
Steel, concrete and composite bridges

Part 10. Code of practice for fatigue

Ponts en acier, ponts en béton, ponts mixtes
Partie 10. Code de bonne pratique relatif à la fatigue

Brücken aus Stahl, Beton und Verbundbau
Teil 10. Richtlinien bezüglich der Ermüdung

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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 :

- 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* Code of practice for bearings
- Part 10 Code of practice for fatigue

*In course of preparation.

British Standard

Steel, concrete and composite bridges

Part 10. Code of practice for fatigue

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.

3.1.12 σ_r - N relationship or σ_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 σ_r - N curve. The σ_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 σ_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 σ_r - N relationship
K_2	Parameter defining the σ_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 $\log \sigma_r / \log N$ curve
N	Number of repetitions to failure of stress range σ_r
N_1, N_2	Number of repetitions to failure of stress ranges $\sigma_{r1}, \sigma_{r2} \dots$ etc., corresponding to $n_1, n_2 \dots$ etc., repetitions of applied cycles
$n_1, n_2 \dots$ etc.	Number of applied repetitions of damaging stress ranges $\sigma_{r1}, \sigma_{r2} \dots$ 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 $\log N$

$\Sigma \frac{P}{N}$	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. Each structural steel detail is classified in accordance with table 17 (see 5.1.2).

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 magnitude and nature of stresses that will cause propagation of a crack and consequently reduce the number of stress repetitions to cause failure are affected by the presence of residual stress, inherent defects in welds and adjacent parent metal, surface defects and any other stress raisers interfering with the flow of stress. These are taken into account in the classifications given in table 17.

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 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 with caution see the footnote to table 17.)

5.1.2.5 The classifications of table 17 are valid for the qualities of hot rolled steel products and welds which meet the requirements of 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. In addition, the class of any critical detail which is classified as D or better should also be indicated on the 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 1. 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 $\sigma_x^2 \geq \sigma_y^2$ 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 2. 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.

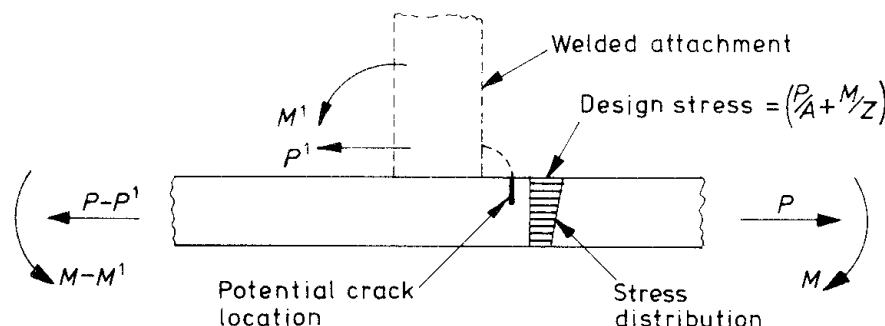


Figure 1. Reference stress in parent metal

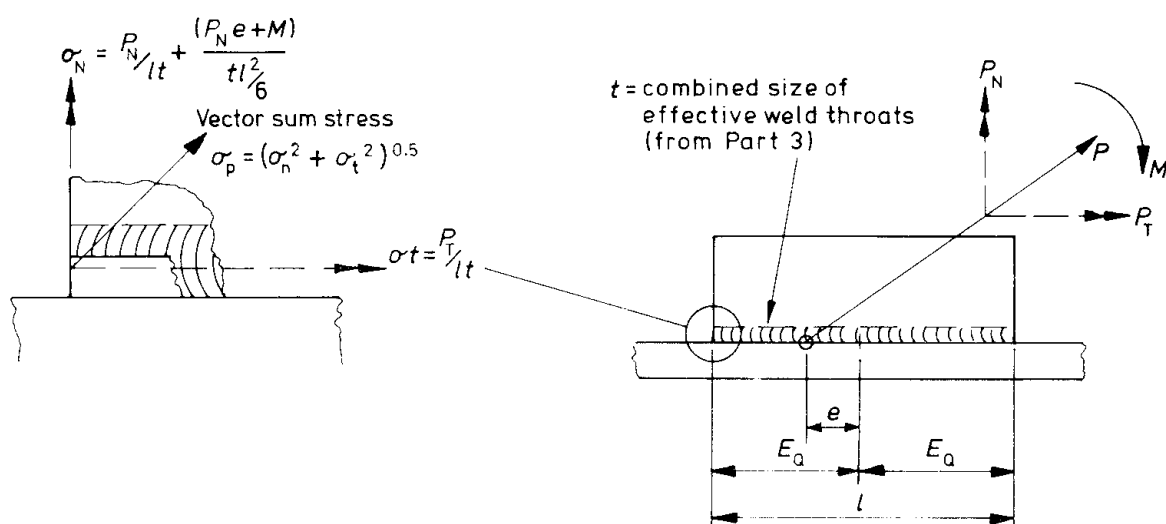


Figure 2. Reference stress in weld throat

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 :

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

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 $0.85 \times$ 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	$0.6 \times \text{actual leg length}$
Thickness of channel web	$0.125 \times (\text{height} \div \text{breadth of bar})$
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.

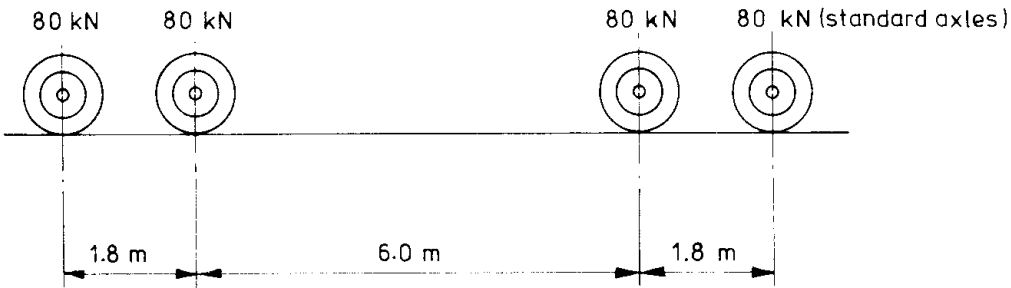


Figure 3. Axle arrangement of standard fatigue vehicle

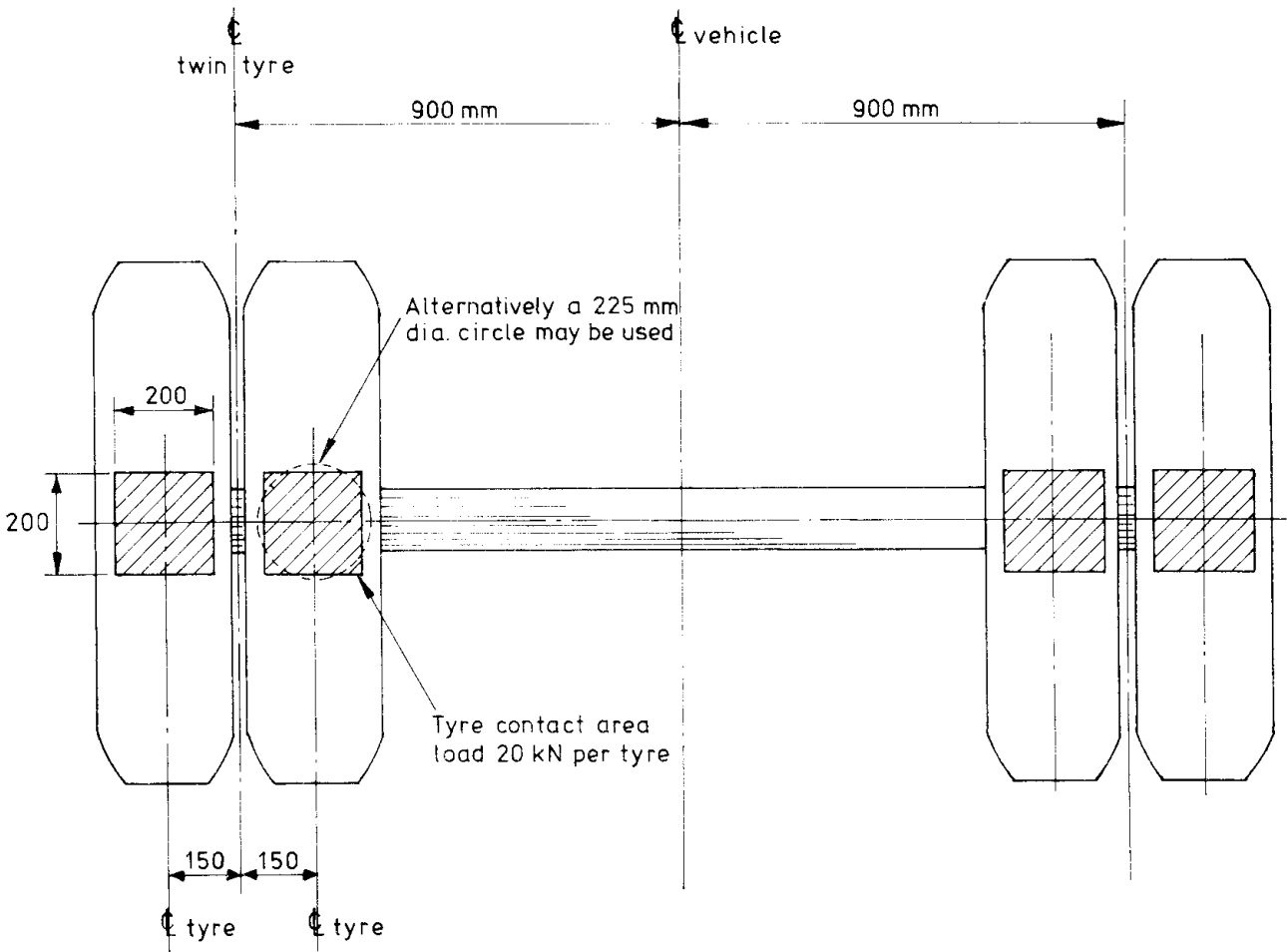


Figure 4. Plan of standard axle

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	Dual	2	1.5	1.0
All purpose	Dual	3		
All purpose	Dual	2		
Slip road	Single	2	1.0	Not applicable
All purpose	Single	3		
All purpose	Single (10 m*)	2		
Slip road	Single	1	0.5	Not applicable
All purpose	Single (7.3 m*)	2		

*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:

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

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

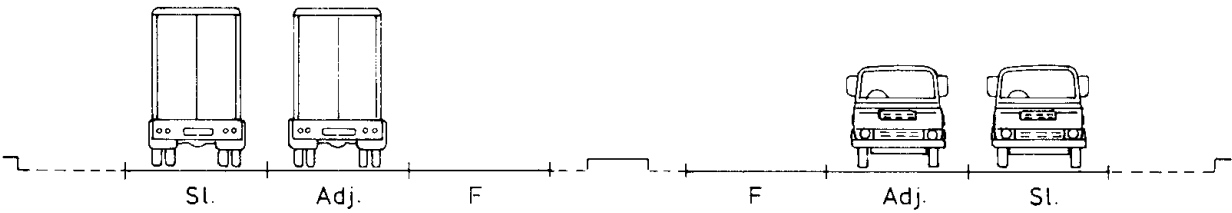
$$\frac{30\,000}{r + 150} \text{ kN}$$

7.3 Railway loading

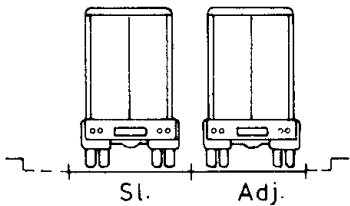
7.3.1 General. The loads to be considered should be the appropriate combination of the nominal live load, impact, lurching and centrifugal force, as specified in Part 2 of this British Standard.

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).

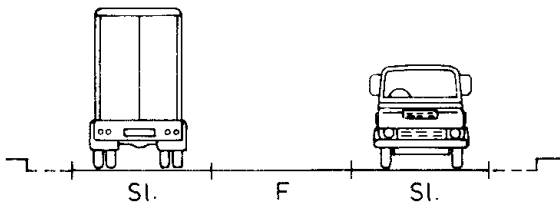
7.3.2 Application of loading. The loads should be applied to the appropriate lengths of the point load influence lines of not more than two tracks, so as to produce the algebraic maximum and minimum values of stress at the detail under consideration.



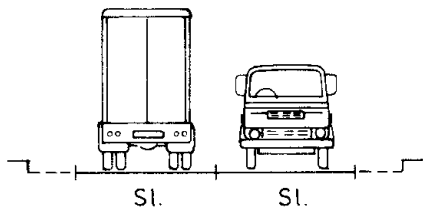
Two and three lane dual carriageways



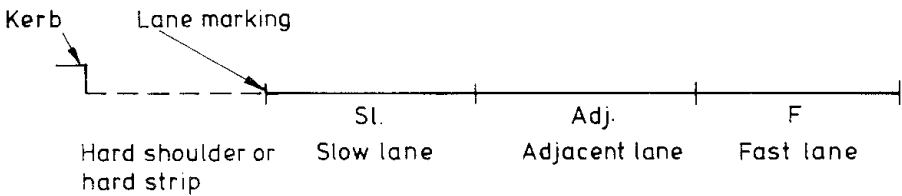
Two lane slip road



Three lane single carriageway

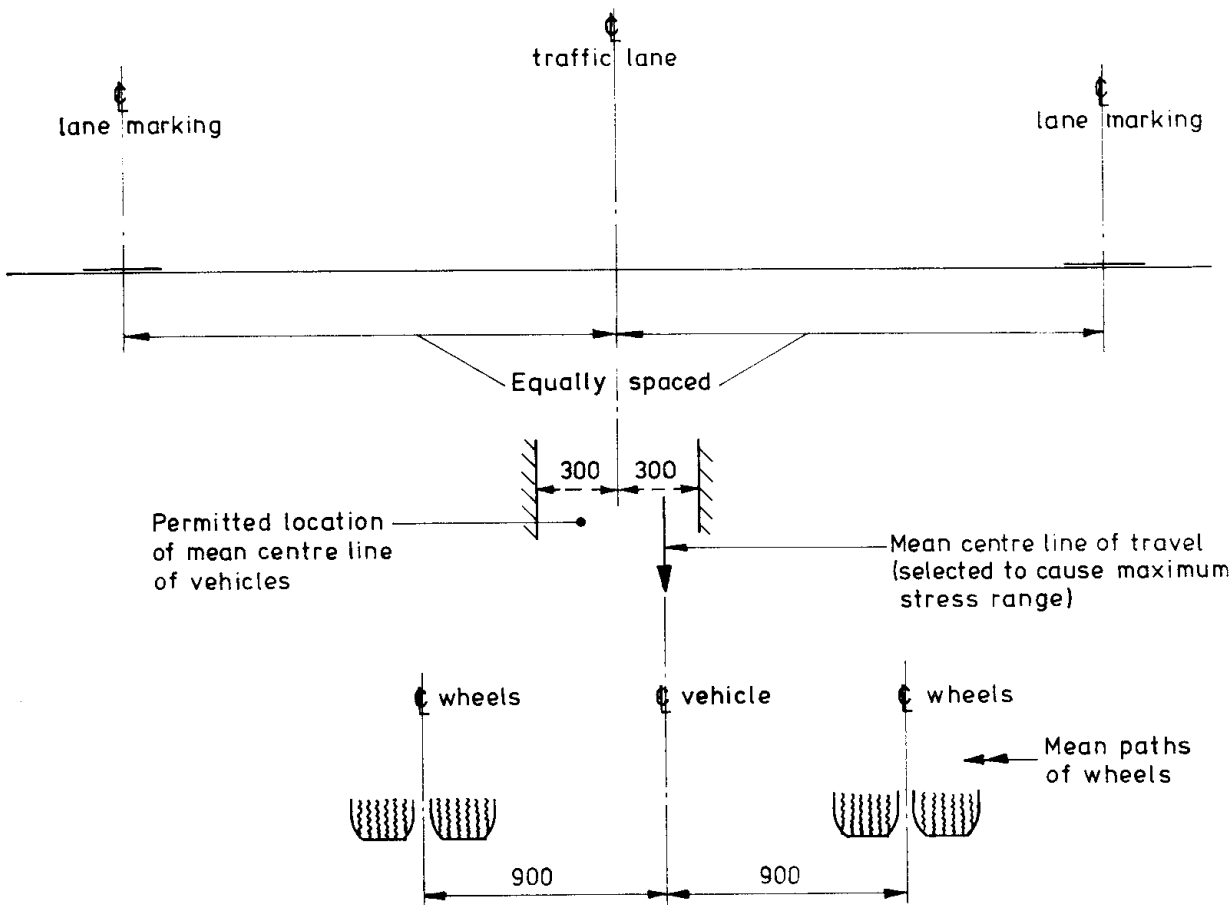


Two lane single carriageway



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

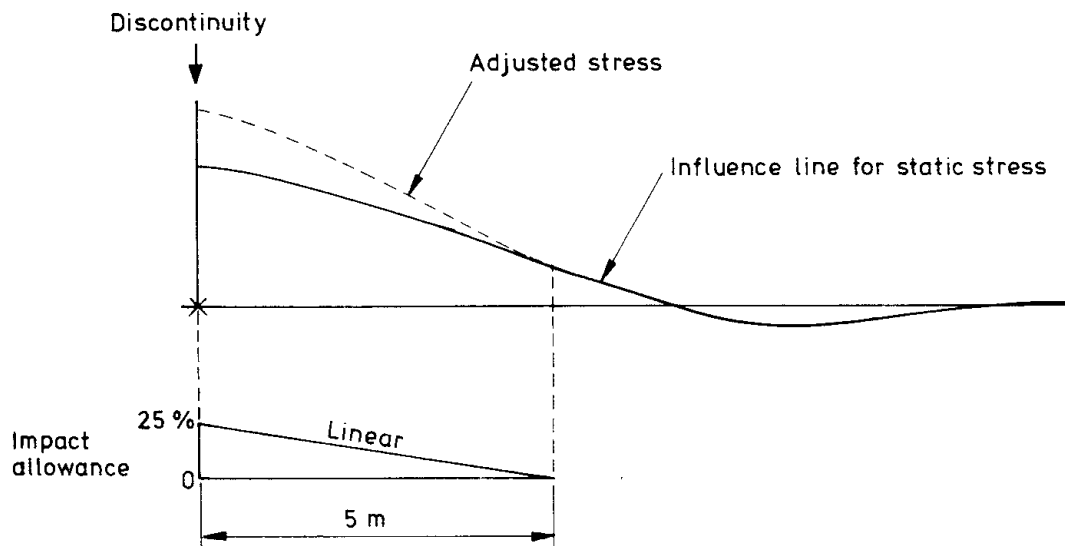


Figure 7. Impact allowance at discontinuities

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

Table 2. Standard load spectra for RU loading

Group number	Heavy traffic								Medium traffic								Light traffic							
	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 ($n_R \times 10^6$) 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
≥ 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 27×10^6 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 :

- the detail class is in accordance with table 17 ;
- the design life is 120 years ;
- the fatigue loading is the standard load spectrum (see 7.2.2.1) ;
- 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) :

- apply the standard fatigue vehicle to each slow and each adjacent lane in turn, in accordance with 7.2.3 ;
- 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

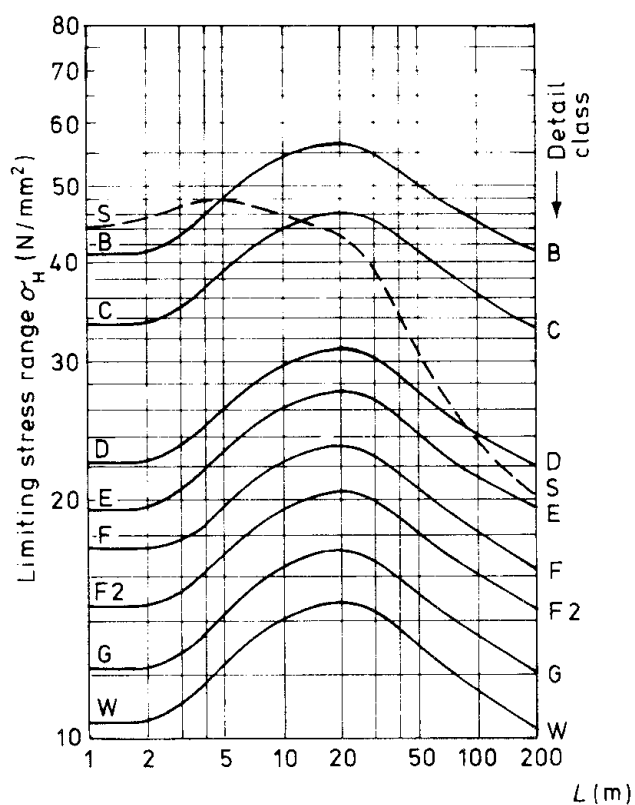
8.3.1 General. This method determines the fatigue life of the detail in question and may be used where a more precise assessment than that provided by the method of 8.2 is required or where the standard design life and/or the annual flows given in table 1 are not applicable. It should only be used where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

- the detail class is in accordance with table 17 but is not class S,
- the fatigue loading is the standard load spectrum (see 7.2.2.1).

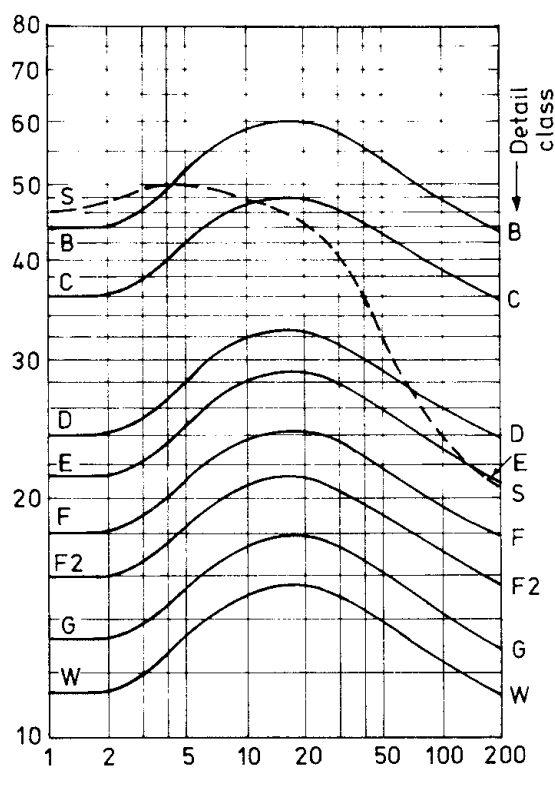
8.3.2 Procedure

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

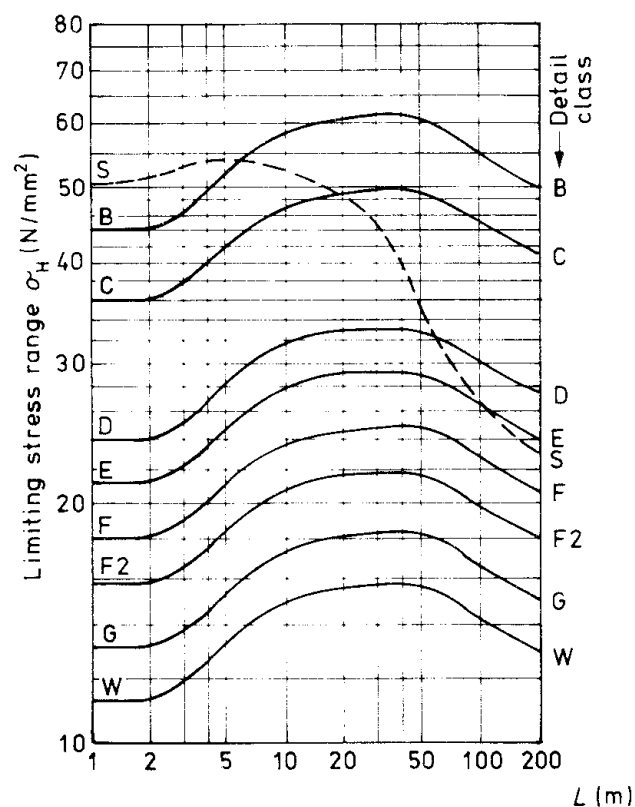
- Apply the standard fatigue vehicle to each slow lane and each adjacent lane in turn, in accordance with 7.2.3.
- Apply the impact allowance of 7.2.4, if appropriate, and determine the algebraic value of principal stress, or for weld throat, the vector sum stress at the detail being



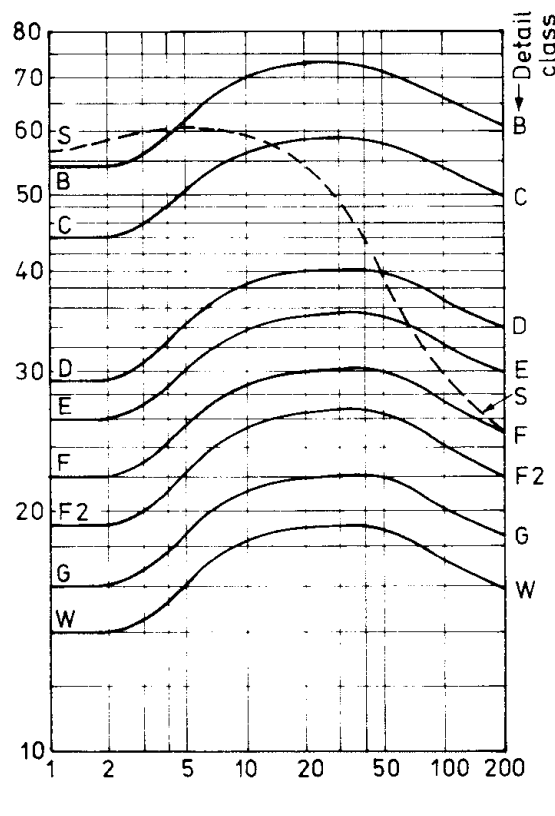
(a) Dual three lane motorway



(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, $\bar{n}_c$ 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 $\log \sigma_r / \log N$ 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

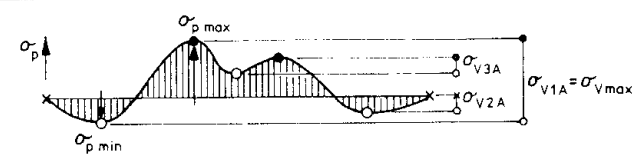
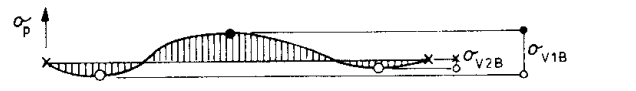
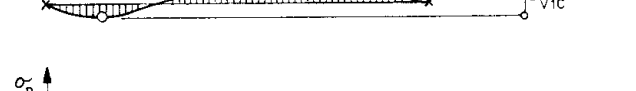
8.4.2.1 The individual stress spectra for the detail being assessed should be derived by traversing each vehicle in the load spectrum along the various lanes. Account should also be taken of the possibility of higher stress ranges due to some of the vehicles occurring simultaneously in one or more lanes 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.

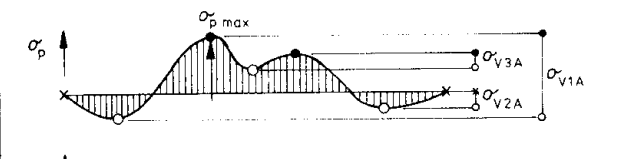
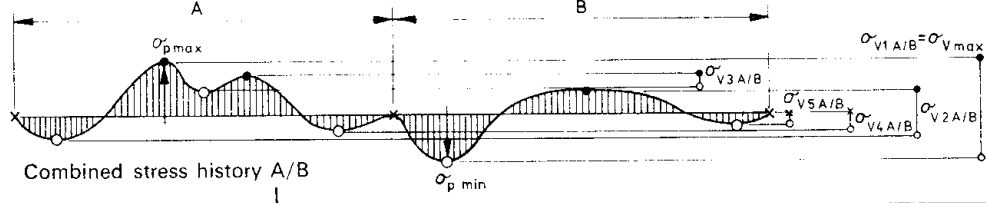
8.4.2.3 In assessing an existing structure, a design spectrum may be compiled from strain readings or traffic records obtained from continuous monitoring.

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.

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	n_{CA}^2
Lane B			2	n_{CB}^2
Lane C			1	n_{CC}
Lane D			1	n_{CD}
Combined stress history A/B			5	$n_{CA} \times n_{CB}$ $n_{CA} + n_{CB}$

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

Key

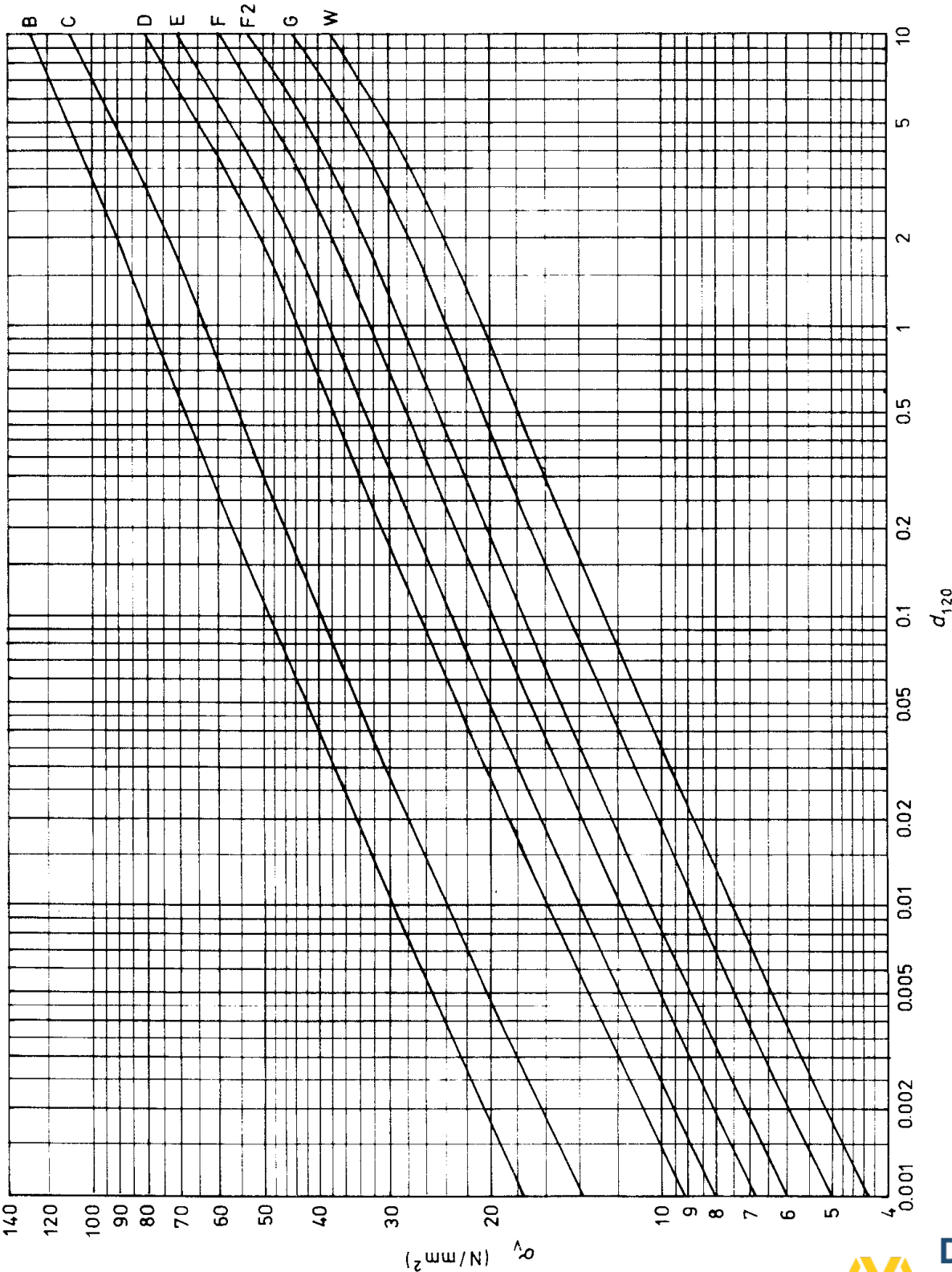
- Peak stress
- Trough stress
- X Datum stress

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})