1, have been derived with the aid of the damage chart of figure 10 (see **C.3.2**). The worst assumptions have been made about the stress contributions from each lane and about the allocation of traffic flows between lanes so that the results are always on the safe side. The adjustment factor K_F, from figure 11, has also been included in the derivation of figure 8.

C.3.2 ***Damage factors*** d_{120}. (See figure 10 and **8.3.**) The damage chart of figure 10 is based on the cumulative fatigue damage caused by the design spectrum which is obtained from the passage of the vehicles represented by the gross weight spectrum of table 13 over an influence line base length of 25 m. It assumes 10^6 cycles of stress per year for the 120 year design life and the damage, assessed by Miner's summation, is given in relation to the stress range caused by the passage of a standard fatigue vehicle.

The adjustment factor K_F should be applied to the results from figure 10 in order to allow for influence line base lengths of less than 25 m and for the effects of multiple vehicles. The effects of numbers of vehicles other than the 120 million assumed are allowed for by multiplying the lifetime damage from figure 10 by n_c.

C.4 Adjustment factors

C.4.1 ***General.*** Because the assessment charts of figures 8 and 10 are based on the passage of single vehicles and also on an assumed influence line base length of 25 m, adjustment factors are necessary to allow for shorter base lengths and for the effects of combinations of vehicles. These factors, which are labelled X, Y and Z in figure 18, make up the Miner's summation adjustment factor K_F given in figure 11. Factor X is described in **C.4.2** and factors Y and Z in **C.4.3**. It should be noted that the values of these factors cannot be determined precisely but that the values of K_F given are sufficiently accurate for design purposes.

The adjustment factor K_F has been included in the derivation of figure 8 but should be applied explicitly when figure 10 is used. For the assessment method of **8.2** there is an additional adjustment, which may be made in the case of class S details, where the design is based on reduced HB loading and this is described in **C.4.4.2**.

C.4.2 ***Influence line base length less than*** 25 m. The X component of the K_F adjustment factor has been obtained by comparing the stress histories for a selection of the most damaging vehicles in table 11 with those derived by representing the vehicles by standard fatigue vehicles with the same gross weights. The two groups of vehicles were traversed across 11 different shapes of influence line, each with a range of loop lengths, and the Miner's summation for damage calculated using the $\sigma_r - N$ relationships for all the the detail classes except S. The X factor is the average value of the ratio of damage due to the table 11 vehicles to that due to the equivalent standard fatigue vehicles. The scatter in results between different influence line shapes was found to be acceptable for design purposes.

C.4.3 ***Multiple vehicle effects.*** During normal conditions of traffic flow, instances will occur when more than one vehicle will contribute to the stress in a detail at any particular time and the stress may be increased above that due to either vehicle alone. These multiple vehicle effects may be sub-divided according to the following :

(a) more than one vehicle in the same lane simultaneously ;

(b) vehicles in different lanes simultaneously causing stress of the same sign ;

(c) vehicles in different lanes in alternating sequence causing stresses of opposite signs and so increasing the stress range.

The chance that damaging vehicles will be sufficiently close to each other, either in the same or different lanes, has been assessed on a probability basis. Correction factors have been derived from the comparison between the damage from the combinations of vehicles and the damage due to such vehicles on their own.

Table 11. Typical commercial vehicle groups

Total axles	Chassis type	Average axle spacings, m	Loading Group	Total weight, kN	Axle loads, kN	No. in each group per million commercial vehicles	Vehicle designation
18	Girder Trailer and 2 tractors	4.5 1.5 4.0 7.5 13.5 7.5 4.0 4.5 1.5	H	3 680	80 160 160 240 (6 no.) 240 (6 no.) 80 160 160	10	18GT-H
			M	1 520	80 160 160 60 (6 no.) 60 (6 no.) 80 160 160	30	18GT-M
9	Tray Trailer and tractor	4.5 1.5 4.0 1.5 1.5 1.5 1.5 1.5	H	1 610	70 140 140 210 210 210 210 210 210	20	9TT-H
			M	750	50 110 110 80 80 80 80 80 80	40	9TT-M
7	Girder Trailer and tractor	4.5 1.5 4.0 2.0 10.0 2.0	H	1 310	70 140 140 240 240 240 240	30	7GT-H
			M	680	60 130 130 90 90 90 90	70	7GT-M
	Articulated	3.0 1.5 9.5 1.5 1.5 1.5	H	790	70 100 100 130 130 130 130	20	7A-H
5	Articulated	3.0 1.5 13.5 1.5	H	630	70 130 130 150 150	280	5A-H
		3.0 1.5 5.0 1.5	M	360	60 70 70 80 80	14 500	5A-M
			L	250	40 45 45 60 60	15 000	5A-L
4	Articulated	3.0 6.5 1.5	H	335	55 100 90 90	90 000	4A-H
			M	260	45 85 65 65	90 000	4A-M
			L	145	35 50 30 30	90 000	4A-L
	Rigid	1.5 3.5 1.5	H	280	50 50 90 90	15 000	4R-H
			M	240	40 40 80 80	15 000	4R-M
			L	120	20 20 40 40	15 000	4R-L
3	Articulated	2.5 4.5	H	215	45 85 85	30 000	3A-H
			M	140	30 55 55	30 000	3A-M
			L	90	20 35 35	30 000	3A-L
	Rigid	4.0 1.5	H	240	60 90 90	15 000	3R-H
			M	195	55 70 70	15 000	3R-M
			L	120	40 40 40	15 000	3R-L
2	Rigid	4.0	H	135	50 85	170 000	2R-H
			M	65	30 35	170 000	2R-M
			L	30	15 15	180 000	2R-L

Key. ◉ Standard axle, 4 tyre, 1.8 m track ◎ Steering axle, 2 tyre, 2.0 m track **O** Special axle, 2 to 8 tyres, up to 3.4 m outer track

The effect given in (a) above is allowed for by means of factor Y (see figure 18) which shows that the effect is negligible for L less than 50 m but increases with increasing L due to the greater probability of having two vehicles simultaneously on the same influence line base length.

The effect given in (b) above is allowed for by means of factor Z (see figure 18) which is given for several values of factor K_B (see figure 11), i.e. the ratio between the major stress ranges produced by vehicles travelling separately in the two lanes producing the most severe stress effect. Combinations of vehicles in more than two lanes do not generally increase the total damage significantly.

The effect given in (c) above is taken account of by the additional combined stress history, referred to as case 2 in figure 9. In this case the effect of vehicles travelling simultaneously in two lanes may be neglected and so factor Z can be taken as zero by making K_B equal to zero.

C.4.4 ***Class S details***

C.4.4.1 ***General.*** Trial calculations indicate that for class S details, significant variations occur in the values of X (see **C.4.2**) both with variations in L and with variations in influence line shapes. Hence the adjustment factor X does not apply to class S details and the assessment procedure of **8.3** cannot be applied.

C.4.4.2 ***Reduced HB design loads.*** (See **8.2.3** (c).) Table 12 shows that the heaviest abnormal load vehicles of table 11 contribute a very small percentage of the total damage for all detail classes except class S. Hence no relaxation is provided in the assessment procedures of **8.2** and **8.3** for these other classes when the bridge is subject to reduced values of HB loading.

However, these heavy vehicles do contribute the greater proportion of the total damage for class S details and hence **8.2.3** (c) provides reduction factors where the bridge is designed to carry less than 45 units of HB loading.

Table 12. Proportional damage from individual groups of typical commercial vehicles

Detail class	D to G, F2 and W (m = 3.0)		C (m = 3.5)		B (m = 4.0)		S (m = 8.0)	
Group*	L = 1.5	L = 25	L = 1.5	L = 25	L = 1.5	L = 25	L = 1.5	L = 25
18 GT-H	1 %	2 %	1 %	5 %	2 %	11 %	27 %	80 %
9 TT-H	1 %	2 %	1 %	4 %	1 %	7 %	9 %	15.2 %
18 GT-M and 9TT-M to 5A-H	2 %	4 %	3 %	6 %	4 %	9 %	32 %	4.72 %
5A-M and L	4 %	14 %	4 %	14 %	3 %	13 %	1 %	0.02 %
4A-H, M and L	57 %	67 %	57 %	63 %	57 %	54 %	22 %	0.057 %
4 R-H to 2 R-L	35 %	11 %	34 %	8 %	33 %	6 %	9 %	0.003 %

*See table 11.

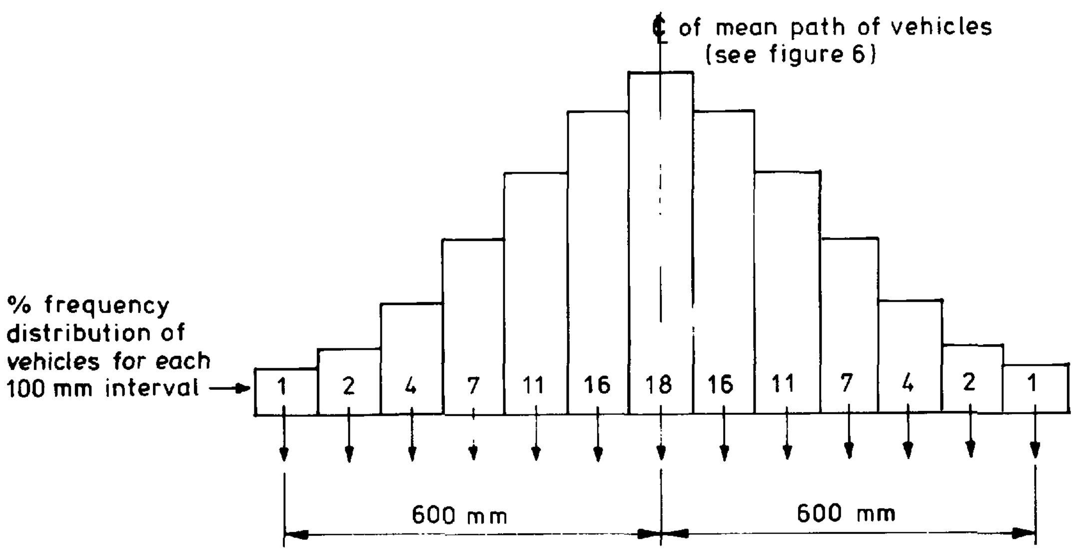


Figure 17. Multiple paths

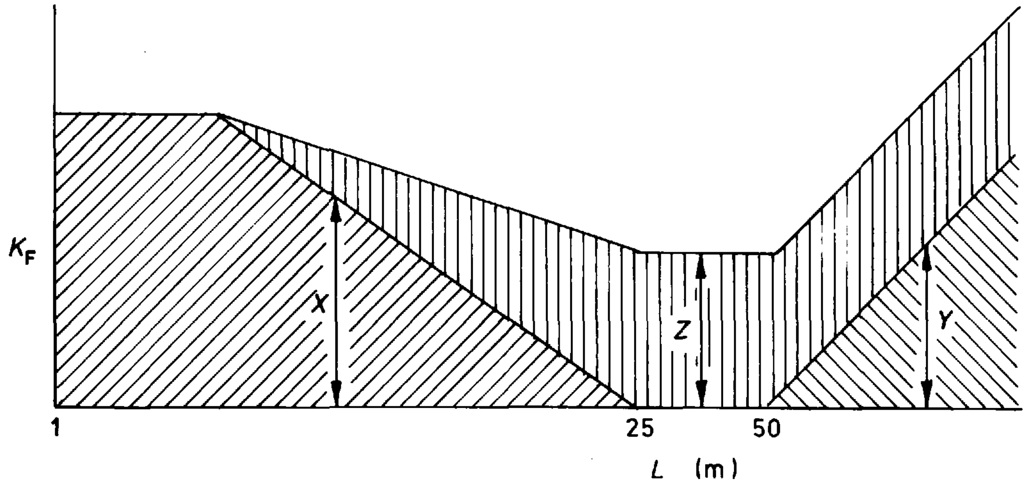


Figure 18. Typical Miner's summation adjustment curve

Table 13. Typical commercial vehicle gross weight spectrum

Vehicle designation	Proportion of standard fatigue vehicle gross weight	Proportion of total vehicles
18GT-H	6.75	0.000 01
18GT-M	2.38	0.000 03
9TT-H	5.03	0.000 02
9TT-M	2.34	0.000 04
7GT-H	4.09	0.000 03
7GT-M	2.13	0.000 07
7A-H	2.47	0.000 02
5A-H	1.97	0.000 28
5A-M	1.13	0.014 50
5A-L	0.78	0.015
4A-H	1.05	0.090
4A-M	0.81	0.090
4A-L	0.45	0.090
4R-H	0.88	0.015
4R-M	0.75	0.015
4R-L	0.38	0.015
3A-H	0.67	0.030
3A-M	0.44	0.030
3A-L	0.28	0.030
3R-H	0.75	0.015
3R-M	0.61	0.015
3R-L	0.38	0.015
2R-H	0.42	0.170
2R-M	0.20	0.170
2R-L	0.09	0.180
	Total	1.0

NOTE. This table is based on L = 25 m (rectangular loop).

Table 14. Typical commercial vehicle axle weight spectrum

Total axle weight	Total number of axles for 10^6 vehicles
264	240
231	120
176	160
165	560
154	100
143	780
121	80
110	90 040
99	240 280
93	320 000
88	59 320
77	59 350
71	180 000
66	59 930
61	165 000
55	290 040
49	150 000
44	120 000
39	320 000
33	380 000
22	60 000
17	360 000
Total	2 856 000 axles for 10^6 vehicles

NOTE 1. This table is based on $L \leq 1.5$ m.

NOTE 2. These values include the 10 % increase referred to in **C.2**.

Appendix D

Examples of fatigue assessment of highway bridges by simplified methods

D.1 General. D.2 to **D.4** give examples of typical calculations for fatigue assessment.

For all detail classes, **8.2** and figure 8 provide a limiting stress range σ_H which will always be safe where standard loading conditions are applicable but which may be too conservative in some cases. The σ_H values given in figure 8 thus provide a simple check which is very suitable for initial design purposes.

Alternatively, **8.3** provides a more precise method for detail classes B to G and F2 and W leading to a life prediction (to 97.7% probability of survival) and indicating the extent of required changes in the detail when the predicted life is too short.

The basic assessment procedures for a steelwork detail are illustrated in **D.2**, while the application of a combined stress history is illustrated in **D.3** (see **8.3.2.1** (c)). A typical procedure for shear connectors is given in **D.4**.

D.2 Example using the basic assessment procedures for a steelwork detail

D.2.1 ***Type of bridge.*** A three span (50 m/75 m/45 m) twin girder highway bridge that carries a dual two lane motorway and is designed to carry standard UK loading.

D.2.2 ***Details of the problem.*** These assess the fatigue resistance of the main girder bottom flanges at the mid-span of the main span, which is adjacent to the transverse weld of a vertical stiffener, given that analysis, in accordance with **8.2.2.1** (a) and (b), produces principal maximum and minimum stress values of 17.8 N/mm² tensile and 8.0 N/mm² compressive respectively, under the standard fatigue vehicle, with the vehicle positioned in the same lane.

D.2.3 ***Classification.*** The potential fatigue crack should be classified as F (from type 2.9 of table 17 (b)), provided that the weld end is not within 10 mm of the flange toe.

D.2.4 ***Assessment using 8.2***

From **D.2.2**, $\sigma_{p\,max} = 17.8$ N/mm²

$\sigma_{p\,min} = -8.0$ N/mm²

Hence $\sigma_{v\,max} = 17.8 - (-8.0) = 25.8$ N/mm² (see **8.2.2.1** (c)).

For a dual two lane motorway with $L = 75$ m and a class F detail,

$\sigma_H = 20.5$ N/mm² (see figure 8(b)).

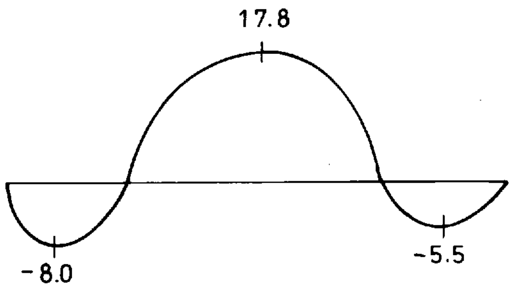
Hence $\sigma_{v\,max}$ exceeds σ_H and adequate fatigue life is not demonstrated (see **8.2.2.3**). By reference to **8.2.2.4** either

(a) the procedure of **8.3** may be used, since $\sigma_{v\,max} < 1.55\sigma_H$

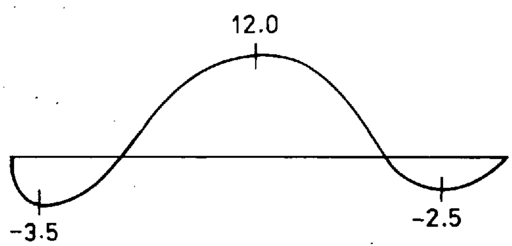
or

(b) the detail should be strengthened to reduce $\sigma_{v\,max}$ or improved to a higher class.

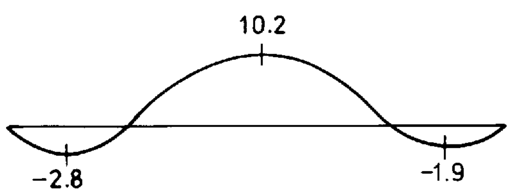
D.2.5 ***Assessment using 8.3.*** Given that the stress histories for each loading event (passage of the standard fatigue vehicle in each lane) are as follows:



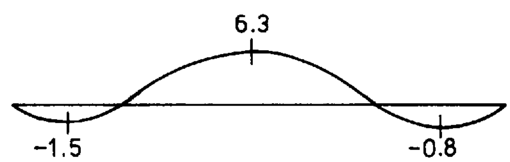
1st carriageway, slow lane



1st carriageway, adjacent lane

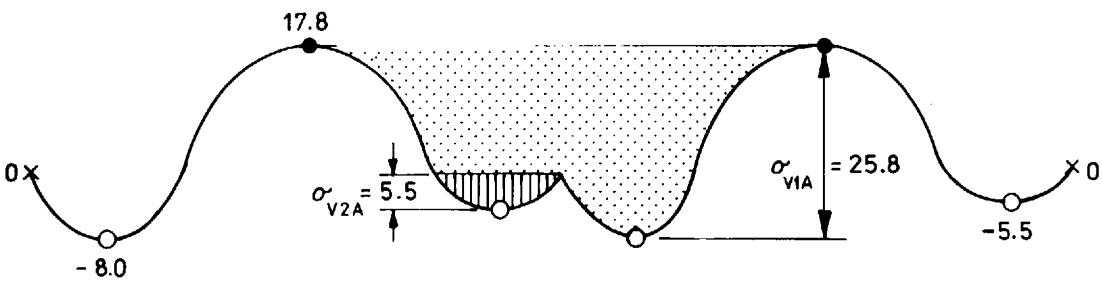


2nd carriageway, adjacent lane



2nd carriageway, slow lane

From figure 9, in the first carriageway, the slow lane will be designated lane A, since it has the greatest stress range (i.e. 17.8 − (−8.0)   25.8). The reservoir method (see appendix B) may be used to determine the values of σ_v as shown below:



The values of σ_v for the other lanes may be determined in a similar way.

This assessment procedure can be tabulated as follows:

Lane	Reference	Cycle number	σ_p* Peak	Trough	Range σ_v	d_{120}† Class F	$\bar{n}_c$‡	Damage $\bar{n}_c d_{120}$
1st Slow	A	1	17.8	—8.0	25.8 (σ_{v1A})	0.32	1.5	0.48
		2	0	—5.5	5.5	Neglect (<0.001)		—
1st Adj.	B	1	12.0	—3.5	15.5 (σ_{v1B})	0.032	1.0	0.03
		2	0	—2.5	2.5	Neglect (<0.001)		—
2nd Adj.	C	1	10.2	—2.8	13.0	0.015	1.0	0.02
		2	0	—1.9	1.9	Neglect (<0.001)		—
2nd Slow	D	1	6.3	—1.5	7.8	0.001	1.5	0.00
		2	0	—0.8	0.8	Neglect (<0.001)		—
						$\Sigma\bar{n}_c\, d_{120} = 0.53$		

*See appendix B.
†See figure 10.
‡See table 1 and figure 9.

Adjustment factor K_F (see figure 11)

$K_B = 15.5/25.8 = 0.6$

$L = 75$ m

hence $K_F = 1.59$

Total damage $= \Sigma K_F \bar{n}_c d_{120} = 1.59 \times 0.53 = 0.843$

Estimated life $= 120/0.843 = 142$ years (see **8.3.2.1** (h)) which is greater than the specified design life of 120 years and so the detail may be regarded as satisfactory.

D.2.6 *Comments.* The method of **8.3** predicts a fatigue life that is in excess of the design life and hence a reduction in cross section could be tolerated. By reference to **8.3.2.2**, the stress range could be increased by $\left(\frac{142}{120}\right)^{0.25}$ i.e. by 1.04.

In contrast, the procedure of **8.2** produces a limiting value of σ_H equal to $20.5/25.8 = 0.79$ times the applied maximum stress range, thus demonstrating the conservatism of this method. Nevertheless, the simplicity of determination of σ_H is such that the method should, in many cases, prove a useful 'first stage' as an alternative to the more precise method of **8.3**, or the lengthier procedure of **8.4**.

D.3 Example of the application of a combined stress history

D.3.1 ***Type of bridge.*** A highway bridge that carries a two lane, single carriageway, all-purpose road and is designed to carry standard UK loading, but is deemed to be subject to 1.2×10^6 commercial vehicles per year, in each lane, for a design life of 60 years.

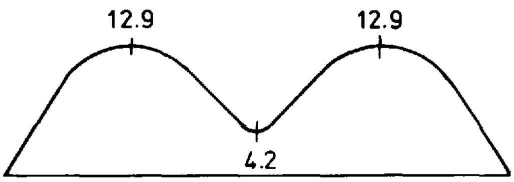
D.3.2 ***Details of the problem.*** Given that the analysis predicts, as shown in **D.3.5**, the forces in a transverse bracing member due to the standard fatigue vehicle in alternate lanes and given that the influence line base length is 8 m, determine an area of cross section for the member, to provide adequate fatigue resistance at the lap-welded bracing/gusset connection.

D.3.3 ***Classification.*** The potential fatigue crack should be classfied as G (from type 2.11 of table 17(b)) since the weld will be at the edge of the member (see also figure 1).

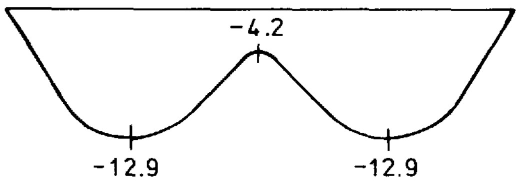
D.3.4 ***Assessment using 8.2.*** The information given in **D.3.1** and **D.3.2** does not comply with (b) and (d) of **8.2.1** and therefore **8.2** is not applicable.

D.3.5 ***Assessment using 8.3.*** Assume a cross-sectional area of 2200 mm^2 and determine the stress histories for passage of the standard fatigue vehicle in each lane.

Given that these are as follows:

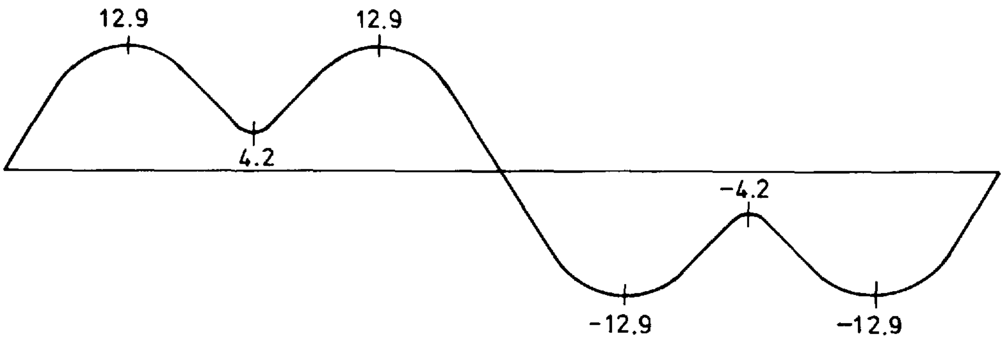


Lane A



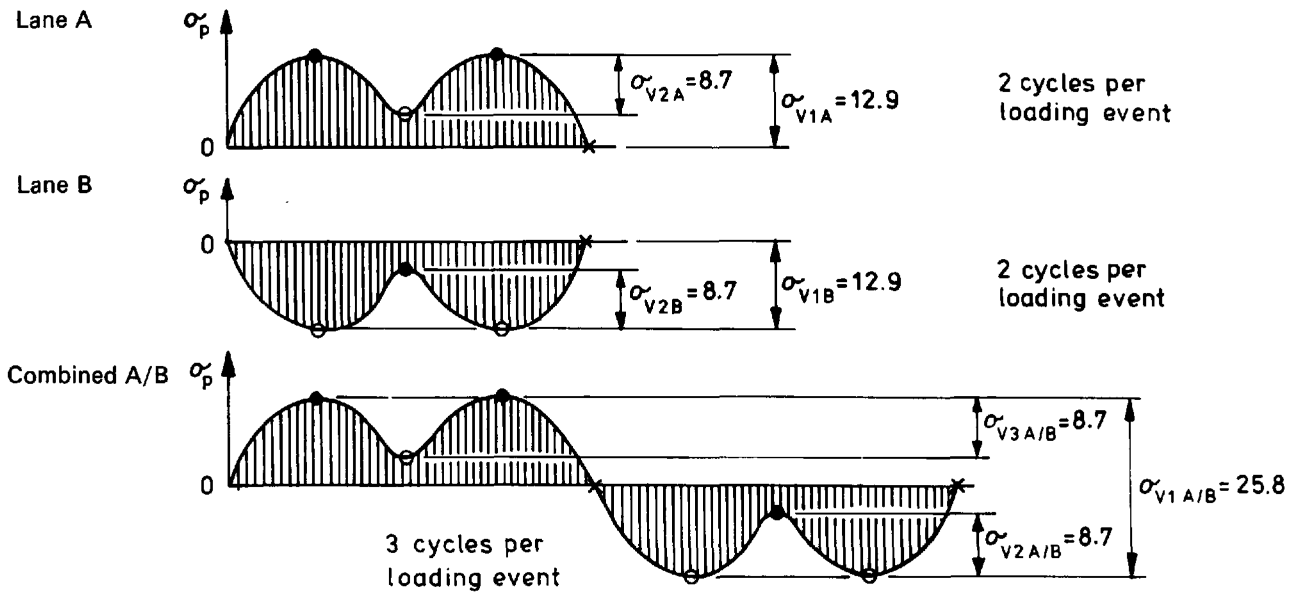
Lane B

Derive the combined stress history (see **8.3.2.1** (c) and case 2 of figure 9).



Lane A/B

The stress ranges σ_v may be determined by the reservoir method (see appendix B) and the resulting stress spectra (see figure 9) for the individual and combined lane stress histories will be as shown below:



The assessment procedure can be tabulated as follows:

Lane	Cycle number	σ_p* Peak	Trough	Range σ_v	d_{120}† Class G	n_c‡	Damage $\bar{n}_c\, d_{120}$
A	1	12.9	0	12.9	0.055	$\frac{1.2^2}{1.2 - 1.2} = 0.6$	0.03
	2	12.9	4.2	8.7	0.01		0.01
B	1	0	—12.9	12.9	0.055	$\frac{1.2^2}{1.2 - 1.2} = 0.6$	0.03
	2	—4.2	—12.9	8.7	0.01		0.01
						$\Sigma\bar{n}_c\, d_{120}$	0.08
A/B	1	12.9	—12.9	25.8	1.40	$\frac{1.2^2}{1.2 - 1.2} = 0.6$	0.84
	2	—4.2	—12.9	8.7	0.01		0.01
	3	12.9	4.2	8.7	0.01		0.01
						$\Sigma\bar{n}_c\, d_{120}$	0.86

*See appendix B.
†See figure 10.
‡See figure 9.

Adjustment factor K_F (see figure 11) for separate histories

$K_B = 12.9/12.9 = 1.0$

$L = 8$ m

hence $K_F = 1.81$

for a combined history

$K_B = 0$

$L = 8$ m

hence $K_F = 1.47$

Total damage $= \Sigma K_F \bar{n}_c d_{120}$

$= 1.81 \times 0.08 + 1.47 \times 0.86$

$= 1.41$

Estimated life = 120/1.41 = 85 years (see **8.3.2.** (h)) which is greater than the specified design life of 60 years and so the detail may be regarded as satisfactory.

D.3.6 *Comments.* The estimated life is in excess of the 60 year design life and hence a reduction in area is allowable.

The reduction factor will be $\left(\frac{60}{85}\right)^{0.25}$ i.e. 0.917 (see **8.3.2.2**)

and hence an acceptable area will be 2017 mm². Repeat the assessment with an amended initial assumption of the area.

NOTE. Iteration provides a solution of 2027 mm².

D.4 Example of a typical procedure for shear connectors

D.4.1 ***Type of bridge.*** A 30 m simply supported composite highway bridge that carries a dual carriageway all-purpose road and is subject to standard UK loading, but which is limited to 37.5 units of HB loading.

D.4.2 ***Details of the problem.*** To investigate the fatigue capacity of the attached shear connectors (which are in accordance with Part 5) in a normal density reinforced concrete deck slab of flat soffit.

D.4.3 ***Classification.*** Since the connectors are in accordance with Part 5, they also comply with **6.4.1** and the weld throat stresses may be calculated according to **6.4.2** or **6.4.3**.

The potential fatigue crack should be classified as S (from type 3.12 of table 17(c).

D.4.4 ***Assessment using 8.2.*** Assuming, for this example, that the shape of the shear force influence line of the girder under consideration is similar to that of a single simply supported girder, the value of L will lie between 15 m, for connectors at mid-span, and 30 m for connectors at the ends of the member.

Hence, for a dual carriageway all-purpose road and class S (see figure 8(b), with the 1.3 factor allowed for 37.5 units of HB loading (see **8.2.3**(c)), $\sigma_H = 46 \times 1.3 = 59.8$ N/mm² for connectors at mid-span, and $\sigma_H = 40 \times 1.3 = 52$ N/mm² for connectors at the ends.

The above values may be checked against $\sigma_{v\,max}$ as described in **8.2.2** and illustrated in **D.2** or, alternatively, for preliminary design purposes in this particular type of example it may be noted that:

at mid-span $\sigma_{p\,max}$ will equal $-\sigma_{p\,min}$
at the ends $\sigma_{p\,min}$ will equal zero.

Hence, the limiting value of $\sigma_{p\,max}$ may be determined as:

29.9 N/mm² for connectors at mid-span and
52 N/mm² for connectors at the ends.

Thus for stud connectors (see **6.4.2**) the maximum allowable shear load per stud under loading from the standard fatigue vehicle, positioned in accordance with **7.2.3** may be expressed as:

$29.9 P_u/425 = 0.070 P_u$ kN at mid-span and
$52.0 P_u/425 = 0.122 P_u$ kN at the ends

where
P_u is the nominal strength of the stud from Part 5.

Similarly, for bar or channel connectors, the maximum allowable shear load per connector under loading from the standard fatigue vehicle, positioned in accordance with **7.2.3**, may be expressed as:

$29.9 \times A_1 \times 10^{-3}$ kN at mid-span and
$52.0 \times A_1 \times 10^{-3}$ kN at the ends.

where
A_1 is the effective weld throat area in mm² for the particular type of connector, obtained from **6.4.3.1**.

D.4.5 ***Comments.*** It is not possible to use **8.3** for the assessment of shear connectors as the damage chart (see figure 10) does not include factors for class S details.

Appendix E

Derivation of standard railway load spectra

E.1 RU loading. The load spectra given in table 2 have been based on the typical trains shown in figure 19. The numbers of these trains, assumed for the three broad traffic types, are shown in table 15 together with the make-up of the total annual tonnages. These spectra will cover most traffic of this type running on lines in Europe.

For further information on the derivation of the spectra, the following reports published by the Office for Research and Experiments of the International Union of Railways should be consulted:

Report ORE D128/RP5
Report ORE D128/RP6
Report ORE D128/RP7

E.2 RL loading. The load spectra given in table 3 have been based on the typical trains shown in figure 20. The numbers of each type of train assumed for the standard spectra, together with the make-up of the total annual tonnages, are shown in table 16.

Table 15. RU loading: annual traffic tonnage for standard traffic types

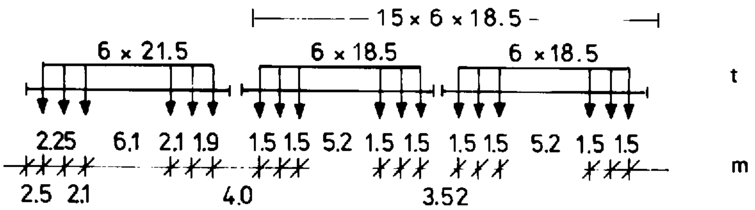
Traffic type	Train type*	Train weight, tonnes	Number of trains per annum	Total annual tonnage, tonnes × 10⁶
Heavy	7	1120	4 821	5.40
	8	1120	7 232	8.10
	9	852	15 845	13.50
			Total	27.00
Medium	5	600	22 500	13.50
	7	1120	2 411	2.70
	8	1120	6 027	6.75
	1	1794	2 257	4.05
			Total	27.00
Light	1	1794	752	1.35
	2	372	14 516	5.40
	3	344	23 546	8.10
	4	172	47 093	8.10
	5	600	4 500	2.70
	6	572	2 360	1.35
			Total	27.00

*See figure 19.

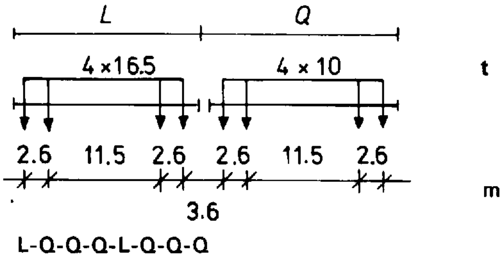
Table 16. RL loading: annual traffic tonnage and composition of standard traffic mix

Train type*	Train weight, tonnes	Number of trains per annum	Total annual tonnage, tonnes × 10⁶
1	246	11 545	2.84
2	253	54 032	13.67
3	280	9 786	2.74
4	203	6 453	1.31
5	209	26 986	5.64
6	231	3 463	0.80
		Total	27.00

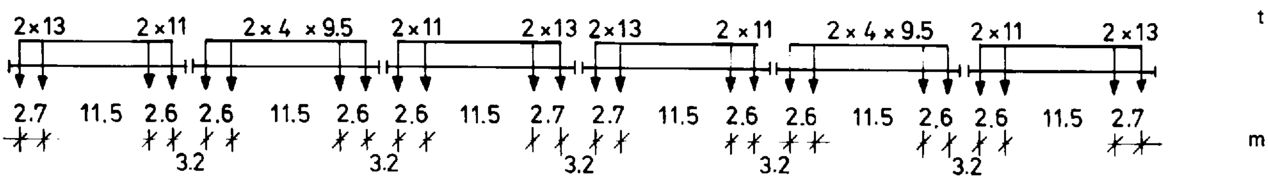
*See figure 20.



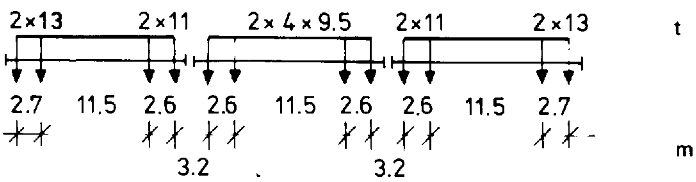
1 Steel train total load = 1794 t V = 80 km/h



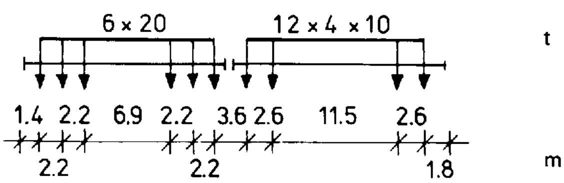
2 Electric multiple unit total load = 372 t V = 145 km/h



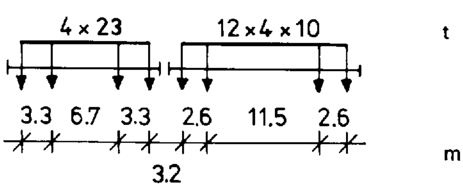
3 Southern Region suburban total load = 344 t V = 145 km/h



4 Southern Region suburban total load = 172 t V = 145 km/h

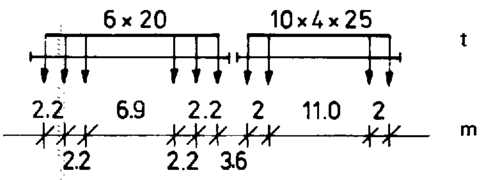


5 Diesel hauled passenger train total load = 600 t V = 160 km/h

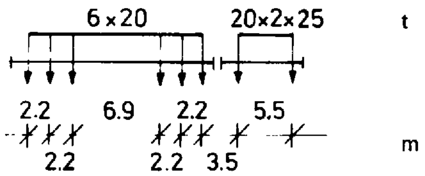


6 Electric hauled passenger train total load = 572 t V = 160 km/h

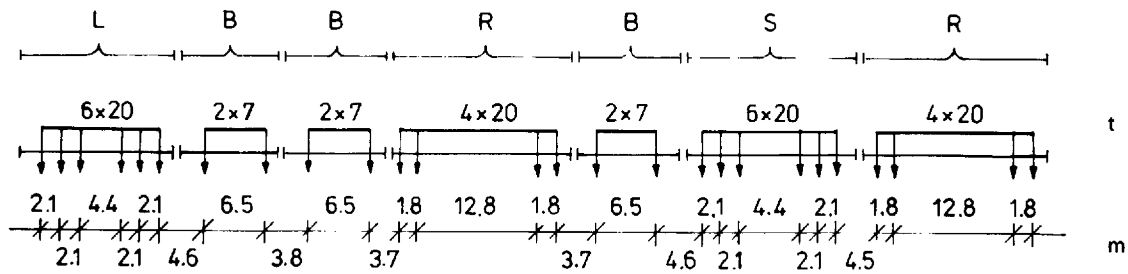
Figure 19. Trains included in table 2 spectra



7 Heavy freight total load = 1120 t V = 72 km/h for medium traffic and 120 km/h for heavy traffic



8 Heavy freight total load = 1120 t V = 72 km/h for medium traffic and 120 km/h for heavy traffic

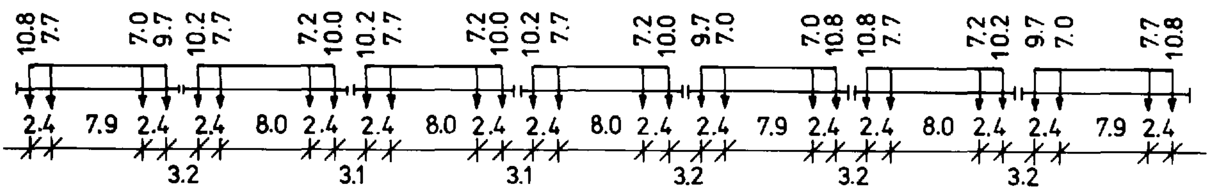


L-B-B-R-B-B-B-B-B-B-B-B-B-B-S-R-B-B-R-B-B-B-B-S

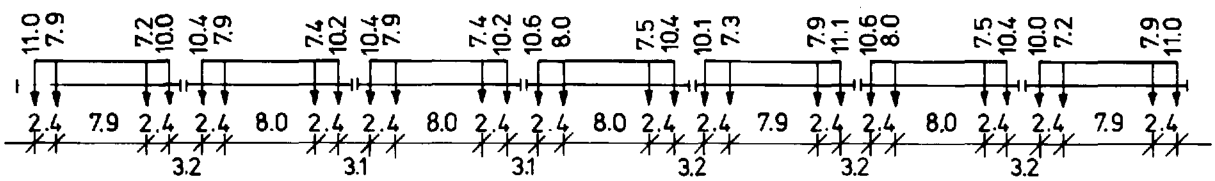
9 Mixed freight total load = 852 t V = 120 km/h

NOTE. In deriving the table 2 spectra, impact effects were calculated in accordance with the recommendations of Leaflet 776-1 R, published by the International Union of Railways (UIC), 14 Rue Jean-Ray F, 75015 Paris.

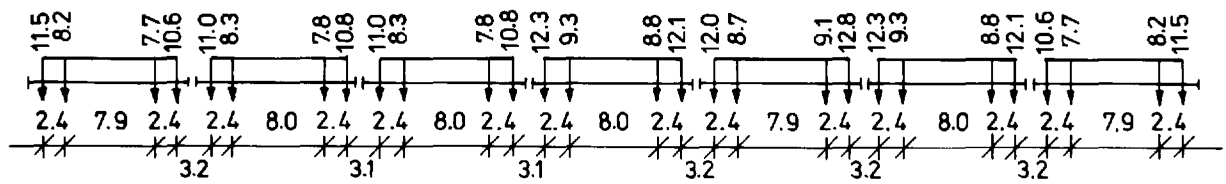
Figure 19. (*Concluded*)



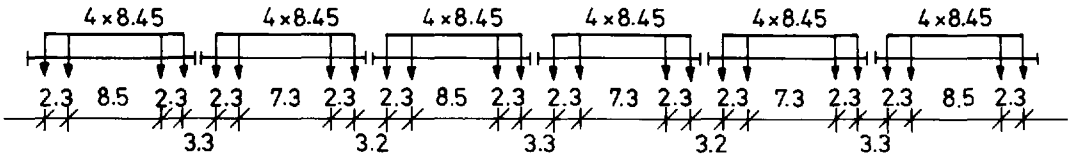
1 total load = 246 t



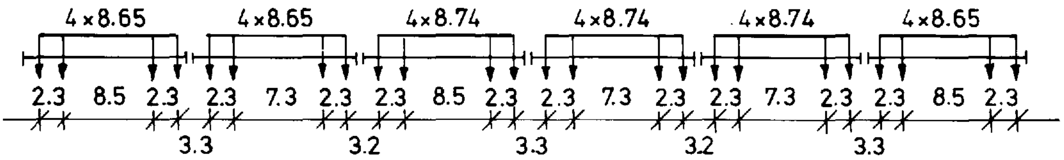
2 total load = 253 t



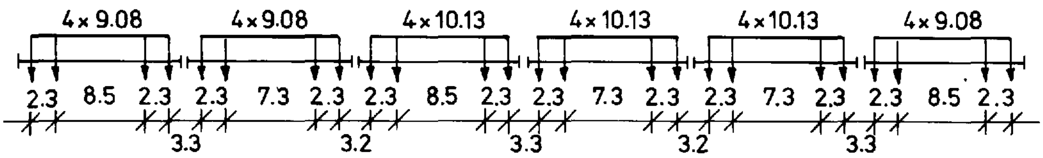
3 total load = 280 t



4 total load = 203 t



5 total load = 209 t



6 total load = 231 t

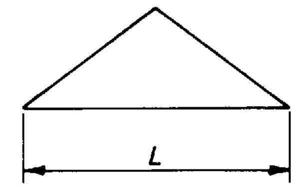
NOTE. In deriving the table 3 spectra, an impact of 30 % was taken for all trains and all spans

Figure 20. Trains included in table 3 spectra.

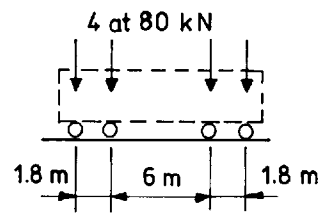
Appendix F

Examples of stress histories and cycle counting procedure

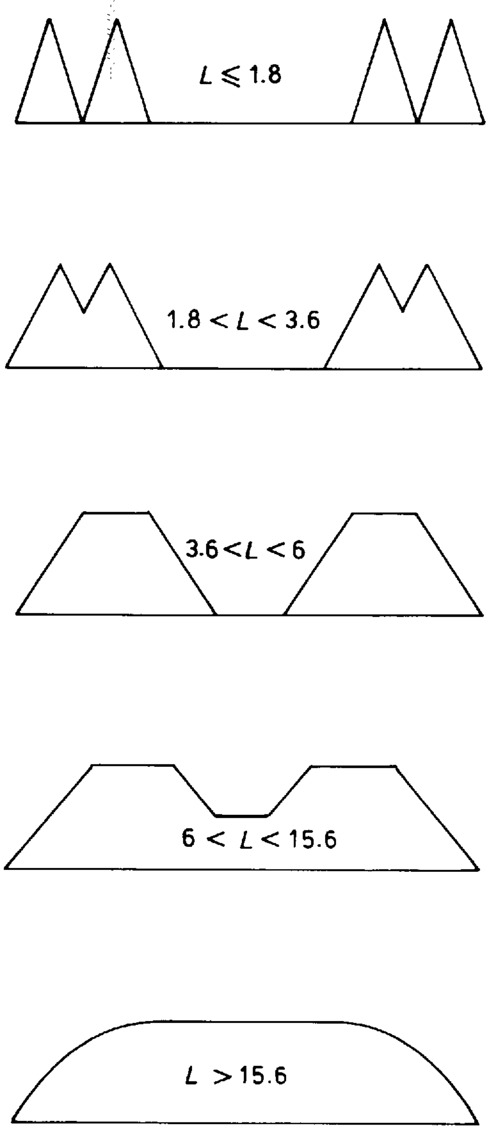
Example 1. Highway bridge. This shows the midspan bending of simply supported spans loaded by the standard fatigue vehicle and illustrates the effect of variations in L.



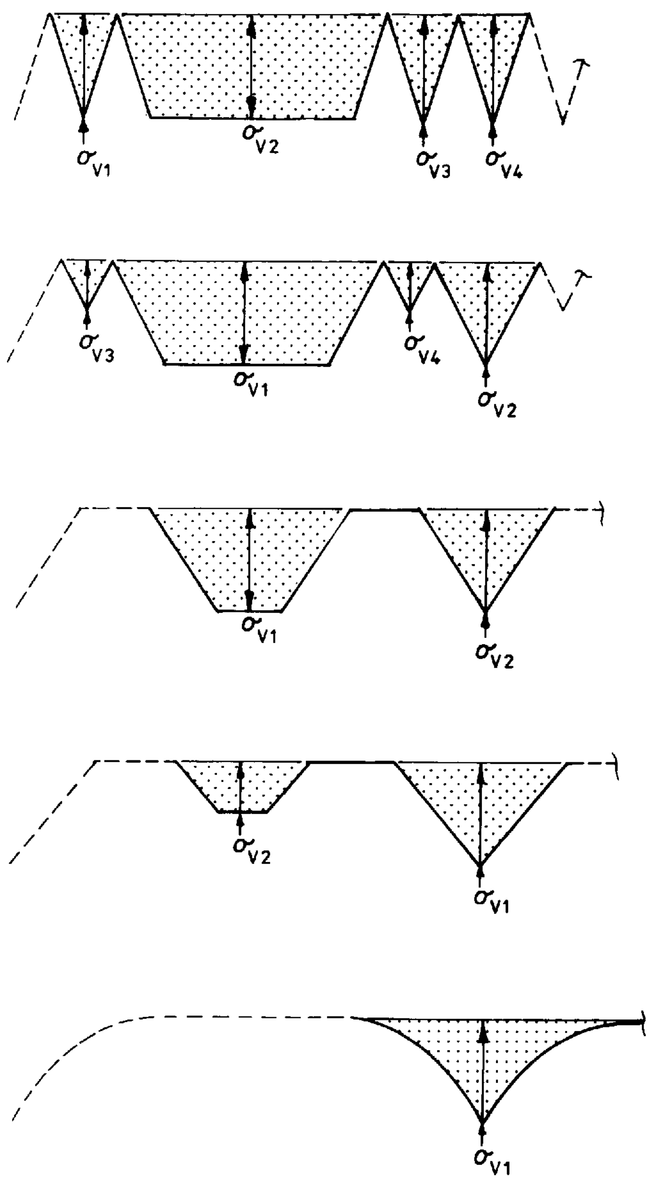
Influence line diagram



Standard fatigue vehicle

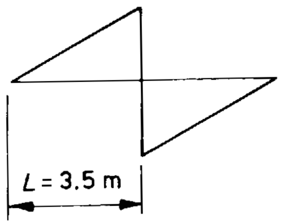


Stress histories

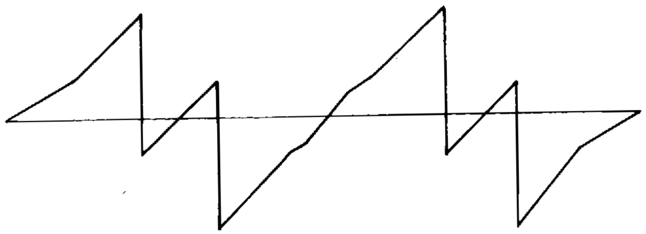


Cycle counting diagrams

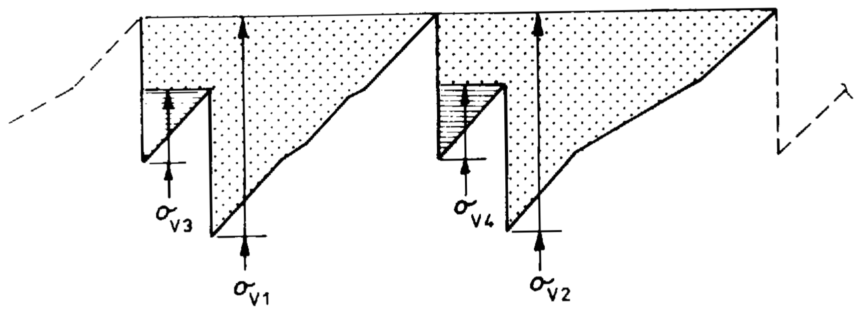
Example 2. Highway bridge. This shows the shear at mid-span of a 7 m simply supported span loaded by the standard fatigue vehicle.



Influence line diagram

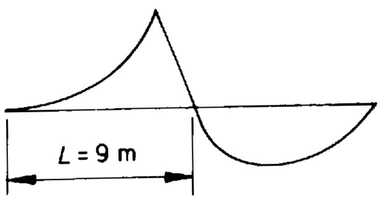


Stress history

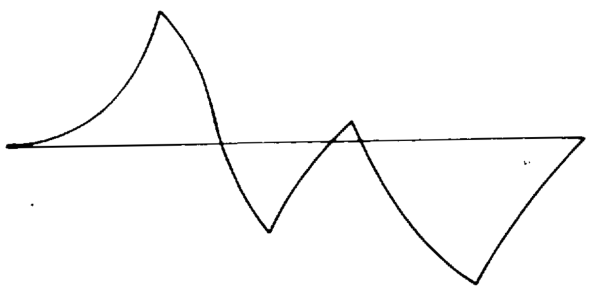


Cycle counting diagram

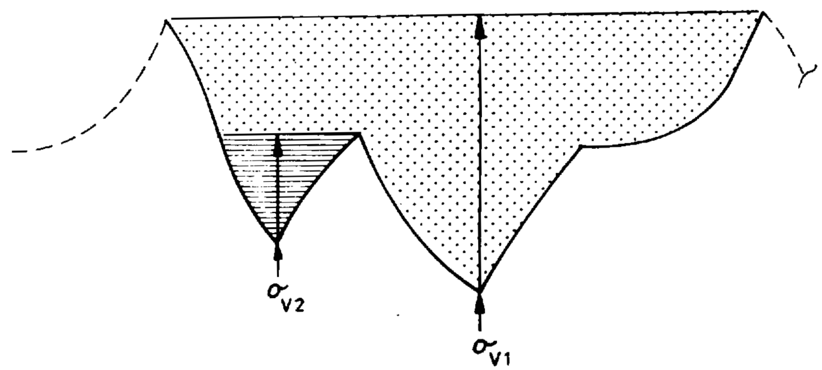
Example 3. Highway bridge. This shows the 4/5 span-point bending of a 2 × 9 m span continuous beam loaded by the standard fatigue vehicle.



Influence line diagram

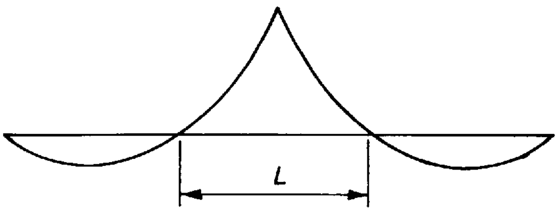


Stress history



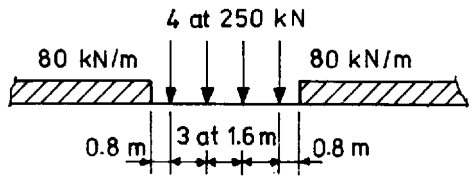
Cycle counting diagram

Example 4. Railway bridge. This shows the mid-span bending of a three-span continuous beam loaded with standard RU loading.

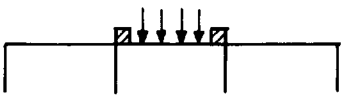


Influence line diagram

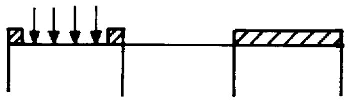
L = base length of loop containing largest ordinate (measured in direction of travel)



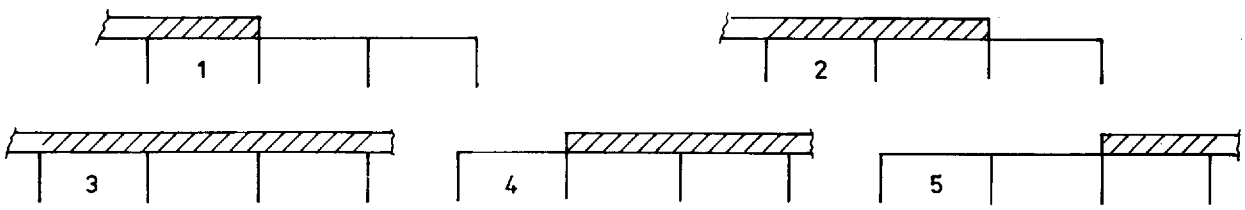
RU Loading to be multiplied by the dynamic factor



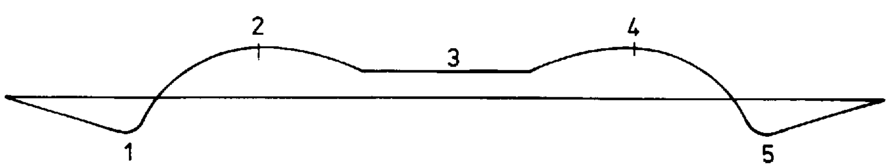
Loading diagram for $\sigma_{p\,max}$



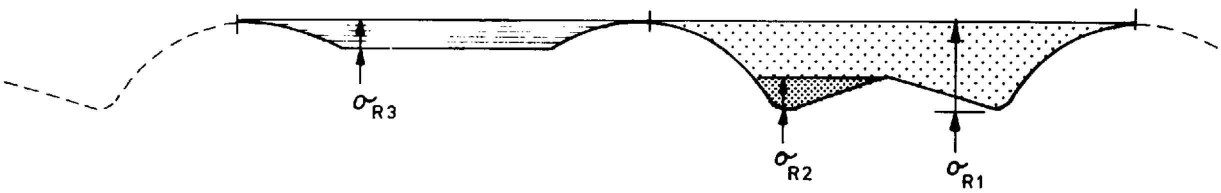
Loading diagram for $\sigma_{p\,min}$



Loading diagrams for stress history



Stress history



Cycle counting diagram

NOTE. In examples 1 to 4 given above, the cycle counting diagrams, for comparative purposes, follow the same profile as the stress history. Since the analogy depends solely on the depth of water retained in each section, it is immaterial whether the profiles are as illustrated or with successive peaks and troughs joined by straight lines (as proposed in appendix B).

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Appendix G

Testing of shear connectors

G.1 General. This appendix outlines the procedures which should be followed if the fatigue strength of shear connectors is to be determined by testing, as required by **6.4.1**.

G.2 Procedure. Test the specimens under constant amplitude loading at frequencies not exceeding 250 cycles/min. Ensure that the frequency of the applied loading is the same for each specimen within a particular series of tests. Ensure also that the maximum load on any connector does not exceed 0.5 times the nominal static strength of the connector (determined in accordance with Part 5) with the appropriate concrete strength, at the time of testing, being determined in accordance with the requirements of BS 1881.

Stresses may either be determined from the applied test load, in accordance with clause **6**, or derived from strain gauge readings.

G.3 Class S criteria. To enable the weld metal attaching shear connectors to be classified as a class S detail, the welds should be shown by tests carried out in accordance with **G.2** to have a 97.7 % probability of surviving 10^5 and 10^7 repetitions of stress ranges of 146 N/mm² and 82 N/mm² respectively, where the stress ranges are computed in accordance with either **6.4.2** for stud connectors or **6.4.3** for bar and channel connectors.

Where these conditions are not satisfied, the design $\sigma_r - N$ relationship for the detail (i.e. 2.3 % probability of failure) should be derived in accordance with appendix A and the method given in **8.4** should be used to assess the fatigue life.

Appendix H

Explanatory notes on detail classification

H.1 General

H.1.1 ***Scope.*** This appendix gives background information on the detail classifications given in tables 17(a), 17(b) and 17(c). This includes notes on the potential modes of failure, important factors influencing the class of each detail type and some guidance on selection for design.

H.1.2 ***Geometrical stress concentration factors.*** Unless otherwise indicated in table 17, the stress concentrations inherent in the make-up of a welded joint have been taken into account in the classification of the detail. However, where there is a geometrical discontinuity, such as a change of cross section or an aperture (see figure 21) and/or where indicated in table 17, the resulting stress concentrations should be determined either by special analysis or by the use of the stress concentration factors given in figure 22.

H.2 Type 1 classifications, non-welded details. See table 17(a).

H.2.1 ***Notes on potential modes of failure.*** In unwelded steel, fatigue cracks normally initiate either at surface irregularities, at corners of the cross sections, at holes and re-entrant corners or at the root of the thread for bolts or screwed rods. In steel, which is holed and connected with rivets or bolts, failure generally initiates at the edge of the hole and propagates across the net section, but in double covered joints made with H.S.F.G. bolts this is eliminated by the pretensioning, providing joint slip is avoided, and failure initiates on the surface near the boundary of the compression ring due to 'fretting' under repeated strain.

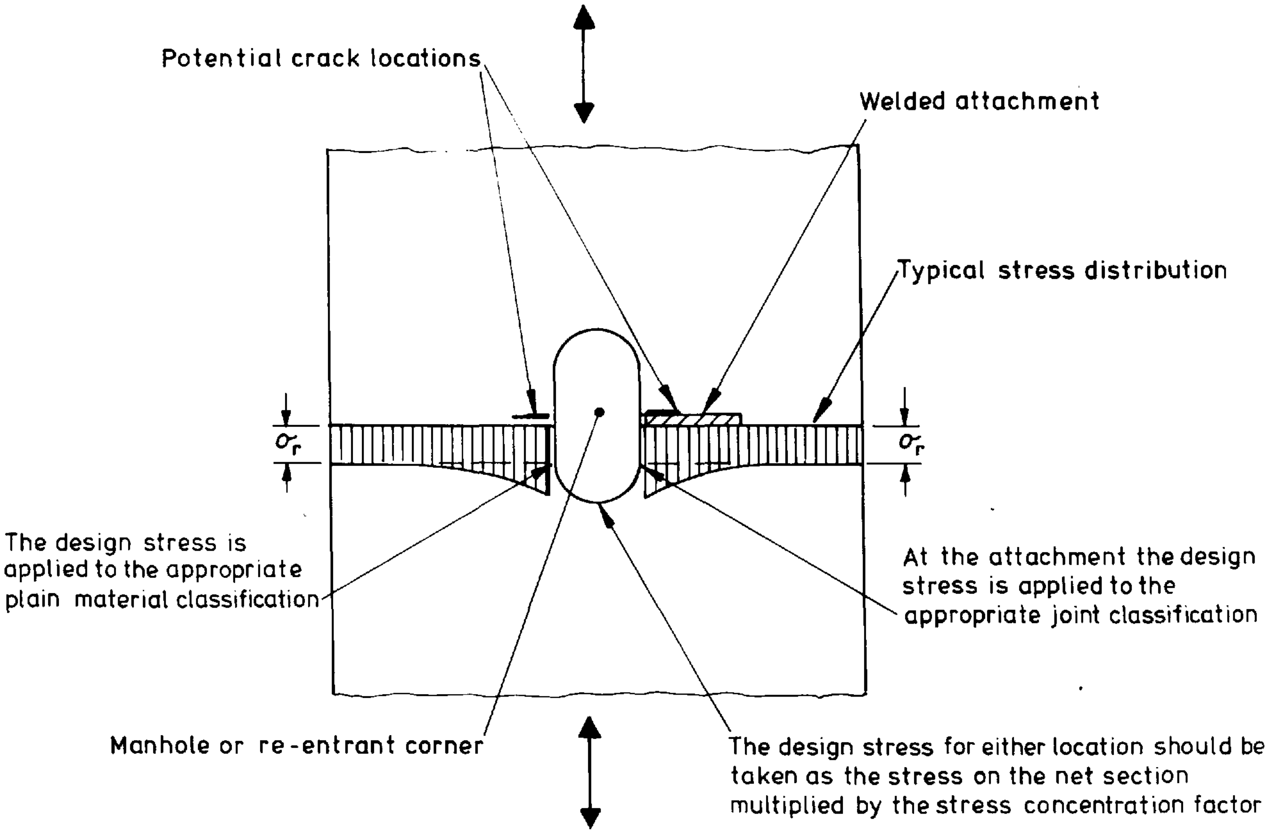
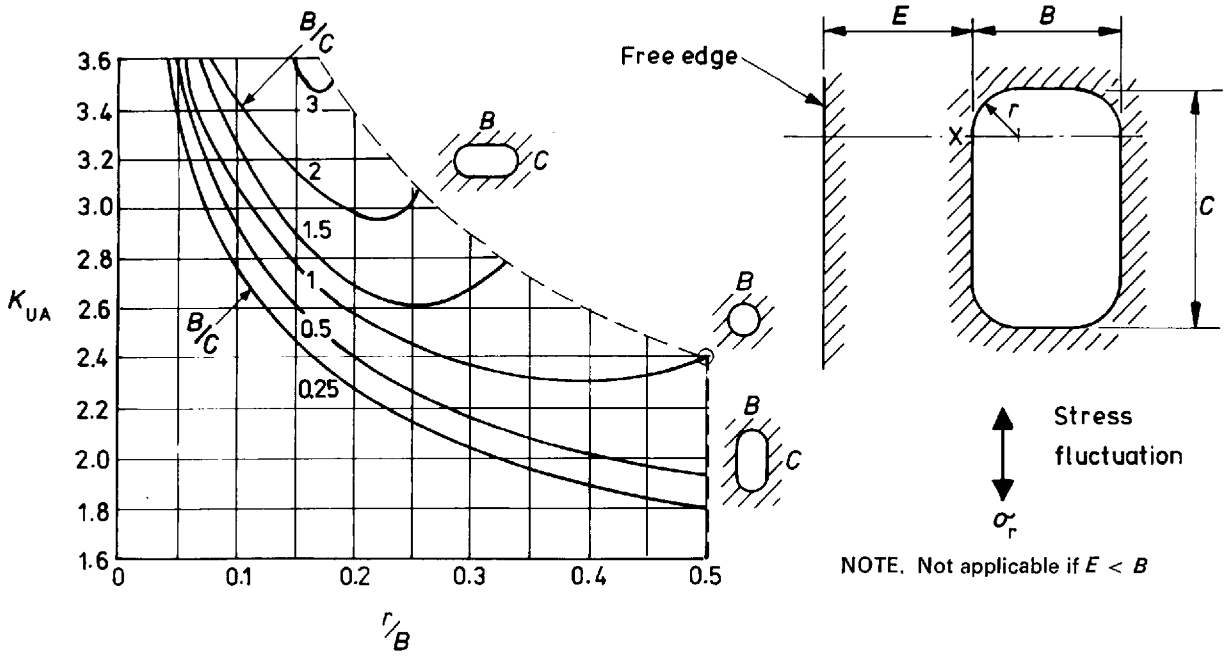
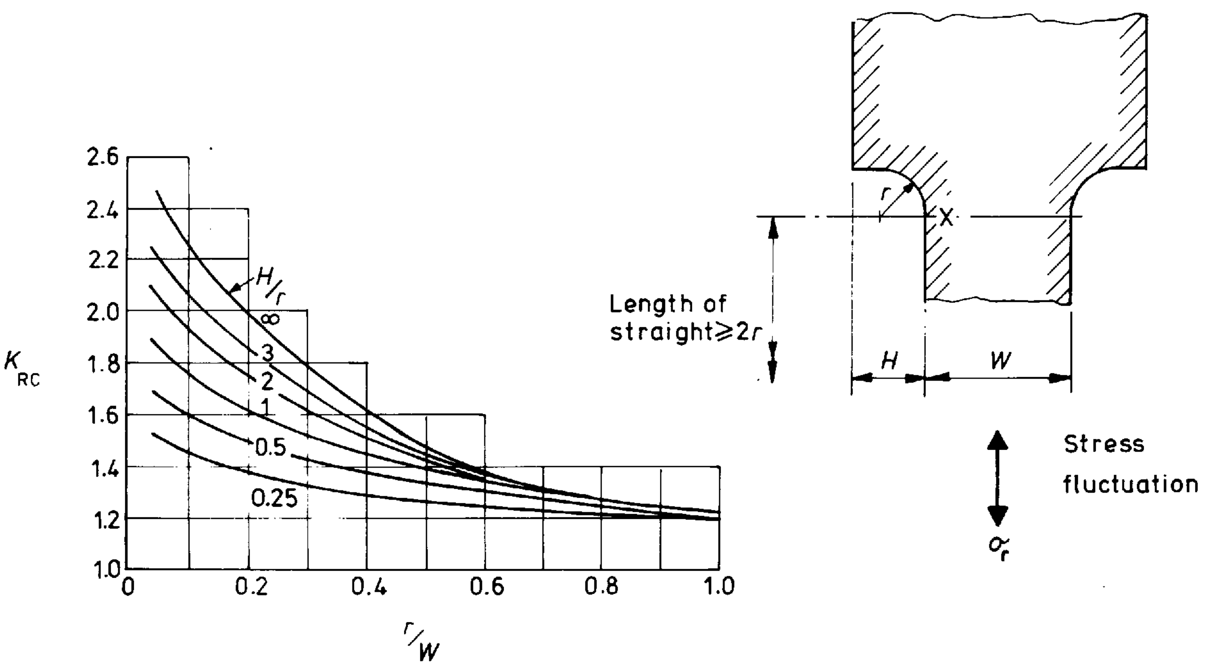


Figure 21. Typical example of stress concentrations due to geometrical discontinuity



(a) Fatigue stress concentration factor for unreinforced apertures K_{UA}
(based on net stress at X)



(b) Fatigue stress concentration factor for re-entrant corners K_{RC}
(based on net stress at X)

Figure 22. Stress concentration factors

H.2.2 ***General comments.*** In welded construction, fatigue failure will rarely occur in a region of unwelded material since the fatigue strength of the welded joints will usually be much lower.

H.2.3 *Comments on particular detail types*

Type 1.3. All visible signs of drag lines should be removed from the flame cut edge by grinding or machining.

Types 1.3 and 1.4. The presence of an aperture or re-entrant corner implies the existence of a stress concentration and the design stress should be the stress on the net section multiplied by the relevant stress concentration factor (see figure 22).

Type 1.4. The controlled flame cutting procedure should ensure that the resulting surface hardness is not sufficient to cause cracking.

Type 1.5. This type may be deemed to include bolt holes for attaching light bracing members where there is negligible transference of stress from the main member in the direction of σ_r.

Type 1.6. This covers connections designed in accordance with Part 3 for slip resistance at the ultimate limit state and where secondary out-of-plane bending of the joint is restrained or does not occur (i.e. double-covered symmetric joints). Failure initiates by fretting in front of the hole.

Type 1.12. This classification applies to failure at the root of the thread in normal commercial quality threaded components. Attention should be paid to the details of head fillets, waisted shanks and thread run-out in components, not covered by an appropriate British Standard, to ensure that they have satisfactory fatigue resistance. A higher fatigue resistance can be obtained with a rolled thread on material which has previously been fully heat-treated, but such components should be subject to special test and inspection procedures.

For the use of black bolts complying with the requirements of BS 4190 and subjected to fluctuating tensile loads, see **6.5**.

Where bolts or screwed rods are pre-tensioned to a value in excess of an applied external load, stress fluctuations will be governed by the elasticity of the pre-compressed elements. The increase in tension will rarely exceed 10 % of an external load applied concentrically with the bolt axis, but where the load is eccentric, a further increase will result from prying action.

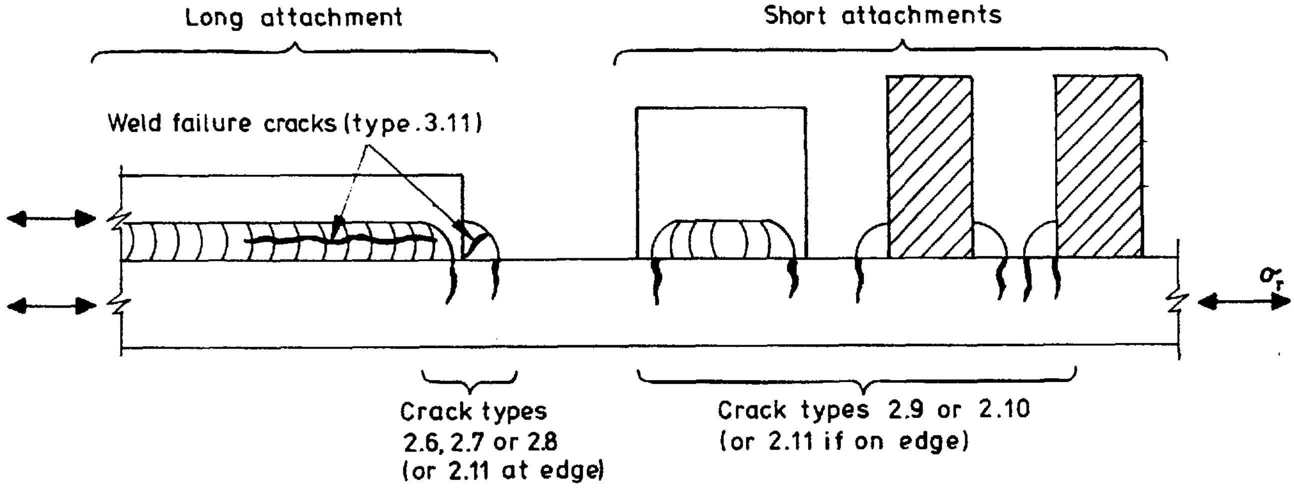
H.3 Type 2 classifications, welded details on surface of member. See table 17(b).

H.3.1 ***Notes on potential modes of failure.*** See figure 23.

When the weld is essentially parallel to the direction of stressing, fatigue cracks will normally initiate at the weld ends, but when the weld is transverse, cracking will initiate at the weld toes. In either case the cracks will then propagate into the stressed element. For attachments connected by single welds, cracks in parent metal may also initiate from the weld root. Cracks in stressed weld metal will initiate from the weld root (see type 3.11). Away from weld ends, fatigue cracks normally initiate at stop-start positions, or if these are not present, at weld surface ripples. With the weld reinforcement dressed flush, failure tends to be associated with weld defects.

H.3.2 *General comments.*

H.3.2.1 *Edge distance.* (See figure 24.) No edge distance criterion exists for continuous or regularly intermittent welds away from the ends of an attachment (see types 2.1 to 2.5). However, a criterion exists (types 2.6 to 2.10) to limit the possibility of local stress concentrations occurring at unwelded corners as a result of, for example, undercut, weld



NOTE. For classification purposes, an 'attachment' should be taken as the adjacent structural element connected by welding to the stressed element under consideration. Apart from the particular dimensional requirements given for each type in table 17(b), the relative size of the 'stressed element' and the 'attachment' is not a criterion.

Figure 23. Failure modes at weld ends

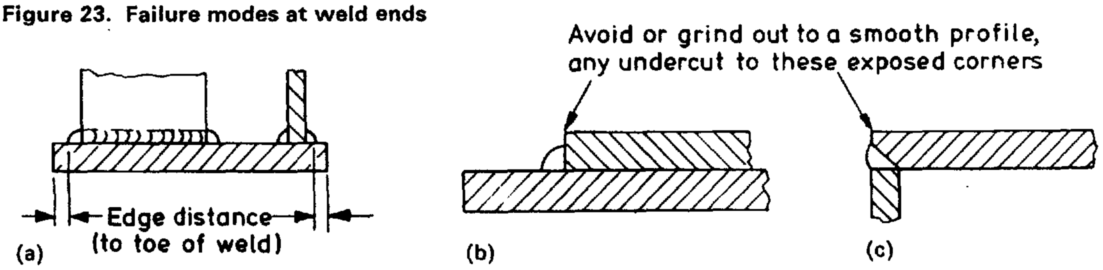


Figure 24. Edge distance

spatter and excessive leg size at stop-start positions or accidental overweave in manual welding. Although this criterion can be specified only for the 'width' direction of an element, it is equally important to ensure that no accidental undercutting occurs on the unwelded corners of, for example, cover plates or box members (see figure 24 (b) and (c)). Where it does occur, it should subsequently be ground out to a smooth profile.

Part 5 recommends the provision of a minimum edge distance of 25 mm for shear connectors, hence the criterion given in this part will automatically be met.

H.3.2.2 *Attachment of permanent backing strips.* If a permanent backing strip is used in making longitudinal butt welded joints it should be continuous or made continuous by welding. These welds and those attaching the backing strip should also comply with the relevant class requirements. The classification will reduce to E or F (type 3.3 or 3.4) at any butt welds in the backing strip or class E at any permanent tack weld (see **H.3.3**, type 2.4). It should be noted that transverse butt welds on backing strips may be downgraded by tack welds close to their ends (see **H.4.3**, type 3.4).

H.3.2.3 *Stress concentrations.* These are increased, and hence the fatigue strength is reduced, where:

(a) the weld ends or toes are on, or near, an unwelded corner of the element (see **H.3.2.1**);

(b) the attachment is 'long' in the direction of stressing, and as a result, transfer of a part of the load in the element to and from the attachment will occur through welds adjacent to its ends;

(c) such load transfer is through joints which are not symmetrical about both axes of cross section of the stressed element.

H.3.2.4 *Weld forms.* Full or partial penetration butt welded joints of T form (such as would connect attachments to the surface of a stressed element) should be completed by fillet welds of leg length at least equal to 25 % of the thickness of the attachment. The fillets exclude the possibility of an increase in stress concentration arising at an acute re-entrant angle between the element surface and the toe of the weld, and thus, in considering the effects on the stressed element, it is immaterial whether the attachment is fillet or butt welded to the surface, since a similar toe profile results in both cases.

H.3.2.5 *Tack welds.* Tack welds, unless carefully ground out or buried in a subsequent run, will provide potential crack locations similar to any other weld end. Their use in the fabrication process should be strictly controlled.

NOTE. Apart from the width transverse to σ_r, neither the shape of the end of an attachment nor the orientation or continuity of the weld at its end affects the class.

H.3.3 *Comments on particular detail types*

Type 2.1 Finish machining should be in the direction of σ_r. The significance of defects should be determined with the aid of specialist advice and/or by the use of a fracture mechanics analysis. The N.D.T. technique should be selected with a view to ensuring the detection of such significant defects. This type is only recommended for use in bridgeworks in exceptional circumstances.

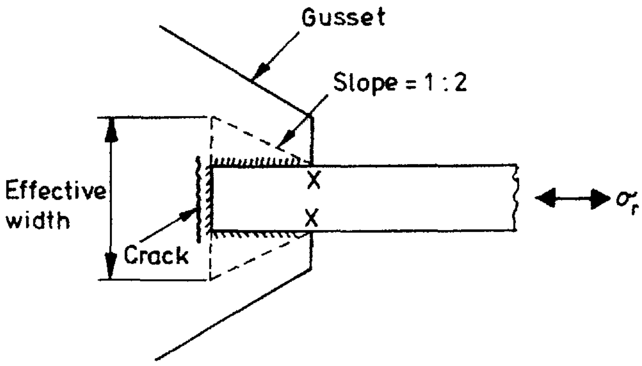
Type 2.2. Accidental stop-starts are not uncommon in automatic processes. Repair to the standard of a C classification should be the subject of specialist advice and inspection and should not be undertaken in bridgeworks.

Type 2.4. The limiting gap ratio m/h applies even though adjacent welds may be on opposite sides of a narrow attachment (as in the case of a longitudinal stiffener with staggered fillet welds). Long gaps between intermittent fillet welds are not recommended as they increase the risk of corrosion and, in the case of compression members, may cause local buckling (see Part 3). If intermediate gaps longer than 2.5 h are required the class should be reduced to F. This type also includes tack welds to the edges of longitudinal backing strips irrespective of spacing, provided that the welds comply in all respects with the workmanship requirements for permanent welds and that any undercut on the backing strip is ground smooth. The effects of tack welds which are subsequently fully ground out or incorporated into the butt weld by fusion, need not be considered.

Type 2.7. The classification may be deemed to include stress concentrations arising from normal eccentricities in the thickness direction.

This type includes parent metal adjacent to the ends of flange cover plates regardless of the shape of the ends.

Types 2.7 and 2.8. Where a narrow attachment is transferring the entire load out of a wide member, as in the case of a welded lap type connection between, for example, a cross brace and a gusset, the stress in the gusset at the end of the cross brace will vary substantially across the section. For assessing the stress in the gusset the effective width should be taken as shown in figure 25.



NOTE. For failure in the cross brace at X the cross brace is the 'member' and the gusset is the 'attachment'.

Figure 25. Effective width for wide lap connections

Type 2.10. This applies where any applied shear stress range is (numerically) greater than 50 % of a co-existent applied direct stress range.

Experimental evidence indicates that where significant shear stress co-exists with direct stress, the use of principal stress values may be conservative and accordingly the classification is upgraded.

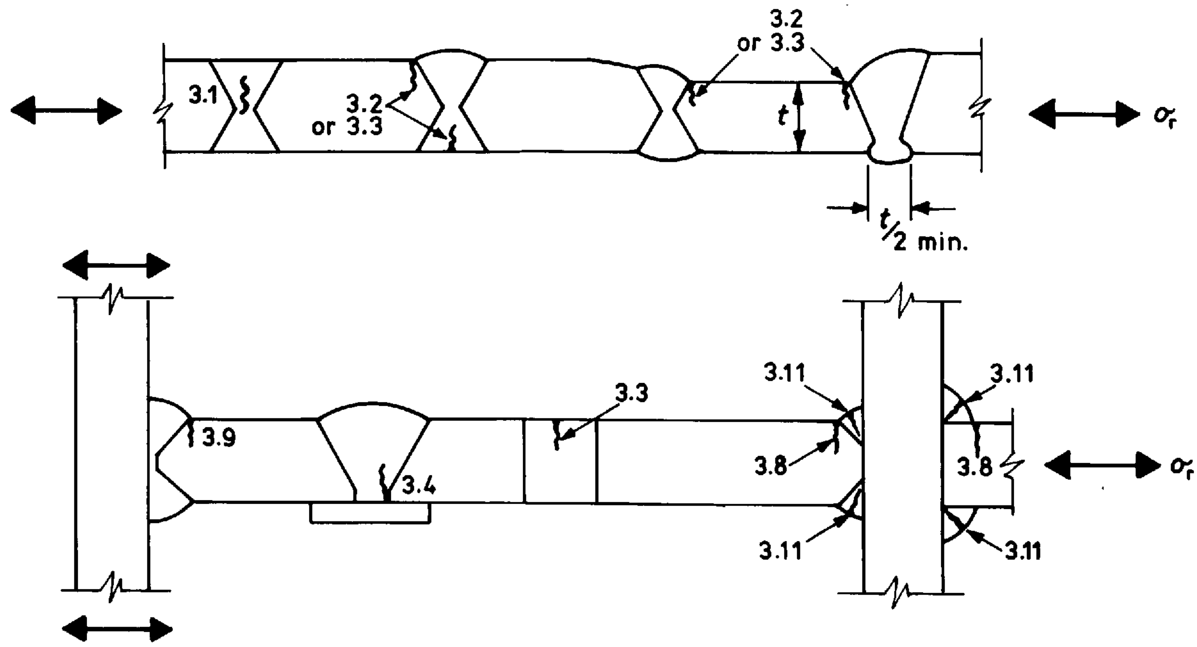
Type 2.11. This type applies regardless of the shape of the end of the attachment. In all cases, care should be taken to avoid undercut on element corners or to grind it out to a smooth profile should it occur. In particular, weld returns across a corner should be avoided and the use of cover plates wider than the flange, to which they are attached, is not recommended.

H.4 Type 3 classifications, welded details at end connections of member. See table 17(c).

H.4.1 ***Notes on potential modes of failure.*** (See figure 26.) With the ends of butt welds machined flush with the plate edges, or as otherwise given below, fatigue cracks in the as-welded condition normally initiate at the weld toe and propagate into the parent metal, so that the fatigue strength depends largely upon the toe profile of the weld. If the reinforcement of a butt weld is dressed flush, failure can occur in the weld material if minor weld defects are exposed, e.g. surface porosity in the dressing area (see **H.4.3**, type 3.3).

In the case of butt welds made on a permanent backing, fatigue cracks initiate at the weld metal-strip junction and then propagate into the weld metal.

In fillet or partial penetration butt welds, fatigue cracks in weld metal will normally initiate from the weld root.



NOTE. Fatigue cracks in reinforcing bars will normally initiate in similar locations to those for structural joints, given similar stress conditions and joint geometry.

Figure 26. Type 3 failure modes

H.4.2 ***General comments***

H.4.2.1 *Misalignments.* The classifications may be deemed to include for the effects of any accidental misalignments up to the maximum value specified in Part 6, provided that the root sides of joints with single sided preparations (i.e. single bevel -J, -U or -V forms) are back-gouged to a total width at least equal to half the thickness of the thinner element.

H.4.2.2 *Design stresses.* For elements where out-of-plane bending is resisted by contiguous construction (e.g. beam flanges supported by webs, wide plates supported by effectively continuous stiffeners, etc.) eccentricities due to axial misalignments in the thickness direction may be neglected. Where such support is not provided (e.g. tension links) the design stress should include an allowance for the bending effects of any intentional misalignment, i.e. the nominal distance between the centres of thickness of the two abutting components. For components tapered in thickness, the centre of the untapered section should be used.

H.4.2.3 *Element edges.* In all cases, failures tend to be associated with plate edges and care should be taken to avoid undercut at the weld toes on the corners of the cross section of the stressed element (or on the edge at the toes of any return welds). Should it occur, any undercut should be ground out to a smooth profile.

H.4.2.4 *Part width welds.* Butt type welds may also occur within the length of a member or individual plate as, for example, in the case of:

(a) a plug weld to fill a small hole;

(b) a weld closing a temporary access hole with an infill plate;

(c) a hole or slot for a transverse member to slot through a wider member.

Although such geometries have not been given specific categories in table 17(c), types 3.3 and 3.4 may be deemed to cover plug and infill plate welds and types 3.7 and 3.8 may be deemed to cover slotted members. See also **H.4.3**.

H.4.2.5 *Joints welded from one side only.* Unless made on a permanent backing (type 3.4) welds made entirely from one side are not classified since the root profile will be dependent upon the welding procedures adopted. Accordingly, their use is not recommended unless subject to special tests and strict procedural control.

H.4.2.6 *Partial penetration butt welds.* All butt welds transmitting stress between ends of plates, sections or built-up members in bridgeworks should be full penetration, except where permitted in types 3.8 and 3.10 (junctions with transverse members). In these latter cases, even though the joint may be required to carry wholly compressive stresses and the non-penetrated surfaces may be machined to fit, for fatigue purposes, the total stress fluctuation should be considered to be transmitted through the welds (e.g. column caps and bearing stiffeners).

H.4.2.7 *Welding of reinforcing bars for concrete.* Welding of reinforcement should comply with Parts 7 and 8. Lap welding of bars is not classified since adequate control cannot be exercised over the profile of the root beads and its use is not recommended under fatigue conditions.

H.4.3 ***Comments on particular detail types***

Type 3.1. The significance of defects should be determined with the aid of specialist advice and/or by the use of a fracture mechanics analysis. The N.D.T. technique should be selected with a view to ensuring the detection of such significant defects. This class should not normally be used in bridgeworks (see **5.1.2.5**).

Type 3.2. Shop welds made entirely in the downhand position, either manually or by an automatic process other than submerged arc, tend to have a better reinforcement shape from the point of view of fatigue than positional, site or submerged arc welds (i.e. larger re-entrant angles at the toes and more uniform profiles). Accordingly, to cater for exceptional circumstances, joints made in this manner may be up-graded to class D.

Types 3.2 and 3.3. Thickness variations and surface misalignments up to the maximum values specified in Part 6 may be deemed to be included (see **H.4.2.1**).

These types do not normally include joints between rolled or built-up sections. See the note to type 3.6.

Type 3.3. Grinding smooth the reinforcement of butt welds until flush with the plate surface on both sides is generally beneficial. Provided that N.D.T. is done after grinding, this treatment can be assumed to raise the class to D.

Types 3.3 and 3.4. These types may be used for holes which are either filled with plugs of weld metal or welded infill plates. Such holes may also be required for stitching laminations or repairing lamellar tears. The welds should be full penetration and should be considered to be equivalent to type 3.3 or, if welded onto permanent backing material, type 3.4. The slot or hole dimensions should be in accordance with appendix A of BS 5135 : 1974.

Plug welds should not be used in bridgeworks for transmitting tensile force across two lapping plates. Their use for transmitting shear force is not recommended for major structural connections, but where they have to be used, failure through the weld throat should be considered to be class W, based on the minimum throat area projected in the σ_r direction.

Type 3.4. If the backing strip is fillet or tack welded to the plate (type 2.9) the detail class will not be reduced below class F unless permanently tacked within 10 mm of the member edge, in which case it will be class G (type 2.11).

Type 3.5. The effect of the stress concentration at the corner of the joint between two individual plates of different widths in line may be included in the classification. Where the end of one plate is butt welded to the side of another, refer to type 3.9.

Stress concentrations due to abrupt changes of width can often be avoided by tapering the wider plate (see types 3.2, 3.3 and 3.4).

Type 3.6. Butt welds between rolled sections or between built-up sections are prone to weld defects, which are difficult to detect, in the region of the web/flange junction (see figure 27). Special preparations, procedures and inspection may be undertaken in exceptional circumstances and type 3.3 may then be applied unless the weld is made on a permanent backing (type 3.4, see figure 27). Dressing of the weld reinforcement is advised to overcome poor reinforcement shape resulting from the greater misalignments which may occur in the jointing of sections.

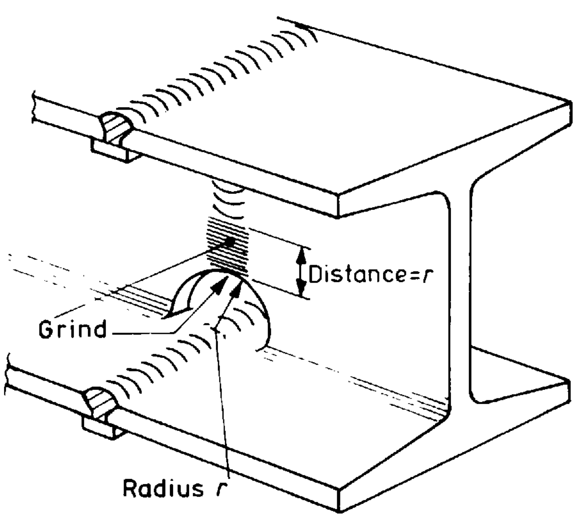


Figure 27. Type 3.6 joint

NOTE. This joint is frequently made using a semi-circular cope hole. This gives improved access to the flange butt welds when webs or longitudinal stiffeners have already been attached. The end of the web butt weld at the cope hole can be considered to be equivalent to class D with a stress concentration factor of 2.4 provided that the end of the butt weld and the reinforcement within a distance equal to the radius (r) are ground flush. Cope holes of 45° mitre are not recommended.

Types 3.7 and 3.8. Weld metal failure will not govern with full penetration welds.

Where the third member is a plate it may be assumed that plane sections remain plane in the main members and that axial and bending stress distribution in the σ_r direction are unaffected. Where the third member is an open shape, for example, an I section or a hollow tube, particularly if different in width, a discontinuity in the main member stress pattern will occur. In this case the stress parameter should be the peak stress concentration at the joint. In the absence of published data on a particular joint configuration, the stress concentration factor may have to be determined by finite element or model analysis.

Plane sections may be assumed to remain plane where the main member stress can be continued through the transverse member by additional continuity plating of comparable cross-sectional area, which is in line with the main member components (see figure 28). In this type of connection it is important that the joint regions of the third member are checked before welding for lameller rolling defects and after welding for lamellar tears.

Where two flat plates intersect in the same plane, as in the case of flanges at the junction of two girders, the stress concentration factor due to the abrupt change of width should be used (see figure 29). If the weld is a full penetration butt carried out in accordance with all the recommendations for type 3.5 the detail may be classed as F2 without applying a stress concentration factor.

These types may be deemed to cover the case where a narrow third member is slotted through a single main member away from an end connection (see figure 30). In this case, the third member should be assumed to transmit the stress which the parent material would have carried before the slot was cut. If the length of the slot is longer than 150 mm in the σ_r direction, type 3.7 (full penetration butt joints) should be reclassed from F to F2. Note that this detail should generally be avoided, when possible, as slots are difficult to cut accurately and fit-up for welding is often poor. Where member B is called upon to carry high tensile stress, a slot in A avoids any risk from lamellar tears. However, with respect to stress fluctuation in member B the detail shown in figure 30 is type 2.11 (class G) at point Y. If B is critical and A is not, circular cut-outs at the corners of B will improve the class to F (type 2.9).

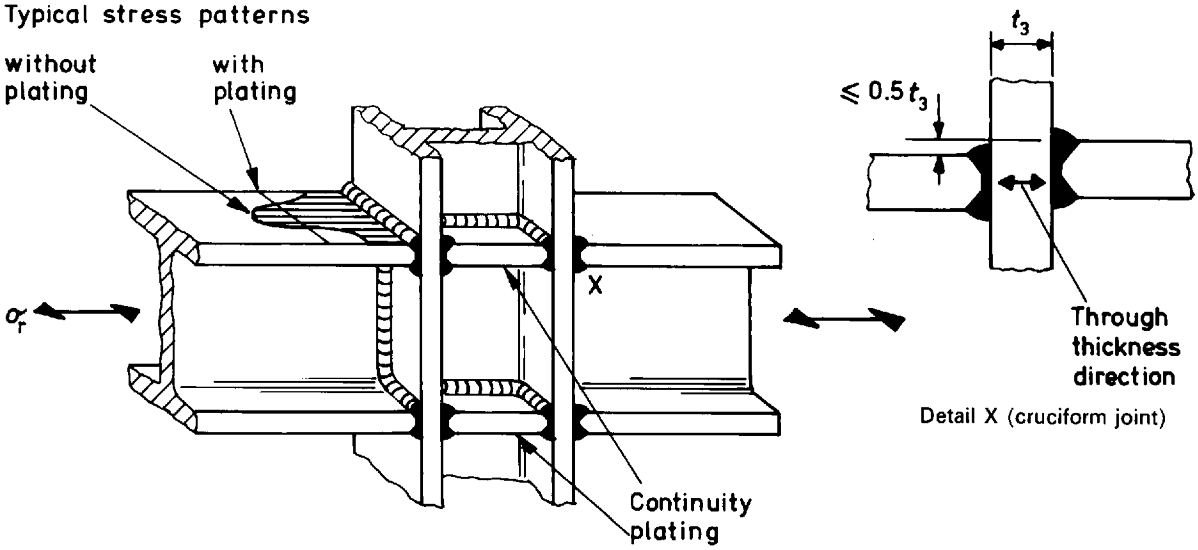


Figure 28. Use of continuity plating to reduce stress concentrations in type 3.7 and 3.8 joints

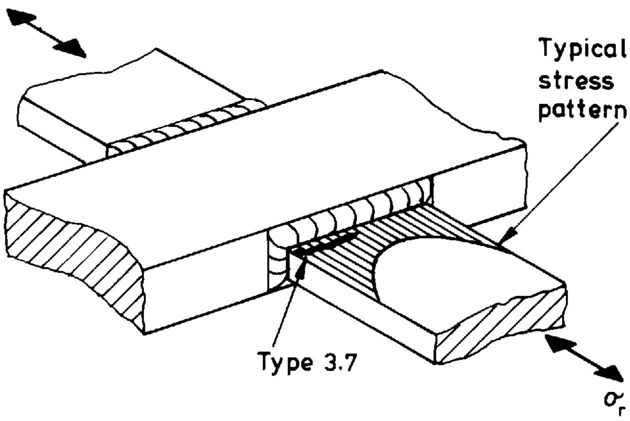
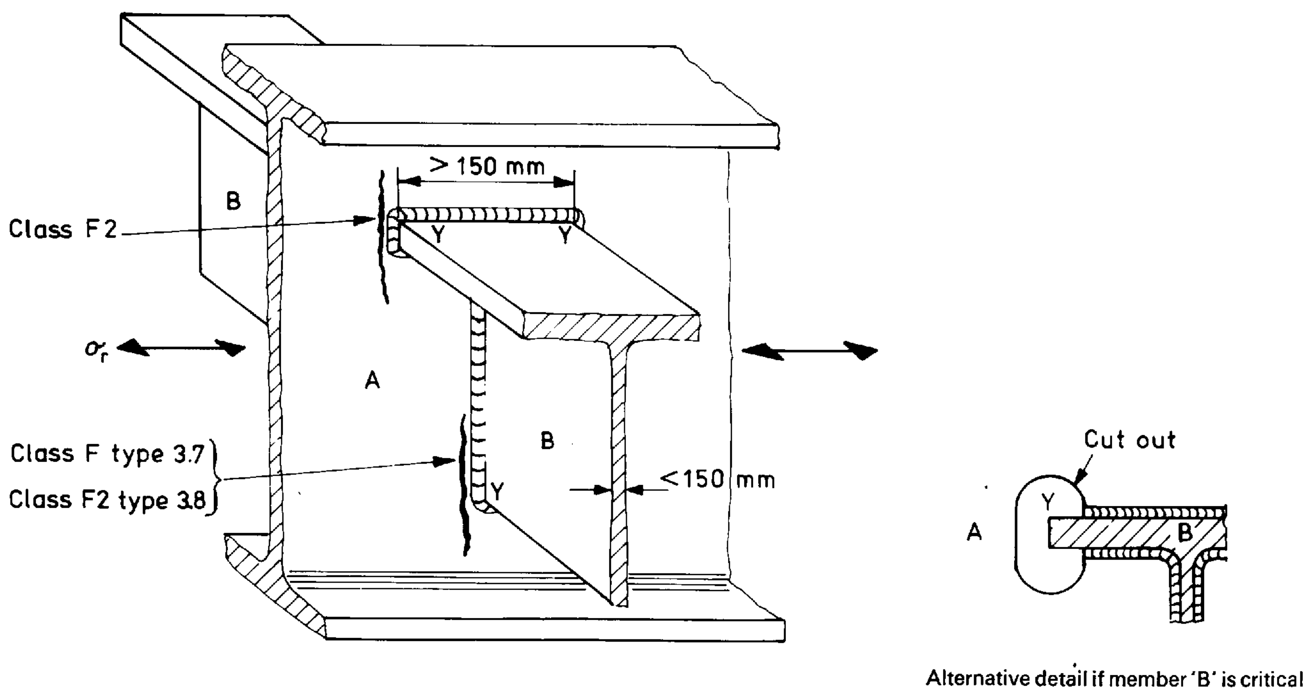


Figure 29. Cruciform junction between flange plates



NOTE. Main member 'A' is slotted, 'third' member 'B' is continuous and is welded all round the slot to 'A'.

Figure 30. Example of a 'third' member slotted through a main member

Type 3.8. In a fillet welded joint, weld metal failure (see type 3.11) will normally govern, unless the total weld leg length is about twice the element thickness. It will also govern in a partial penetration butt welded joint except where reinforced with fillet welds of adequate size.

Types 3.9 and 3.10. These detail types are distinguished from types 3.7 and 3.8 by the absence of a similar member in line on the far side of the joint. In this case an axial component in the first member will induce bending moment and hence curvature in the transverse member. Unless the latter is very stiff in bending an uneven stress distribution will result. Members with bolted end connections via transversely welded end plates are particularly susceptible to local increase of stress (see figure 31). Note that if the transverse member is an open or hollow section, local bending will increase the peak stress further (as in the case of types 3.7 and 3.8).

As far as fatigue failure of the transverse member is concerned, the first member is treated as a type 2 attachment and the stress parameter is the stress in the transverse member without the application of a stress concentration factor. In hollow or open transverse members this stress is often magnified by local bending of the walls.

Often the load is transmitted from a member to a transverse member primarily via flange plates in the same plane. This can occur in the case of a junction between cross girders and main girders, diagonals and truss chords, or in vierendeel frames (see figure 32). If the transverse member is relatively stiff (*i.e. its width is at least* 1.5 *times the width of the first member*) and a full penetration butt weld is used in accordance with the recommendations of type 3.5, the classification in this particular case may be considered to be effectively F2 with a stress concentration factor of unity. Otherwise the class shall be F with the appropriate stress concentration factor (see comment on detail types 3.7 and 3.8). Note that in the case of trusses, secondary stresses due to joint fixity should be taken into account. The fatigue strength of both flange plates may be improved by the insertion of a smoothly radiused gusset plate in the transverse member so that all butt welds are well away from re-entrant corners (see figure 33).

Type 3.11. Class W is primarily intended to apply to all fillet or partial penetration butt weld joints where bending action across the throat does not occur. Where lapped joints are welded on two or more sides, or tee or cruciform joints are welded from both sides (as shown in table 17(c)), such bending action is normally prevented. In certain cases difficulty of access may only allow welding to be done on one side of the joint. This applies particularly to small hollow members with welded corners, which if subject to loading that distorts the cross section, may cause failure of the corner weld in bending (see figure 34). Where axial stress is also present, the stress range at the face of the weld may be different from that at the root. Failure from ripples or stop-start positions on the face may give a higher strength than class W, but expert advice should be sought if a higher strength is required. In most cases failure from stress fluctuation in this root will be critical and this should always be classified as W.

Type 3.12. The stress ratio and effective weld size criteria of this clause are intended to aid in the exclusion of premature failures by local crushing of the concrete, by tearing of the attached flange or in the body of the connector.

This type covers embedded shear connectors at any position along a girder. The reference to 'end connections' in the title of table 17(c) refers to the end of the member in which failure occurs; in this case the welded end of the shear connector.

Type 3.13. Single sided manual metal arc procedures, with or without the use of backing material, are not recommended unless specialist advice is sought. Weld metal failure need not be considered.

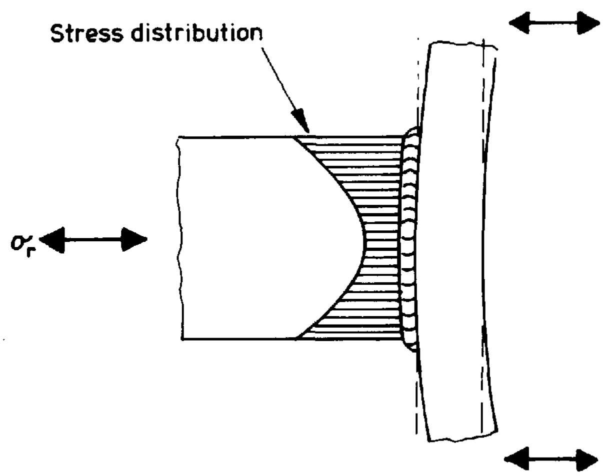


Figure 31. Example of type 3.9 or 3.10 joint

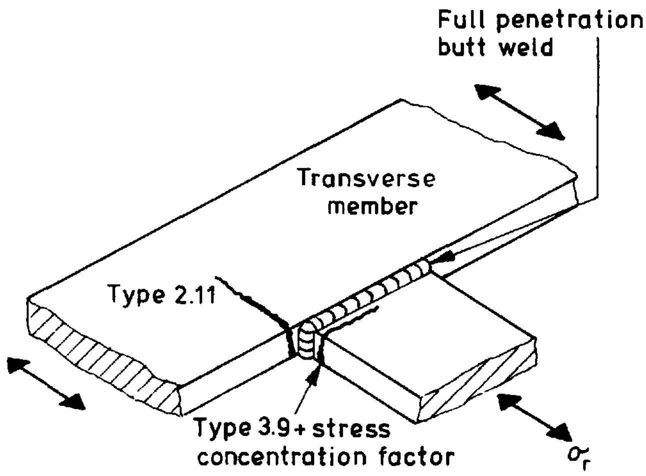


Figure 32. Tee junction of two flange plates

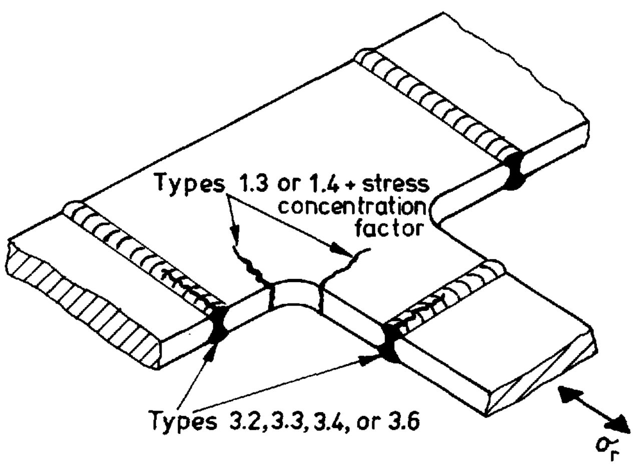


Figure 33. Alternative method of joining two flange plates

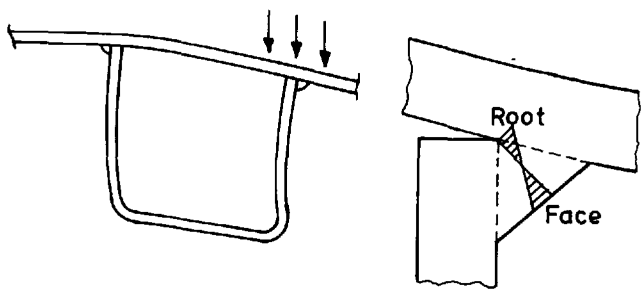
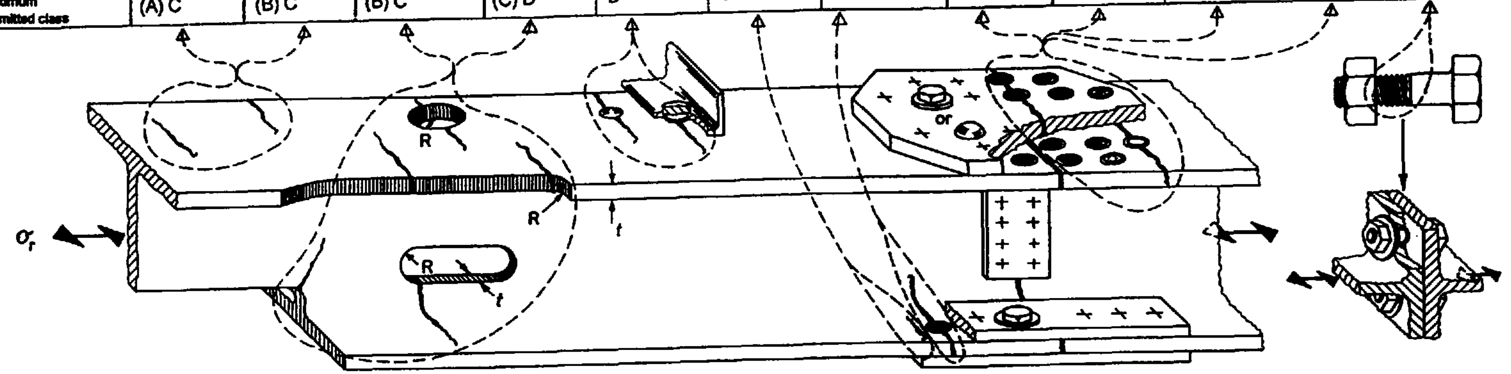


Figure 34. Single fillet corner weld in bending

Table 17. Classification of details

(a) Non-welded details

Product form	Rolled steel structural plates and sections											Threaded fasteners
Location of potential crack initiation	Away from all welding											In a butt joint, fastener axis parallel to σ_r
	Away from all structural connections					At a lapped or spliced connection fastened with:						
	On a member of constant or smoothly varying cross section		At any external or internal edge		At a small hole (may contain bolt for minor fixtures)	high strength friction grip bolts			rivets	precision bolts	black bolts	In thread root
						Away from hole	At a hole					
Dimensional requirements	No holes		Any aperture or re-entrant corner radius > t		Hole diameter ≤ $3t$	Double covered symmetrical joints only				Close tolerance hole		
	No re-entrant corners											
Manufacturing requirements (see also Part 6)	All surfaces fully machined and polished	Edges as rolled or machined smooth	Any flame cut edges subsequently machined or ground smooth		Hole drilled or reamered							Bolts to BS 3692 or BS 4395 Screw threads to BS 3643 : Part 2
						Tighten to BS 4604 : Parts 1, 2 and 3				Torque to full capacity or use lock nuts		
	No flame cutting			Any cutting of edges by planing or machine flame cutting with controlled procedure								
Special inspection requirements												
Design stress area			Net cross section			Gross	Net cross section					Core area (minor diameter)
Special design stress parameter			Use stress concentration factor for apertures or re-entrant corners			Designed for no slip at ultimate load (see Part 3)						See 7.5
Type number	1.1	1.2	1.3*	1.4*	1.5*	1.6*	1.7	1.8	1.9	1.10	1.11	1.12*
Maximum permitted class	(A) C	(B) C	(B) C	(C) D	D	C‡	C	D	D	E	G‡	B



Key Typical fatigue crack location Cut edge Fastener Direction of stress fluctuation

*See H.2.3.

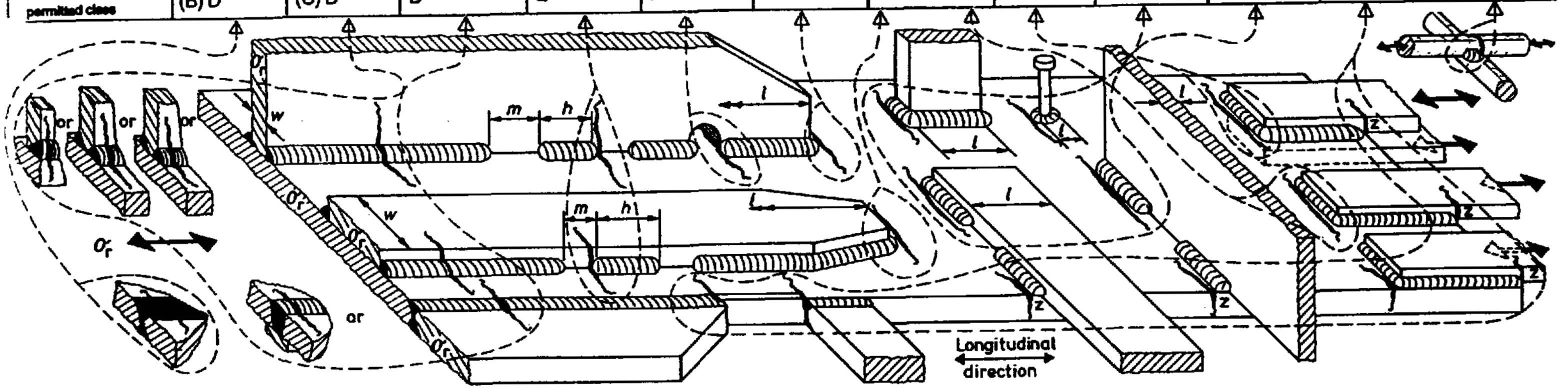
‡ Classifications that should be used with caution.

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are achieved.

NOTE. Tables 17(a), (b) and (c) are also available separately as a set of wall charts (BS 5400 : Part 10C).

(b) Welded details other than at end connections of a member

Product form	Rolled steel structural plates, sections and built-up members											Reinforcing steel in concrete
Location of potential crack initiation	At a long welded attachment (in direction of σ_r)								At a short welded attachment		At any attachment	At welded intersections in fabric or between hot rolled bars
	Away from weld end			At an intermediate gap in a longitudinal weld	At a cope hole	At a weld end						
					Narrow attachment		Wide attachment				Close to edge of member	
							On one side only	On both sides symmetrically				
Dimensional requirements	Butt weld full penetration			Fillet weld		Weld toe not less than 10 mm from member edge					Weld toe within 10 mm of member edge	
				Intermittent $\frac{m}{h} < 2.5$		Weld length (parallel to σ_r) $l > 150$ mm			$l < 150$ mm			
					Attachment width $w \leq 50$ mm		$w > 50$ mm					
Manufacturing requirements (see also Part 6)	Grind smooth any undercut on member edges										Grind any undercut	Resistance or manual plus grind smooth undercut
	Dress flush reinforcement	Automatic no stop-starts									Avoid weld returns round laps (see z)	
Special inspection requirements	Proved free of all significant defects											
Design stress area	Minimum transverse cross section of member at location of potential crack initiation											
Special design stress parameter									$\tau \leq 0.5\sigma$	$\tau > 0.5\sigma$		
Type number	2.1*	2.2*	2.3	2.4*	2.5	2.6	2.7*	2.8*	2.9	2.10*	2.11*	2.12
Maximum permitted class	(B) D	(C) D	D	E	F	F2	G	F2	F	E	G	D



Key Typical fatigue crack location Surface grinding Direction of stress fluctuation

NOTE. Weld throat fatigue cracks are type 3 (see table 17(c)).

*See H.3.3.

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are archieved.

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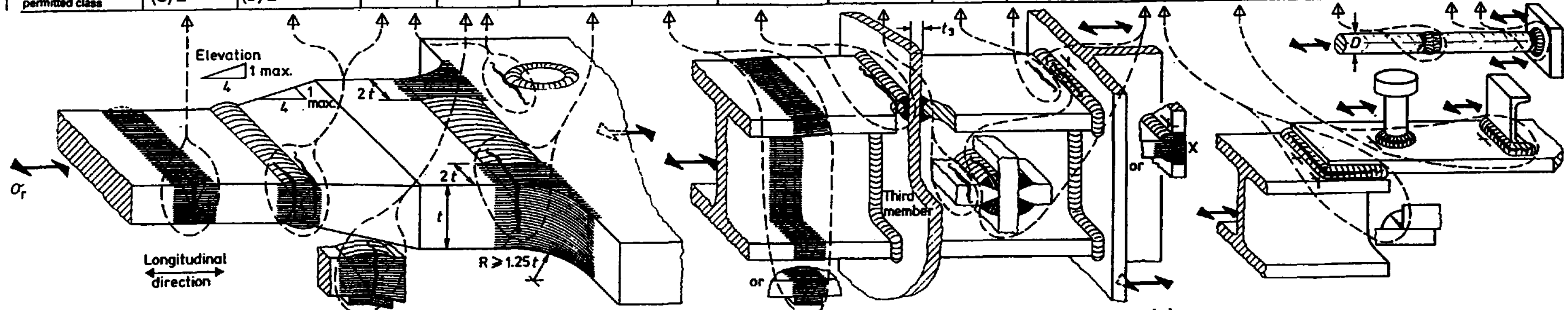
(c) Welded details at end connections of member

Product form	Rolled steel plates only					Rolled steel sections of built-up members						Shear connectors in concrete	Rolled steel reinforcing bars in concrete		
							(including plates)								
Location of potential crack initiation	At transverse weld joining:										In weld throat		At transverse weld		
	two single plates end to end					two members end to end			end of one member to side of another		In any in-completely fused joint	Between enclosed connector and any member	Between ends of bars	Between end of bar and surface of plate	
							with third member transverse through joint								
Dimensional requirements	Longitudinal axes in line								Full penetration butt weld	Partial penetration butt or fillet weld			Axes in line	Axes perpendicular	
	Full penetration butt weld							Partial penetration butt or fillet weld					Full penetration butt weld		Fillet weld leg size $\geq 0.4D$
	Equal width	Any width change $\leq$ 1 in 4 slope			Abrupt width change								Slope at any diameter change $\leq$ 1 in 4	Plus reinforcing fillets with leg size $\geq 0.25D$	
						Similar profile									
	Equal thickness	Any thickness change $\leq$ 1 in 4 slope													
			[also includes plug welds (see footnote)]				[also applies if third member is narrow and slots through first (see appendix H)]								
Manufacturing requirements (see also Part 6)	Welded from both sides			On permanent backing strip	Build-up corners to radius $\geq 1.25t$	Misalignment slope $\leq$ 1 in 4	If width permits make weld continuous round joint otherwise grind weld ends flush (see X)						Flash butt or manual welding from both sides		
	Misalignment slope $\leq$ 1 in 4	Downhand shop welds		No permanent tack welds within 10 mm of edge			Maximum misalignment of main members $\leq 0.5t_3$								
	Dress flush reinforcement				Grind corners within $2t$	Dress flush reinforcement									
	Temporary run-on and run-off plates used, weld and ground smooth														
	Grind smooth any undercut particularly on external corners												Grind smooth any undercut		
Special inspection requirements	Proved free of all significant defects						All regions stressed in through-thickness direction to be free from lamellar defects and tears							Check plate for lamellar defects and tears	
Design stress area	Minimum transverse cross section of member at location of potential crack initiation										'Effective' weld throat area		Minimum area of bar		
Special design stress parameter							Use the stress concentration factor unless third member is plate or has continuity plating		Stress concentration factor shall be used		See 6.3	See 6.4			
Type number	3.1*	3.2*	3.3*	3.4*	3.5*	3.6*	3.7*	3.8*	3.9*	3.10*	3.11*	3.12*	3.13*	3.14	3.15
Maximum permitted class	(C) E**	(D) E**	E**	F	F2	F2	F	F2	F	F2	W	S	(D) E**	F	F2



Key Typical fatigue crack location Surface grinding Direction of stress fluctuation Permitted variation to joint dimensional requirements [ ]

*See H.4.3.

NOTE. Potential fatigue crack locations marked are type 2 (see table 17(b)).

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are achieved.

** The classes shown are compatible with the highest workmanship standards set in Part 6 of this British Standard.

Standards publications referred to

BS 1881	Methods of testing concrete
BS 3643	ISO metric screw threads
BS 3692	ISO metric precision hexagon bolts, screws and nuts
BS 4190	ISO metric black hexagon bolts, screws and nuts
BS 4395	High strength friction grip bolts and associated nuts and washers for structural engineering
BS 4604	The use of high strength friction grip bolts in structural steelwork. Metric series
BS 5135	Metal-arc welding of carbon and carbon manganese steels
BS 5400	Steel concrete and composite bridges

BS 5400 : Part 10 : 1980

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BS 5400 : Part 10 : 1980

Amendments issued since publication

Amd. No.	Date of issue	Text affected

AMD 9352

Amendment No. 1
published and effective from 15 March 1999
to BS 5400 : Part 10 : 1980

Steel, concrete and composite bridges —

Part 10. Code of practice for fatigue

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BRITISH STANDARD

BS 5400 :
Part 10 : 1980

Incorporating Amendment No. 1

Steel, concrete and composite bridges —

Part 10: Code of practice for fatigue

ICS 93.040

Summary of pages

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Page	Issue	Page	Issue
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b	blank	44	original
c	blank	45 to 48	original
d	1	49	2
1	2	50	blank
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3	2	52	blank
4	2	53	2
4a	1	54	blank
4b	blank	Inside back cover	original
5	2	Back cover	2

Contents

Foreword

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

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

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

1. Scope

1.1 General. This Part of this British Standard recommends methods for the fatigue assessment of parts of bridges which are subject to repeated fluctuations of stress.

1.2 Loading. Standard load spectra are given for both highway and railway bridges.

1.3 Assessment procedures. The following alternative methods of fatigue assessment are described for both highway and railway bridges:

(a) simplified methods that are applicable to parts of bridges with classified details and which are subjected to standard loadings;

(b) methods using first principles that can be applied in all circumstances.

1.4 Other sources of fatigue damage. The following topics are not specifically covered by this Part of this British Standard but their effects on the fatigue life of a structure may need to be considered:

(a) aerodynamically induced oscillations;

(b) fluctuations of stress in parts of a structure immersed in water, which are due to wave action and/or eddy induced vibrations;

(c) reduction of fatigue life in a corrosive atmosphere (corrosion fatigue).

1.5 Limitations

1.5.1 ***Steel decks.*** Highway loading is included in this Part and is applicable to the fatigue design of welded orthotropic steel decks. However, the stress analysis and classification of details in such a deck is very complex and is beyond the scope of this Part of this British Standard.

1.5.2 ***Reinforcement.*** The fatigue assessment of certain details associated with reinforcing bars is included in this Part but interim criteria for unwelded bars are given in Part 4.

NOTE. These criteria are at present under review and revised criteria may be issued later as an amendment.

1.5.3 ***Shear connectors.*** The fatigue assessment of shear connectors between concrete slabs and steel girders acting compositely in flexure is covered in this Part, but the assessment of the effects of local wheel loads on shear connectors between concrete slabs and steel plates is beyond the scope of this Part of this British Standard. This effect may, however, be ignored if the concrete slab alone is designed for the entire local loading.

2. References

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

3. Definitions and symbols

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

3.1.1 ***fatigue.*** The damage, by gradual cracking of a structural part, caused by repeated applications of a stress which is insufficient to induce failure by a single application.

3.1.2 ***loading event.*** The approach, passage and departure of either one train or, for short lengths, a bogie or axle, over a railway bridge or one vehicle over a highway bridge.

3.1.3 ***load spectrum.*** A tabulation showing the relative frequencies of loading events of different intensities experienced by the structure.

NOTE. A convenient mode of expressing a load spectrum is to denote each load intensity as a proportion (K_w) of a standard load and the number of occurrences of each load as a proportion (K_n) of the total number of loading events.

3.1.4 ***standard load spectrum.*** The load spectrum that has been adopted in this Part of this British Standard, derived from the analysis of actual traffic on typical roads or rail routes.

3.1.5 ***stress history.*** A record showing how the stress at a point varies during a loading event.

3.1.6 ***combined stress history.*** A stress history resulting from two consecutive loading events, i.e. a single loading event in one lane followed by a single loading event in another lane.

3.1.7 ***stress cycle (or cycle of stress).*** A pattern of variation of stress at a point which is in the form of two opposing half-waves, or, if this does not exist, a single half-wave.

3.1.8 ***stress range (or range of stress)*** (σ_r). Either

(a) in a plate or element, the greatest algebraic difference between the principal stresses occurring on principal planes not more than 45° apart in any one stress cycle; or

(b) in a weld, the algebraic or vector difference between the greatest and least vector sum of stresses in any one stress cycle.

3.1.9 ***stress spectrum.*** A tabulation of the numbers of occurrences of all the stress ranges of different magnitudes during a loading event.

3.1.10 ***design spectrum.*** A tabulation of the numbers of occurrences of all the stress ranges caused by all the loading events in the load spectrum, which is to be used in fatigue assessment of the structural part.

3.1.11 ***detail class.*** A rating given to a detail which indicates its level of fatigue resistance. It is denoted by the following: A, B, C, D, E, F, F2, G, S, or W.

NOTE. The maximum permitted class is the highest recommended class, that can be achieved with the highest workmanship specified in Part 6 (see table 17). The minimum required class to be specified for fabrication purposes relates to the lowest $\sigma_r - N$ curve in figure 14, which results in a life exceeding the design life.

3.1.12 ***$\sigma_r - N$ relationship or $\sigma_r - N$ curve.*** The quantitative relationship between σ_r and N for a detail which is derived from test data on a probability basis.

3.1.13 ***design $\sigma_r - N$ curve.*** The $\sigma_r - N$ relationship adopted in this Part of this British Standard for design on the basis of 2.3 % probability of failure.

3.1.14 ***design life.*** The period in which a bridge is required to perform safely with an acceptable probability that it will not require repair.

3.1.15 ***standard design life.*** 120 years, adopted in this Part of this British Standard.

3.1.16 ***Miner's summation.*** A cumulative damage summation based on the rule devised by Palmgren and Miner.

3.2 Symbols. The symbols in this Part of this British Standard are as follows.

A	Net area of cross section
A_1	Effective weld throat area for the particular type of connector
d	Number of standard deviations below the mean line $\sigma_r - N$ curve
d_{120}	Life time damage factor (Miner's summation for 120 million repetitions of a stress range σ_v in a highway bridge)
F	Design stress parameter for bolts
K_0	Parameter defining the mean line $\sigma_r - N$ relationship
K_2	Parameter defining the $\sigma_r - N$ relationship for two standard deviations below the mean line
K_B	Value of ratio $\sigma_{v1B}/\sigma_{v1A}$ (highway bridges)
K_F	Miner's summation adjustment factor (highway bridges)
K_n	Proportion factor for occurrences of vehicles of a specified gross weight (320 K_w kN) in any one lane of a highway bridge
K_{RC}	Fatigue stress concentration factor for re-entrant corners
K_{UA}	Fatigue stress concentration factor for unreinforced apertures
K_w	Ratio of actual : standard gross weights of vehicles, trains, bogies or axles in a load spectrum
$k_1 - k_6$	Coefficients in the simplified assessment procedure for a railway bridge
L	Base length of that portion of the point load influence line which contains the greatest ordinate (see figure 12) measured in the direction of travel
M, M_1	Applied bending moments
m	Inverse slope of log σ_r/log N curve
N	Number of repetitions to failure of stress range σ_r
N_1, N_2	Number of repetitions to failure of stress ranges σ_{r1}, σ_{r2} ... etc., corresponding to n_1, n_2 ... etc., repetitions of applied cycles
n_1, n_2 .. etc.	Number of applied repetitions of damaging stress ranges σ_{r1}, σ_{r2} ... etc., in a design spectrum
n_c	Number of vehicles (in millions per year) traversing any lane of a highway bridge
$\bar{n}_c$	Effective value of n_c
n_R	Total number of live load cycles (in millions) for each load proportion K_w in a railway bridge
P, P_1	Applied axial forces
P_u	Basic static strength of the stud
z	Elastic modulus of section
γ_f	Partial safety factor for load (the product $\gamma_{f1} \cdot \gamma_{f2} \cdot \gamma_{f3}$, see Part 1)
γ_{fL}	Product of $\gamma_{f1} \cdot \gamma_{f2}$
γ_m	Partial safety factor for strength
Δ	Reciprocal of the antilog of the standard deviation of log N
$\Sigma \frac{n}{N}$	Miner's summation
σ_B	Stress on the core area of a bolt, determined on the basis of the minor diameter
σ_H	Limiting stress range under loading from the standard fatigue vehicle on a highway bridge
σ_N	Stress on net section
σ_o	Constant amplitude non-propagating stress range (σ_r at $N = 10^7$)
σ_p	Algebraic value of stress in a stress history
$\sigma_{p\,max}$, $\sigma_{p\,min}$	Maximum and minimum values of σ_p from all stress histories produced by standard loading
σ_r	Range of stress (stress range) in any one cycle
σ_{r1}, σ_{r2} ... etc	Individual stress ranges (σ_r) in a design spectrum
$\sigma_{R\,max}$	($\sigma_{p\,max} - \sigma_{p\,min}$) for a railway bridge
σ_{R1}, σ_{R2} ... etc	Stress ranges (in descending order of magnitude) in a stress history of a railway bridge under unit uniformly distributed loading
σ_T	Limiting stress range under standard railway loading
σ_u	Nominal ultimate tensile strength, to be taken as $1.1\sigma_Y$ unless otherwise specified
σ_v	Value of σ_r under loading from the standard fatigue vehicle (highway bridges)
$\sigma_{v\,max}$	($\sigma_{p\,max} - \sigma_{p\,min}$) for a highway bridge
σ_{v1}, σ_{v2} ... etc	Values of σ_v (in descending order of magnitude) in any one stress history for one lane of a highway bridge
σ_{v1A}	The largest value of σ_{v1} from all stress histories (highway bridges)
σ_{v1B}	The second largest value of σ_{v1} from all stress histories (highway bridges)
σ_x, σ_y	Coexistent orthogonal direct stresses
σ_Y	Nominal yield strength
τ	Shear stress coexistent with σ_x and σ_y

4. General guidance

4.1 Design life. The design life is that period in which a bridge is required to perform safely with an acceptable probability that it will not require repair (see appendix A).

The standard design life for the purposes of this Part of this British Standard should be taken as 120 years unless otherwise specified.

4.2 Classification and workmanship. Each structural steel detail is classified in accordance with table 17 (see **5.1.2**). This shows the maximum permitted class for different types of structural detail. The class denoted in table 17 determines the design of $\sigma_r - N$ curve in figure 14 that may be safely used with the highest workmanship standards specified in Part 6 for the detail under consideration.

In **5.3.1** is defined the information to be provided to the fabricator, to ensure that the appropriate quality standards for Part 6 are invoked.

4.3 Stresses. Stresses should generally be calculated in accordance with Part 1 of this British Standard but clause **6** of this Part supplements the information given in Part 1.

4.4 Methods of assessment. All methods of assessment described in this Part of this British Standard are based on the Palmgren-Miner rule for damage calculation (see clause **11**). The basic methods given respectively in **8.4** and **9.3** for highway and railway bridges may be used at all times. The simplified procedures given in **8.2** and **8.3** for highway bridges and in **9.2** for railway bridges may be used when the conditions stipulated in **8.2.1**, **8.3.1** and **9.2.1** are satisfied.

4.5 Factors influencing fatigue behaviour. The best fatigue behaviour of joints is achieved by ensuring that the structure is so detailed that the elements may deform in their intended ways without introducing secondary deformations and stresses due to local restraints. Stresses may also be reduced, and hence fatigue life increased, by increased thickness of parent metal or weld metal.

The best joint performance is achieved by avoiding joint eccentricity and welds near free edges and by other controls over the quality of the joints. Performance is adversely affected by concentrations of stress at holes, openings and re-entrant corners. Guidance in these aspects is given in table 17 and appendix H. The effect of residual stresses is taken into account in the classification tables.

5. Classification of details

5.1. Classification

5.1.1 *General*

5.1.1.1 For the purpose of fatigue assessment, each part of a constructional detail subject to fluctuating stress should, where possible, have a particular class designated in accordance with the criteria given in table 17. Otherwise the detail may be dealt with in accordance with **5.2.**

5.1.1.2 The classification of each part of a detail depends upon the following:

(a) the direction of the fluctuating stress relative to the detail;

NOTE. Propagation of cracks takes place in a direction perpendicular to the direction of stress.

(b) the location of possible crack initiation at the detail;

(c) the geometrical arrangement and proportions of the detail;

(d) the methods and standards of manufacture and inspection.

5.1.1.3 In welded details there are several locations at which potential fatigue cracks may initiate; these are as follows:

(a) in the parent metal of either part joined adjacent to;

(1) the end of the weld,

(2) a weld toe,

(3) a change of direction of the weld,

(b) in the throat of the weld.

In the case of members or elements connected at their ends by fillet welds or partial penetration butt welds and flanges with shear connectors, the crack initiation may occur either in the parent metals or in the weld throat: both possibilities should be checked by taking into account the appropriate classification and stress range. For other details, the classifications given in table 17 cover crack initiation at any possible location in the detail. Notes on the potential modes of failure for each detail are given in appendix H.

5.1.2 *Classification of details in table 17*

5.1.2.1 Table 17 is divided into three parts which correspond to the three basic types into which details may be classified. These are as follows:

(a) type 1, non-welded details, table 17 (a);

(b) type 2, welded details on surface, table 17 (b);

(c) type 3, welded details at end connections of members, table 17 (c).

5.1.2.2 Each classified detail is illustrated and given a type number. Table 17 also gives various associated criteria and the diagrams illustrate the geometrical features and potential crack locations which determine the class of each detail and are intended to assist with initial selection of the appropriate type number. (For important features that change significantly from one type to another see the footnote to table 17.)

5.1.2.3 A detail should only be designated a particular classification if it complies in every respect with the tabulated criteria appropriate to its type number.

5.1.2.4 Class A is generally inappropriate for bridge work and the special inspection standards relevant to classes B and C cannot normally be achieved in the vicinity of welds in bridge work. (For these and other classifications that should be used only when special workmanship is specified see the footnote to table 17.)

5.1.2.5 The classifications of table 17 are valid for the qualities of steel products and welds which meet the requirements of Part 6, except where otherwise noted. For certain details the maximum permitted class depends on acceptance criteria given in Part 6.

5.2 Unclassified details

5.2.1 *General.* Details not fully covered in table 17 should be treated as class G, or class W for load carrying weld metal, unless a superior resistance to fatigue is proved by special tests. Such tests should be sufficiently extensive to allow the design $\sigma_r - N$ curve to be determined in the manner used for the standard classes (see appendix A).

5.2.2 ***Post-welding treatments.*** Where the classification of table 17 does not give adequate fatigue resistance, the performance of weld details may be improved by post-welding treatments such as controlled machining, grinding or peening. When this is required the detail should be classified by tests as given in **5.2.1.**

5.3 Workmanship and inspection

5.3.1 *General.* Where the classification of a detail is dependent upon particular manufacturing or inspection requirements, which are not generally specified in Part 6 of this British Standard, the necessary standards of workmanship and inspection should be indicated on the relevant drawings.

All areas of the structure where welded details classified as class F or higher are necessary should be shown on the drawings together with the minimum required class and an arrow indicating the direction of stress fluctuation (see figure 1). For inspection purposes this information should be incorporated onto the fabricator's shop instructions.

Note that a joint may have more than one class requirement if it experiences significant stress fluctuations in two or more directions.

NOTE. The level of manufacturing quality can affect the fatigue life of all stuctural details. The manufacturing quality determines the degree to which discontinuities, that may act as stress raisers, may be introduced during the fabrication process. Such discontinuities can act as fatigue points, which may reduce the fatigue life to an unacceptable level for the detail under consideration. Details with a high permitted class are more seriously affected by such discontinuities because of the restrictions already placed by table 17 on stress raisers inherent in the form of the detail itself.

In order to dertermine which level of quality and inspection is required in accordance with Part 6, the minimum required class has to be derived. If a class higher than F2 is required this has to be specified on the drawings, otherwise the required fatigue life may not be achieved. If a class higher than F2 is specified, but not required, an uneconomical fabrication would result.

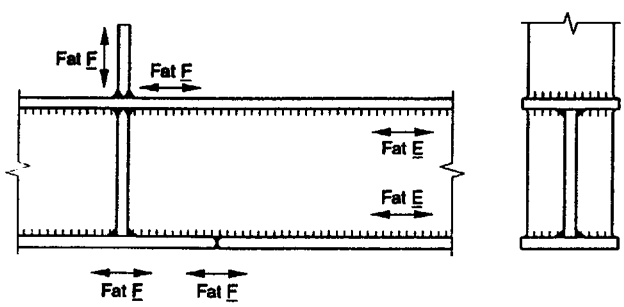


Figure 1. Method of indicating minimum class requirements on drawings

5.3.2 ***Detrimental effects.*** The following occurrences can result in a detail exhibiting a lower performance than its classification would indicate:

(a) weld spatter;

(b) accidental arc strikes;

(c) unauthorized attachments;

(d) corrosion pitting.

5.4 Steel decks. The classifications given in table 17 should not be applied to welded joints in orthotropic steel decks of highway bridges; complex stress patterns usually occur in such situations and specialist advice should be sought for identifying the stress range and joint classification.

6. Stress calculations

6.1 General

6.1.1 ***Stress range for welded details.*** The stress range in a plate or element to be used for fatigue assessment is the greatest algebraic difference between principal stresses occurring on principal planes not more than 45° apart in any one stress cycle.

6.1.2 ***Stress range for welds.*** The stress range in a weld is the algebraic or vector difference between the greatest and least vector sum of stresses in any one stress cycle.

6.1.3 ***Effective stress range for non-welded details***
For non-welded details, where the stress range is entirely in the compression zone, the effects of fatigue loading may be ignored.

For non-welded details subject to stress reversals, the stress range should be determined as in **6.1.1**. The effective stress range to be used in the fatigue assessment should be obtained by adding 60 % of the range from zero stress to maximum compressive stress to that part of the range from zero stress to maximum tensile stress.

6.1.4 *Calculation of stresses*

6.1.4.1 Stresses should be calculated in accordance with Part 1 of this British Standard using elastic theory and taking account of all axial, bending and shearing stresses occurring under the design loadings given in clause **7**. No redistribution of loads or stresses, such as is allowed for checking static strength at ultimate limit state or for plastic design procedures, should be made. For stresses in composite beams the modulus of elasticity of the concrete should be derived from the short term stress/strain relationship (see Part 4). The stresses so calculated should be used with a material factor $\gamma_m = 1$.

6.1.4.2 The bending stresses in various parts of a steel orthotropic bridge deck may be significantly reduced as the result of composite action with the road surfacing. However, this effect should only be taken into account on the evidence of special tests or specialist advice.

6.1.5 ***Effects to be included.*** Where appropriate, the effects of the following should be included in stress calculations:

(a) shear lag, restrained torsion and distortion, transverse stresses and flange curvature (see Parts 3 and 5);

(b) effective width of steel plates (see Part 3);

(c) cracking of concrete in composite elements (see Part 5);

(d) stresses in triangulated skeletal structures due to load applications away from joints, member eccentricities at joints and rigidity of joints (see Part 3).

6.1.6 ***Effects to be ignored.*** The effects of the following need not be included in stress calculations:

(a) residual stresses;

(b) eccentricities necessarily arising in a standard detail;

(c) stress concentrations, except as required by table 17;

(d) plate buckling.

6.2 Stress in parent metal

6.2.1 The reference stress for fatigue assessment should be the principal stress in the parent metal adjacent to the potential crack location, as shown in figure 2a. Unless otherwise noted in table 17, the stress should be based on the net section. Where indicated in table 17, stress concentrations should be taken into account either by special analysis or by the factors given in figure 22 (see also H.1.2).

6.2.2 Shear stress may be neglected where it is numerically less than 15 % of a coexistent direct stress.

6.2.3 The peak and trough values of principal stress should be those on principal planes which are not more than 45° apart. This will be achieved if either

(a) $\sigma_x - \sigma_y$ is at least double the corresponding shear stress τ at both peak and trough, or

(b) the signs of $\sigma_x - \sigma_y$ and τ both reverse or both remain the same at the peak and the trough,

where

σ_x, σ_y and τ are the coexistent values with appropriate signs of the two orthogonal direct stresses and the shear stresses at the point under consideration.

In either (a) or (b), provided that $\sigma_x^2 \geqslant \sigma_y^2$ at both peak and trough, the required stress range will be the algebraic difference between the numerically greater peak principal stress and the numerically greater trough principal stress.

6.3 Stress in weld throats other than those attaching shear connectors. The reference stress for fatigue of a weld throat should be the vector sum of the shear stresses in the weld metal based on an effective throat dimension as defined in Part 3, and on the assumption that none of the load is carried in bearing between parent metals. This is illustrated in figure 2b. When calculating the stress range, the vector difference of the greatest and the least vector sum stress may be used instead of the algebraic difference.

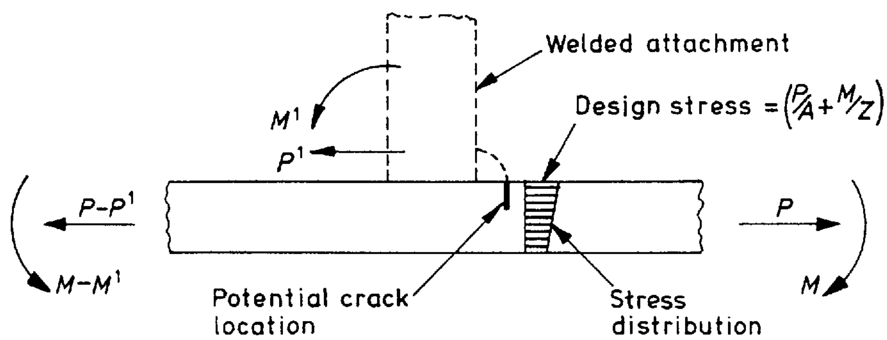


Figure 2a. Reference stress in parent metal

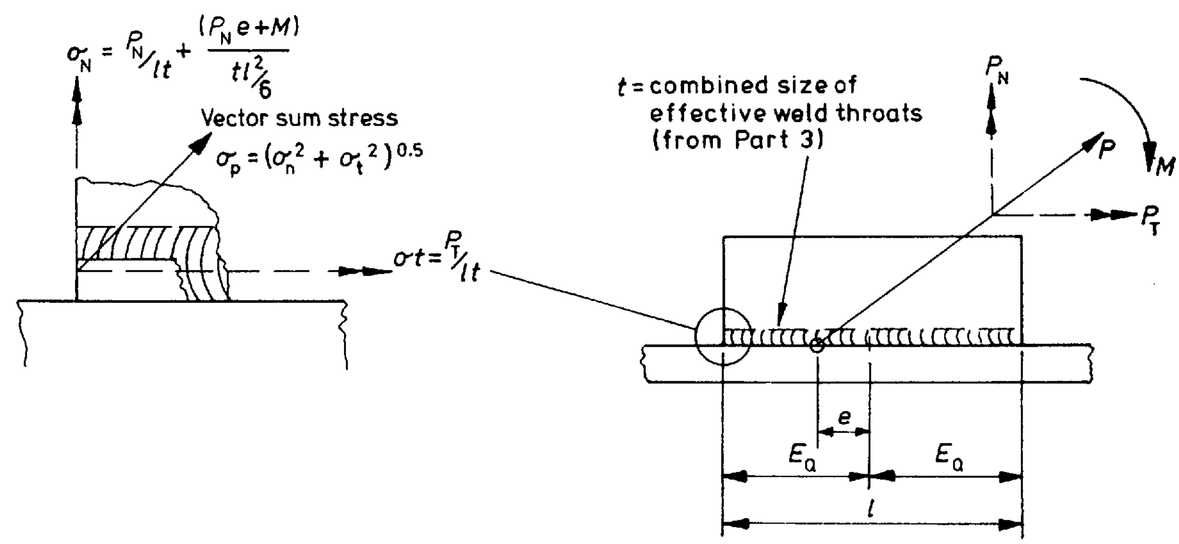


Figure 2b. Reference stress in weld throat

6.4 Stresses in welds attaching shear connectors

6.4.1 ***General.*** For shear connectors in accordance with the dimensional recommendations of Part 5, the design stresses for fatigue in the weld metal should be calculated in accordance with **6.4.2** and **6.4.3.** Where the dimensions of the shear connectors and/or the concrete haunches are not in accordance with Part 5, the fatigue strength should be determined in accordance with appendix G of this Part.

6.4.2 ***Stud connectors.*** The stresses in the weld metal attaching stud shear connectors should be calculated from the following expression :

stress in weld =

$$\frac{\text{longitudinal shear load on stud}}{\text{appropriate nominal static strength (from Part 5)}} \times 425\ \text{N/mm}^2$$

6.4.3 ***Channel and bar connectors***

6.4.3.1 The stresses in the weld metal attaching channel and bar shear connectors should be calculated from the effective throat area of weld, transverse to the shear flow, when the concrete is of normal density and from 0.85 × throat area when lightweight concrete is used. For the purposes of this clause the throat area should be based on a weld leg length which is the least of the dimensions tabulated below.

Channel connector	Bar connector
Actual leg length	0.6 × actual leg length
Thickness of channel web	0.125 × (height + breadth of bar)
Half the thickness of beam flange	half the thickness of beam flange

6.4.3.2 It may assist calculation to note that in normal density concrete, where the thickness of the beam flange is at least twice the actual weld leg length and the weld dimensions comply with Part 5, the effective weld areas are :

50 × 40 bar connectors × 200 mm long, 1697 mm^2
25 × 25 bar connectors × 200 mm long, 1018 mm^2
127 and 102 channel connectors × 150 mm long, 1272 mm^2
76 channel connectors × 150 mm long, 1081 mm^2

6.5 Axial stress in bolts. The design stress for fatigue in bolts complying with the requirements of BS 4395 and bolts to dimensional tolerances complying with the requirements of BS 3692 should be calculated from the following expression :

$$\text{stress in bolt} = \frac{F}{\sigma_u} \times \sigma_B$$

where

$F = 1.7\ \text{kN/mm}^2$ for threads of nominal diameter up to 25 mm

or

$F = 2.1\ \text{kN/mm}^2$ for threads of nominal diameter over 25 mm

σ_B is the stress range on the core area of the bolt determined on the basis of the minor diameter

σ_u is the nominal ultimate tensile strength of the bolt material in kN/mm^2

When subjected to fluctuating stresses, black bolts complying with the requirements of BS 4190 may only be used if they are faced under the head and turned on shank in accordance with the requirements of BS 4190.

7. Loadings for fatigue assessment

7.1 Design loadings. Highway and railway design loadings appropriate for bridges in the UK are given in **7.2** and **7.3** respectively.

The load factors γ_{fL} and γ_{f3} should be taken as equalling 1.0 (see Part 2).

7.2 Highway loading

7.2.1 ***General.*** In determining the maximum range of fluctuating stress, generally, only the vertical effects of vehicular live load as given in clause 7 should be considered, modified where appropriate to allow for impact as given in **7.2.4**. In welded members the dead load stress need not be considered. In unwelded members the dead load stress will have to be considered in determining the effective stress range when compression stresses occur (see **6.1.3**).

Centrifugal effects need only be considered for substructures (see **7.2.5**).

7.2.2 ***Standard loading***

7.2.2.1 *Standard load spectrum.* The standard load spectrum should be as shown in table 11 which gives the weight intensities and relative frequencies of commercial traffic on typical trunk roads in the UK. The minimum weight taken for a commercial vehicle is 30 kN. All vehicles less than 30 kN are ignored when considering fatigue.

7.2.2.2 *Standard fatigue vehicle.* The standard fatigue vehicle is a device used to represent the effects of the standard load spectrum ; for highway bridges this is a single vehicle with a weight of 320 kN. It consists of four standard axles with the dimensions as shown in figures 3 and 4.

NOTE. See appendix C for the derivation of the standard fatigue vehicle.

7.2.2.3 *Number of vehicles.* The numbers of commercial vehicles that are assumed to travel along each lane of a bridge per year should be taken from table 1. If for any reason vehicle numbers other than these are adopted, suitable adjustments may be made to the fatigue analysis in accordance with **8.2.3** or **8.3.2.1** (e).

7.2.3 ***Application of loading***

7.2.3.1 *Demarcation of lanes.* For the purposes of this Part of this British Standard the lanes should be the actual traffic lanes marked on the carriageway. They should be designated in accordance with figure 5 and the loading should be applied to the slow and the adjacent lanes only. Where a crawler lane is provided it should be treated as an additional slow lane.

7.2.3.2 *Path of vehicles.* The mean centre line of travel of all vehicles in any lane should be along a path parallel to, and within 300 mm of, the centre line of the lane as shown in figure 6. The transverse position of the centre line of the vehicle should be selected so as to cause the maximum stress range in the detail being considered. In some instances it may be found that the use of multiple paths results in significantly less calculated damage and guidance on this is given in **C.1.4.**

7.2.3.3 *Standard loading.* The passage of one standard fatigue vehicle along the entire length of one lane should be taken as one loading event.

7.2.3.4 *Non-standard load spectrum.* If a load spectrum is used, which differs in any way from the standard load spectrum, the passage of each vehicle forming the load spectrum should be considered to provide a separate loading event.

H.2.2 ***General comments.*** In welded construction, fatigue failure will rarely occur in a region of unwelded material since the fatigue strength of the welded joints will usually be much lower.

H.2.3 ***Comments on particular detail types***

Type 1.3. All visible signs of drag lines should be removed from the flame cut edge by grinding or machining.

Types 1.3 and 1.4. The presence of an aperture or re-entrant corner implies the existence of a stress concentration and the design stress should be the stress on the net section multiplied by the relevant stress concentration factor (see figure 22).

Type 1.4. The controlled flame cutting procedure should ensure that the resulting surface hardness is not sufficient to cause cracking.

Type 1.5. This type may be deemed to include bolt holes for attaching light bracing members where there is negligible transference of stress from the main member in the direction of σ_r.

Type 1.6. This covers connections designed in accordance with Part 3 for slip resistance at the ultimate limit state and where secondary out-of-plane bending of the joint is restrained or does not occur (i.e. double-covered symmetric joints). Failure initiates by fretting in front of the hole.

Type 1.12. This classification applies to failure at the root of the thread in normal commercial quality threaded components. Attention should be paid to the details of head fillets, waisted shanks and thread run-out in components, not covered by an appropriate British Standard, to ensure that they have satisfactory fatigue resistance. A higher fatigue resistance can be obtained with a rolled thread on material which has previously been fully heat-treated, but such components should be subject to special test and inspection procedures.

For the use of black bolts complying with the requirements of BS 4190 and subjected to fluctuating tensile loads, see **6.5**.

Where bolts or screwed rods are pre-tensioned to a value in excess of an applied external load, stress fluctuations will be governed by the elasticity of the pre-compressed elements. The increase in tension will rarely exceed 10 % of an external load applied concentrically with the bolt axis, but where the load is eccentric, a further increase will result from prying action.

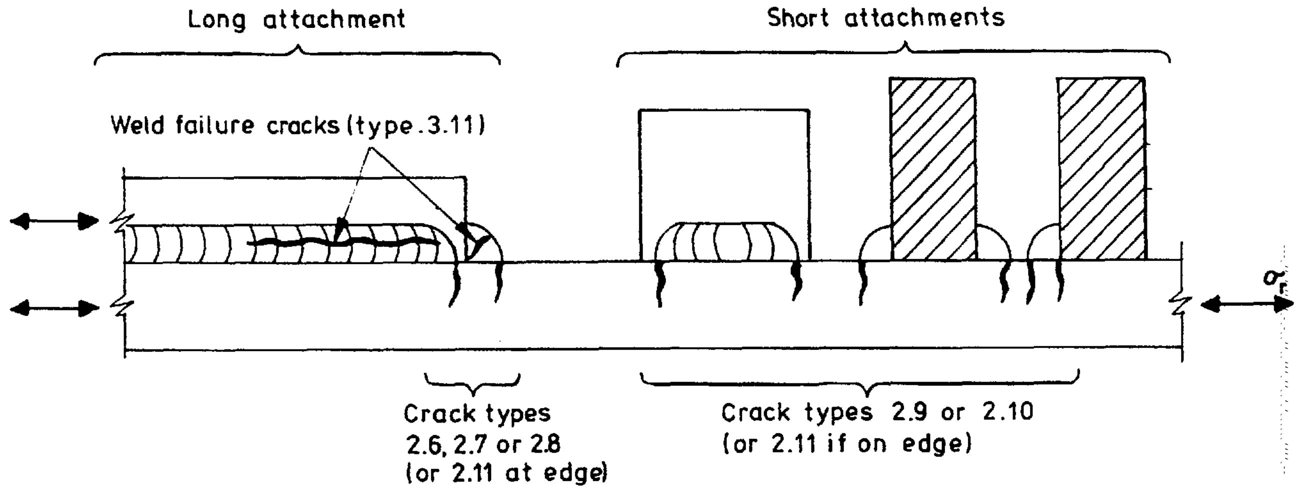
H.3 Type 2 classifications, welded details on surface of member. See table 17(b).

H.3.1 ***Notes on potential modes of failure.*** See figure 23.

When the weld is essentially parallel to the direction of stressing, fatigue cracks will normally initiate at the weld ends, but when the weld is transverse, cracking will initiate at the weld toes. In either case the cracks will then propagate into the stressed element. For attachments connected by single welds, cracks in parent metal may also initiate from the weld root. Cracks in stressed weld metal will initiate from the weld root (see type 3.11). Away from weld ends, fatigue cracks normally initiate at stop-start positions, or if these are not present, at weld surface ripples. With the weld reinforcement dressed flush, failure tends to be associated with weld defects.

H.3.2 ***General comments.***

H.3.2.1 *Edge distance.* (See figure 24.) No edge distance criterion exists for continuous or regularly intermittent welds away from the ends of an attachment (see types 2.1 to 2.5). However, a criterion exists (types 2.6 to 2.10) to limit the possibility of local stress concentrations occurring at unwelded corners as a result of, for example, undercut, weld



NOTE. For classification purposes, an 'attachment' should be taken as the adjacent structural element connected by welding to the stressed element under consideration. Apart from the particular dimensional requirements given for each type in table 17(b), the relative size of the 'stressed element' and the 'attachment' is not a criterion.

Figure 23. Failure modes at weld ends

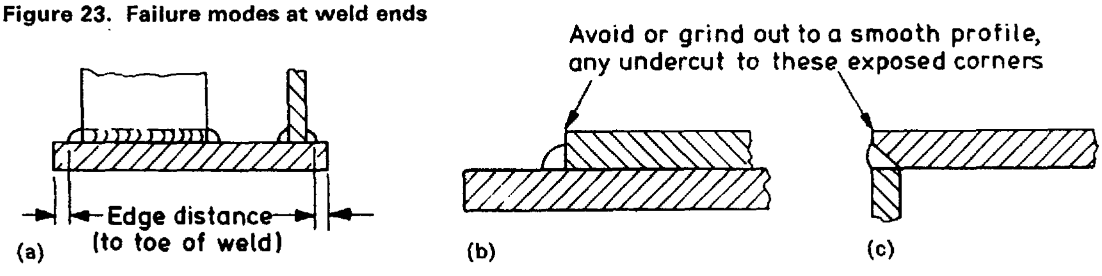


Figure 24. Edge distance

spatter and excessive leg size at stop-start positions or accidental overweave in manual welding. Although this criterion can be specified only for the 'width' direction of an element, it is equally important to ensure that no accidental undercutting occurs on the unwelded corners of, for example, cover plates or box members (see figure 24 (b) and (c)). Where it does occur, it should subsequently be ground out to a smooth profile.

Part 5 recommends the provision of a minimum edge distance of 25 mm for shear connectors, hence the criterion given in this part will automatically be met.

H.3.2.2 *Attachment of permanent backing strips.* If a permanent backing strip is used in making longitudinal butt welded joints it should be continuous or made continuous by welding. These welds and those attaching the backing strip should also comply with the relevant class requirements. The classification will reduce to E or F (type 3.3 or 3.4) at any butt welds in the backing strip or class E at any permanent tack weld (see **H.3.3**, type 2.4). It should be noted that transverse butt welds on backing strips may be downgraded by tack welds close to their ends (see **H.4.3**, type 3.4).

H.3.2.3 *Stress concentrations.* These are increased, and hence the fatigue strength is reduced, where:

(a) the weld ends or toes are on, or near, an unwelded corner of the element (see **H.3.2.1**);

(b) the attachment is 'long' in the direction of stressing, and as a result, transfer of a part of the load in the element to and from the attachment will occur through welds adjacent to its ends;

(c) such load transfer is through joints which are not symmetrical about both axes of cross section of the stressed element.

H.3.2.4 *Weld forms.* Full or partial penetration butt welded joints of T form (such as would connect attachments to the surface of a stressed element) should be completed by fillet welds of leg length at least equal to 25 % of the thickness of the attachment. The fillets exclude the possibility of an increase in stress concentration arising at an acute re-entrant angle between the element surface and the toe of the weld, and thus, in considering the effects on the stressed element, it is immaterial whether the attachment is fillet or butt welded to the surface, since a similar toe profile results in both cases.

H.3.2.5 *Tack welds.* Tack welds, unless carefully ground out or buried in a subsequent run, will provide potential crack locations similar to any other weld end. Their use in the fabrication process should be strictly controlled.

NOTE. Apart from the width transverse to σ_r, neither the shape of the end of an attachment nor the orientation or continuity of the weld at its end affects the class.

H.3.3 *Comments on particular detail types*

Type 2.1 Finish machining should be in the direction of σ_r. The significance of defects should be determined with the aid of specialist advice and/or by the use of a fracture mechanics analysis. The N.D.T. technique should be selected with a view to ensuring the detection of such significant defects. This type is only recommended for use in bridgeworks in exceptional circumstances.

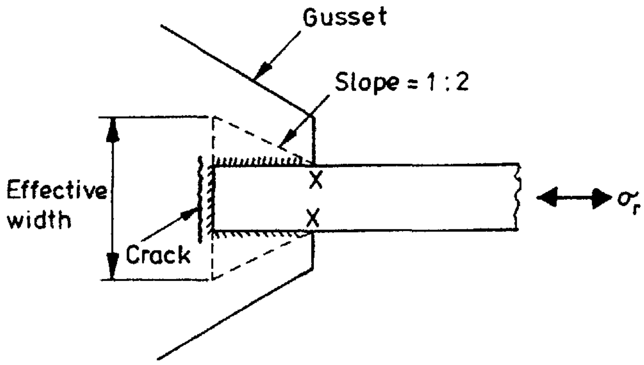
Type 2.2. Accidental stop-starts are not uncommon in automatic processes. Repair to the standard of a C classification should be the subject of specialist advice and inspection and should not be undertaken in bridgeworks.

Type 2.4. The limiting gap ratio m/h applies even though adjacent welds may be on opposite sides of a narrow attachment (as in the case of a longitudinal stiffener with staggered fillet welds). Long gaps between intermittent fillet welds are not recommended as they increase the risk of corrosion and, in the case of compression members, may cause local buckling (see Part 3). If intermediate gaps longer than 2.5 h are required the class should be reduced to F. This type also includes tack welds to the edges of longitudinal backing strips irrespective of spacing, provided that the welds comply in all respects with the workmanship requirements for permanent welds and that any undercut on the backing strip is ground smooth. The effects of tack welds which are subsequently fully ground out or incorporated into the butt weld by fusion, need not be considered.

Type 2.7. The classification may be deemed to include stress concentrations arising from normal eccentricities in the thickness direction.

This type includes parent metal adjacent to the ends of flange cover plates regardless of the shape of the ends.

Types 2.7 and 2.8. Where a narrow attachment is transferring the entire load out of a wide member, as in the case of a welded lap type connection between, for example, a cross brace and a gusset, the stress in the gusset at the end of the cross brace will vary substantially across the section. For assessing the stress in the gusset the effective width should be taken as shown in figure 25.



NOTE. For failure in the cross brace at X the cross brace is the 'member' and the gusset is the 'attachment'.

Figure 25. Effective width for wide lap connections

Type 2.10. This applies where any applied shear stress range is (numerically) greater than 50 % of a co-existent applied direct stress range.

Experimental evidence indicates that where significant shear stress co-exists with direct stress, the use of principal stress values may be conservative and accordingly the classification is upgraded.

Type 2.11. This type applies regardless of the shape of the end of the attachment. In all cases, care should be taken to avoid undercut on element corners or to grind it out to a smooth profile should it occur. In particular, weld returns across a corner should be avoided and the use of cover plates wider than the flange, to which they are attached, is not recommended.

H.4 Type 3 classifications, welded details at end connections of member. See table 17(c).

H.4.1 ***Notes on potential modes of failure.*** (See figure 26.) With the ends of butt welds machined flush with the plate edges, or as otherwise given below, fatigue cracks in the as-welded condition normally initiate at the weld toe and propagate into the parent metal, so that the fatigue strength depends largely upon the toe profile of the weld. If the reinforcement of a butt weld is dressed flush, failure can occur in the weld material if minor weld defects are exposed, e.g. surface porosity in the dressing area (see **H.4.3**, type 3.3).

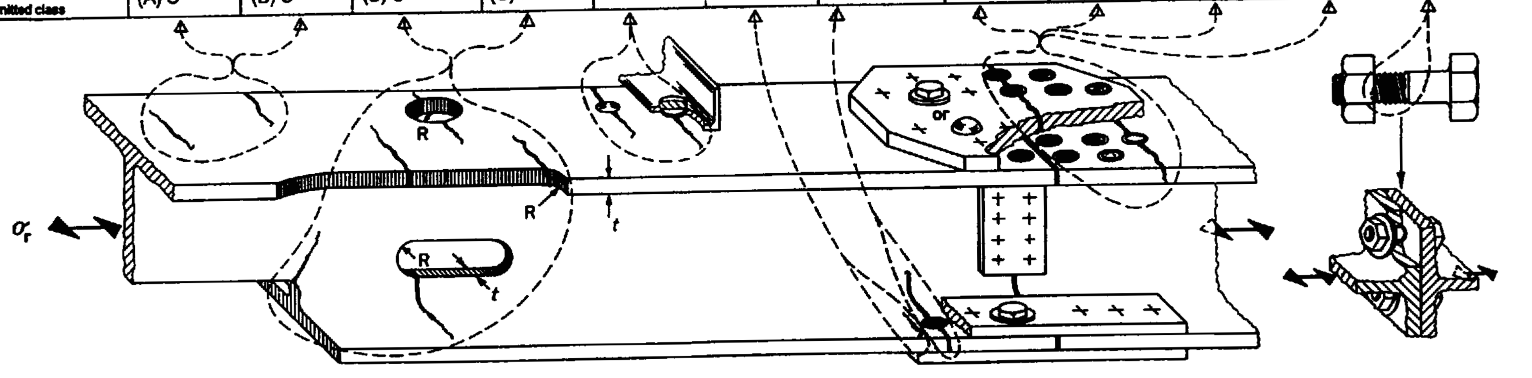
In the case of butt welds made on a permanent backing, fatigue cracks initiate at the weld metal-strip junction and then propagate into the weld metal.

In fillet or partial penetration butt welds, fatigue cracks in weld metal will normally initiate from the weld root.

Table 17. Classification of details

(a) Non-welded details

Product form	Rolled steel structural plates and sections											Threaded fasteners
Location of potential crack initiation	Away from all welding											In a butt joint, fastener axis parallel to σ_r
	Away from all structural connections					At a lapped or spliced connection fastened with:						
	On a member of constant or smoothly varying cross section		At any external or internal edge		At a small hole (may contain bolt for minor fixtures)	high strength friction grip bolts			rivets	precision bolts	black bolts	
						Away from hole	At a hole					In thread root
Dimensional requirements	No holes		Any aperture or re-entrant corner radius > t		Hole diameter < 3t	Double covered symmetrical joints only				Close tolerance hole		
	No re-entrant corners											
Manufacturing requirements (see also Part 6)	All surfaces fully machined and polished	Edges as rolled or machined smooth	Any flame cut edges subsequently machined or ground smooth		Hole drilled or reamered							Bolts to BS 3692 or BS 4395 Screw threads to BS 3643 : Part 2
						Tighten to BS 4604 : Parts 1, 2 and 3				Torque to full capacity or use lock nuts		
	No flame cutting			Any cutting of edges by planing or machine flame cutting with controlled procedure								
Special inspection requirements												
Design stress area			Net cross section			Gross	Net cross section					Core area (minor diameter)
Special design stress parameter			Use stress concentration factor for apertures or re-entrant corners			Designed for no slip at ultimate load (see Part 3)						See 7.5
Type number	1.1	1.2	1.3*	1.4*	1.5*	1.6*	1.7	1.8	1.9	1.10	1.11	1.12*
Maximum permitted class	(A) C	(B) C	(B) C	(C) D	D	C‡	C	D	D	E	G‡	B



Key Typical fatigue crack location Cut edge Fastener Direction of stress fluctuation

*See H.2.3.

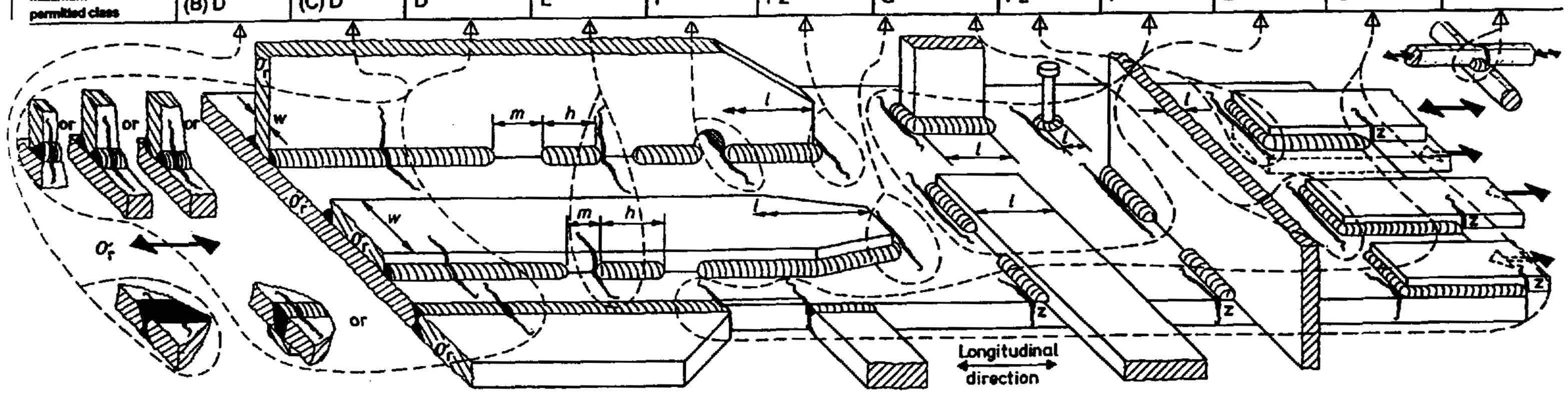
‡ Classifications that should be used with caution.

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are achieved.

NOTE. Tables 17(a), (b) and (c) are also available separately as a set of wall charts (BS 5400 : Part 10C).

(b) Welded details other than at end connections of a member

Product form	Rolled steel structural plates, sections and built-up members											Reinforcing steel in concrete
Location of potential crack initiation	At a long welded attachment (in direction of σ_r)								At a short welded attachment		At any attachment	At welded intersections in fabric or between hot rolled bars
	Away from weld end			At an intermediate gap in a longitudinal weld	At a cope hole	At a weld end						
					Narrow attachment		Wide attachment				Close to edge of member	
							On one side only	On both sides symmetrically				
Dimensional requirements	Butt weld full penetration			Fillet weld		Weld toe not less than 10 mm from member edge					Weld toe within 10 mm of member edge	
				Intermittent $\frac{m}{h} < 2.5$		Weld length (parallel to σ_r) $l > 150$ mm			$l < 150$ mm			
					Attachment width $w < 50$ mm		$w > 50$ mm					
Manufacturing requirements (see also Part 6)	Grind smooth any undercut on member edges										Grind any undercut	Resistance or manual plus grind smooth undercut
	Dress flush reinforcement	Automatic no stop-starts									Avoid weld returns round laps (see z)	
Special inspection requirements	Proved free of all significant defects											
Design stress area	Minimum transverse cross section of member at location of potential crack initiation											
Special design stress parameter									$\tau \leq 0.5\sigma$	$\tau > 0.5\sigma$		
Type number	2.1*	2.2*	2.3	2.4*	2.5	2.6	2.7*	2.8*	2.9	2.10*	2.11*	2.12
Maximum permitted class	(B) D	(C) D	D	E	F	F2	G	F2	F	E	G	D



Key Typical fatigue crack location Surface grinding Direction of stress fluctuation

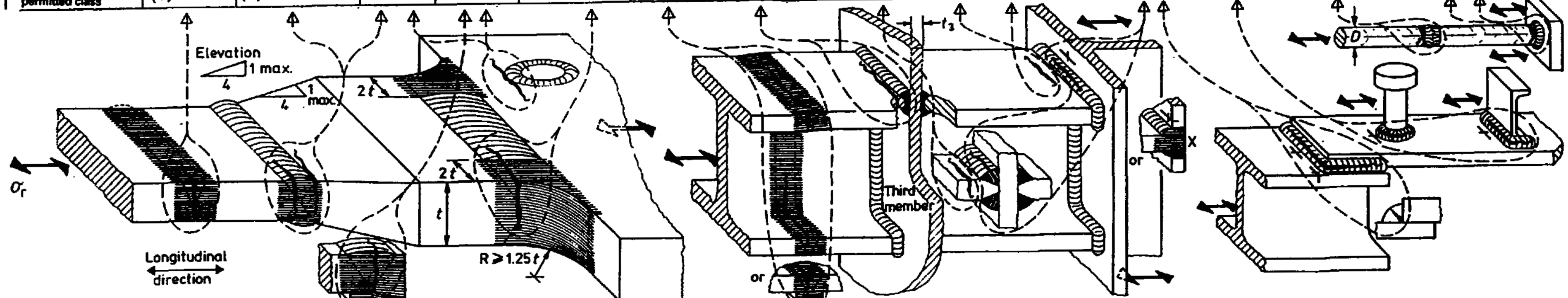
NOTE. Weld throat fatigue cracks are type 3 (see table 17(c)).

*See H.3.3.

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are archieved.

(c) Welded details at end connections of member

Product form	Rolled steel plates only					Rolled steel sections of built-up members						Shear connectors in concrete	Rolled steel reinforcing bars in concrete		
							(including plates)								
Location of potential crack initiation	At transverse weld joining:										In weld throat		At transverse weld		
	two single plates end to end					two members end to end			end of one member to side of another		In any incompletely fused joint	Between encased connector and any member	Between ends of bars	Between end of bar and surface of plate	
							with third member transverse through joint								
Dimensional requirements	Longitudinal axes in line								Full penetration butt weld	Partial penetration butt or fillet weld			Axes in line	Axes perpendicular	
	Full penetration butt weld							Partial penetration butt or fillet weld					Full penetration butt weld		Fillet weld leg size > 0.4 *D*
	Equal width	Any width change ≤ 1 in 4 slope			Abrupt width change	Similar profile							Slope at any diameter change ≤ 1 in 4	Plus reinforcing fillets with leg size ≥ 0.25 *D*	
	Equal thickness	Any thickness change ≤ 1 in 4 slope	[also includes plug welds (see footnote)]				[also applies if third member is narrow and slots through first (see appendix H)]								
Manufacturing requirements (see also Part 6)	Welded from both sides			On permanent backing strip	Build-up corners to radius > 1.25*t*	Misalignment slope ≤ 1 in 4	If width permits make weld continuous round joint otherwise grind weld ends flush (see X)						Flash butt or manual welding from both sides		
	Misalignment slope ≤ 1 in 4	Downhand shop welds		No permanent tack welds within 10 mm of edge			Maximum misalignment of main members < 0.5t_3								
	Dress flush reinforcement				Grind corners within 2*t*	Dress flush reinforcement									
	Temporary run-on and run-off plates used, weld and ground smooth														
	Grind smooth any undercut particularly on external corners												Grind smooth any undercut		
Special inspection requirements	Proved free of all significant defects						All regions stressed in through-thickness direction to be free from lamellar defects and tears							Check plate for lamellar defects and tears	
Design stress area	Minimum transverse cross section of member at location of potential crack initiation										'Effective' weld throat area		Minimum area of bar		
Special design stress parameter							Use the stress concentration factor unless third member is plate or has continuity plating		Stress concentration factor shall be used		See 6.3	See 6.4			
Type number	3.1*	3.2*	3.3*	3.4*	3.5*	3.6*	3.7*	3.8*	3.9*	3.10*	3.11*	3.12*	3.13*	3.14	3.15
Maximum permitted class	(C) E**	(D) E**	E**	F	F2	F2	F	F2	F	F2	W	S	(D) E**	F	F2



Key Typical fatigue crack location

Surface grinding

Direction of stress fluctuation

Permitted variation to joint dimensional requirements [ ]

*See H.4.3.

NOTE. Potential fatigue crack locations marked are type 2 (see table 17(b)).

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are achieved.

** The classes shown are compatible with the highest workmanship standards set in Part 6 of this British Standard.

Standards publications referred to

BS 1881	Methods of testing concrete
BS 3643	ISO metric screw threads
BS 3692	ISO metric precision hexagon bolts, screws and nuts
BS 4190	ISO metric black hexagon bolts, screws and nuts
BS 4395	High strength friction grip bolts and associated nuts and washers for structural engineering
BS 4604	The use of high strength friction grip bolts in structural steelwork. Metric series
BS 5135	Metal-arc welding of carbon and carbon manganese steels
BS 5400	Steel concrete and composite bridges

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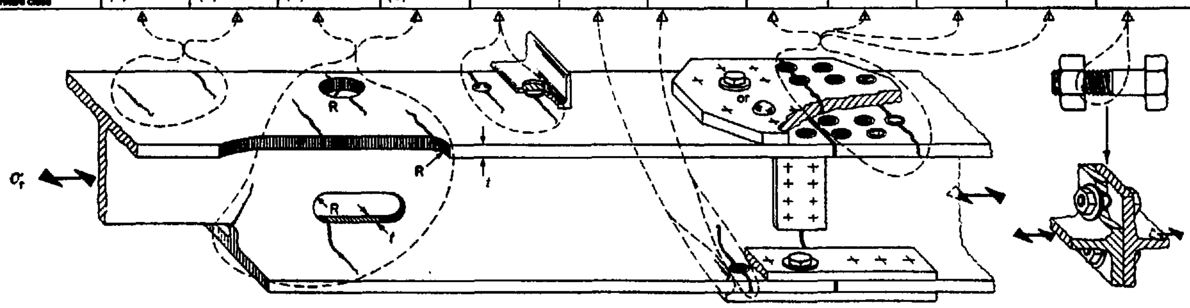
Table 17. Classification of details

(a) Non-welded details

Product form	Rolled steel structural plates and sections											Threaded fasteners
Location of potential crack initiation	Away from all welding											
	Away from all structural connections					At a lapped or spliced connection fastened with:						In a butt joint, fastener axis parallel to σ_r
	On a member of constant or smoothly varying cross section		At any external or internal edge		At a small hole (may contain bolt for minor fixtures)	high strength friction grip bolts			rivets	precision bolts	black bolts	
						Away from hole	At a hole					In thread root
Dimensional requirements	No holes No re-entrant corners		Any aperture or re-entrant corner radius > t		Hole diameter < $3t$	Double covered symmetrical joints only				Close tolerance hole		
Manufacturing requirements (see also Part 6)	All surfaces fully machined and polished	Edges as rolled or machined smooth	Any flame cut edges subsequently machined or ground smooth		Hole drilled or reamered							Bolts to BS 3692 or BS 4395 Screw threads to BS 3643 : Part 2
						Tighten to BS 4604 : Parts 1, 2 and 3				Torque to full capacity or use lock nuts		
	No flame cutting			Any cutting of edges by planing or machine flame cutting with controlled procedure								
Special inspection requirements												
Design stress area			Net cross section			Gross	Net cross section					Core area (minor diameter)
Special design stress parameter	*		Use stress concentration factor for apertures or re-entrant corners			Designed for no slip at ultimate load (see Part 3)						See 7.5
Type number	1.1	1.2	1.3*	1.4*	1.5*	1.6*	1.7	1.8	1.9	1.10	1.11	1.12*
Maximum permitted class	(A) C	(B) C	(B) C	(C) D	D	C‡	C	D	D	E	G‡	B



Key Typical fatigue crack location Cut edge Fastener Direction of stress fluctuation

*See H.2.3

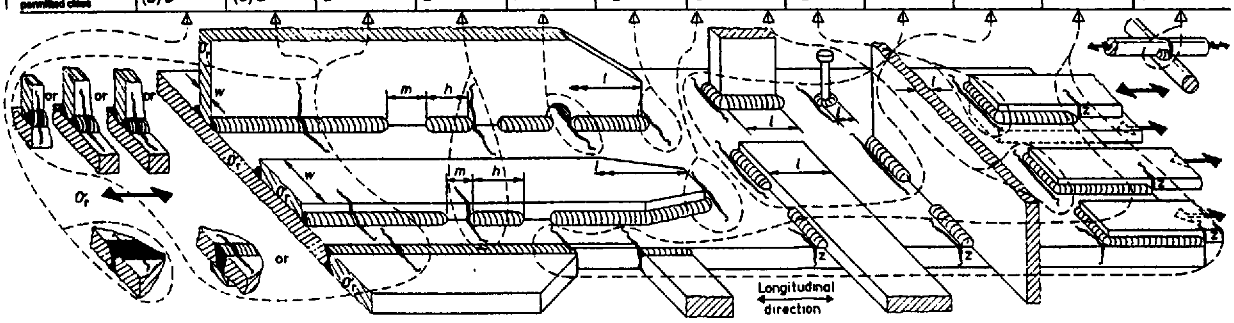
‡ Classifications that should be used with caution.

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are achieved.

NOTE Tables 17(a), (b) and (c) are also available separately as a set of wall charts (BS 5400 : Part 10C)

(b) Welded details other than at end connections of a member

Product form	Rolled steel structural plates, sections and built-up members											Reinforcing steel in concrete
Location of potential crack initiation	At a long welded attachment (in direction of σ_r)								At a short welded attachment		At any attachment	At welded intersections in fabric or between hot rolled bars
	Away from weld end			At an intermediate gap in a longitudinal weld	At a cope hole	At a weld end						
					Narrow attachment		Wide attachment				Close to edge of member	
							On one side only	On both sides symmetrically				
Dimensional requirements	Butt weld full penetration			Fillet weld		Weld toe not less than 10 mm from member edge					Weld toe within 10 mm of member edge	
				Intermittent $\frac{m}{h} < 2.5$		Weld length (parallel to σ_r) $l > 150$ mm			$l < 150$ mm			
					Attachment width $w < 50$ mm		$w > 50$ mm					
Manufacturing requirements (see also Part 6)	Grind smooth any undercut on member edges										Grind any undercut	Resistance or manual plus grind smooth undercut
	Dress flush reinforcement	Automatic no stop-starts									Avoid weld returns round laps (see z)	
Special inspection requirements	Proved free of all significant defects											
Design stress area	Minimum transverse cross section of member at location of potential crack initiation											
Special design stress parameter									$\tau < 0.5\sigma$	$\tau > 0.6\sigma$		
Type number	2.1*	2.2*	2.3	2.4*	2.5	2.6	2.7*	2.8*	2.9	2.10*	2.11*	2.12
Maximum permitted class	(B) D	(C) D	D	E	F	F2	G	F2	F	E	G	D



Key Typical fatigue crack location Surface grinding Direction of stress fluctuation

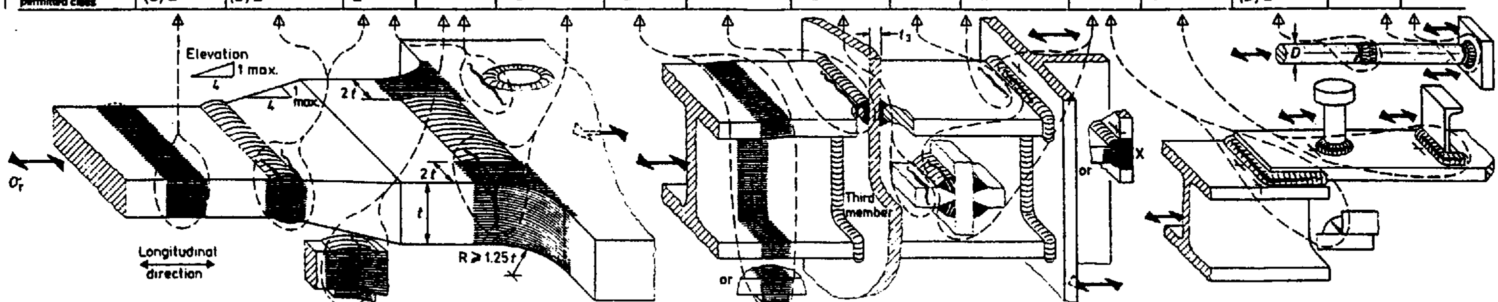
NOTE. Weld throat fatigue cracks are type 3 (see table 17(c))

*See H.3.3.

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are archieved.

(c) Welded details at end connections of member

Product form	Rolled steel plates only					Rolled steel sections of built-up members						Shear connectors in concrete	Rolled steel reinforcing bars in concrete		
							(including plates)								
Location of potential crack initiation	At transverse weld joining:										In weld throat		At transverse weld		
	two single plates end to end					two members end to end			end of one member to side of another		In any incompletely fused joint	Between encased connector and any member	Between ends of bars	Between end of bar and surface of plate	
							with third member transverse through joint								
Dimensional requirements	Longitudinal axes in line								Full penetration butt weld	Partial penetration butt or fillet weld			Axes in line	Axes perpendicular	
	Full penetration butt weld							Partial penetration butt or fillet weld					Full penetration butt weld		Fillet weld leg size ≥ 0.4 D
	Equal width	Any width change ≤ 1 in 4 slope			Abrupt width change								Slope at any diameter change ≤ 1 in 4	Plus reinforcing fillets with leg size ≥ 0.25 D	
	Equal thickness	Any thickness change ≤ 1 in 4 slope				Similar profile									
			[also includes plug welds (see footnote)]				[also applies if third member is narrow and slots through first (see appendix H)]								
Manufacturing requirements (see also Part 8)	Welded from both sides			On permanent backing strip	Build-up corners to radius ≥ 1.25t	Misalignment slope ≤ 1 in 4	If width permits make weld continuous round joint otherwise grind weld ends flush (see X)						Flash butt or manual welding from both sides		
	Misalignment slope ≤ 1 in 4	Downhand shop welds		No permanent tack welds within 10 mm of edge			Maximum misalignment of main members ≤ 0.5t_3								
	Dress flush reinforcement				Grind corners within 2t	Dress flush reinforcement									
	Temporary run-on and run-off plates used, weld and ground smooth														
	Grind smooth any undercut particularly on external corners												Grind smooth any undercut		
Special inspection requirements	Proved free of all significant defects						All regions stressed in through-thickness direction to be free from lamellar defects and tears							Check plate for lamellar defects and tears	
Design stress area	Minimum transverse cross section of member at location of potential crack initiation										'Effective' weld throat area		Minimum area of bar		
Special design stress parameter							Use the stress concentration factor unless third member is plate or has continuity plating		Stress concentration factor shall be used		See 6.3	See 6.4			
Type number	3.1*	3.2*	3.3*	3.4*	3.5*	3.6*	3.7*	3.8*	3.9*	3.10*	3.11*	3.12*	3.13*	3.14	3.15
Maximum permitted class	(C) E**	(D) E**	E**	F	F2	F2	F	F2	F	F2	W	S	(D) E**	F	F2



Key Typical fatigue crack location Surface grinding Direction of stress fluctuation Permitted variation to joint dimensional requirements [ ]

*See H.4.3.

NOTE Potential fatigue crack locations marked are type 2 (see table 17(b)).

( ) Classifications which may be used if special high levels of workmanship above those specified in Part 6 of this British Standard are achieved

** The classes shown are compatible with the highest workmanship standards set in Part 6 of this British Standard.