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Government of India
Ministry of Railways
(Railway Board)

INDIAN RAILWAY STANDARD CODE OF PRACTICE FOR THE DESIGN OF STEEL OR WROUGHT IRON BRIDGES CARRYING RAIL, ROAD OR PEDESTRIAN TRAFFIC

(Steel Bridge Code)

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REVISED - 1962

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This Compilation is for educational reference only. For details refer original correction slips.

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INDIAN RAILWAY STANDARD

CODE OF PRACTICE FOR THE DESIGN OF STEEL OR WROUGHT IRON BRIDGES CARRYING RAIL, ROAD OR PEDESTRIAN TRAFFIC

(Steel Bridge Code)

1. SCOPE

1.1 This code is primarily intended to apply to the superstructure of simply supported steel bridges of spans up to 100 m (325 ft) between centres of bearings. Where appropriate, the provisions of the code may be adopted for larger spans or other types of steel bridges, but care should be taken, in these circumstances to make whatever amendments are necessary for fixity at the supports, continuity and other indeterminate or special conditions.

1.2 Where bridges of the through or semi-through type are adopted, they must be designed to allow for clearances specified in the appropriate schedule of dimensions, for different gauges in the case of Railway bridges or bridges over Railway, and in the case of road bridges clearances as specified by the appropriate authorities.

1.3 For road-bridges the design and construction shall comply with the Standard Specifications and Code of Practice for Road-bridges issued by the Indian Roads Congress.

1.4 Any revision or addition or deletion of the provisions of this code shall be issued only through the correction slip to this code. No cognizance shall be given to any policy directives issued through other means.

Note:-

1. *Unless otherwise specified the word 'span' shall mean effective span.*
2. *Where FPS equivalent are given the figures in the metric units are to be regarded as the standard. The FPS conversions are approximate. More accurate conversions should be based on IS: 786.*
3. *Attention is drawn to the fact that equations in the text, for which no units are specified, are applicable in any system of units, metric or FPS, provided the unit of length and the unit of force used in an equation are the same throughout.*

2. MATERIALS AND WORKMANSHIP

2.1 Materials and workmanship, including protection against atmospheric corrosion, shall comply with the Indian Railway Standard Specifications B-1, B-2 and B-6 and other specifications mentioned therein.

2.2 This code makes reference to the following standards:-

Indian Railway Standard Codes and Specifications

Welded Bridge Code – 1972

B-1 Steel girder bridges
B-2 Erection and riveting of bridge girders
B-6 The manufacture of locomotive turn- tables
M-2 Steel castings

Indian standards		**Amendments No.**
210-1962	Specification for grey iron casting	1 & 2
226-1969	Specification for structural steel (standard quality)	1 & 2
786-1967	Conversion factors and conversion tables	--
961-1962	Specification for structural steel (high tensile)	1 & 2
1148-1964	Specification for rivet bars for structural purposes.(Revised)	1
1149-1964	Specification for high tensile rivet bars for structural purposes	---
1367-1967	Technical supply condition for threaded fasteners.	---
1458-1965	Specification for Railway Bronze ingots and castings	1 to 4
1875-1971	Specification for carbon steel billets, blooms, slabs and bars for forgings subject to the following stipulations:-	

(i) Both chemical composition and mechanical properties to comply with specification requirements.

(ii) The maximum limits of sulphur and phosphorus are restricted to 0.040% each for class 3 and 4 steels.

and

(iii) Bend test requirements to be met as per specification.

2004-1970	Specification for carbon steel forgings for general engineering purposes with the additional stipulation of Bend Test to be carried out as per clause 8.2 of the specification.	---
2062-1969	Specification for Structural steel (fusion welding quality).	---
4000-1992	High strength Bolts in Steel Structures- Code of Practice	

NOTE:

Reference to Indian Standards, wherever appearing in this Code, shall mean the particular edition with amendments as indicated in this clause.

3. LOADS, FORCES AND STRESSES

3.1 Loads and Forces to be Taken into Account- For the purpose of computing stresses, the following items shall, where applicable be taken into account in accordance with the requirements specified in the Bridge Rules:-

(a) Dead load.
(b) Live load.
(c) Impact effect.
(d) Forces due to curvature and eccentricity of Track.
(e) Temperature effect.
(f) Resistance of expansion bearings to movements
(g) Longitudinal force.
(h) Racking force.
(j) Forces on parapets.
(k) Wind pressure effect.
(l) Forces and effects due to earthquake.
(m) Erection forces and effects.
(n) Derailment loads.

Subject to the provisions of other clauses, all forces shall be considered as applied and all loaded lengths chosen in such a way that the most adverse effect is caused on the member under consideration.

3.2 Combination of Loads and Forces- The following combination of forces shall be considered.

3.2.1 The worst combination possible of dead load with live load, impact effect and forces due to curvature and eccentricity of track. When considering the member whose primary function is to resist longitudinal and racking forces due to live load, the term live load shall include these forces.

3.2.2 In case of bridges situated in seismic zones I to III as given in Bridge Rules, only bridges of overall length more than 60 m or individual span more than 15 m for the worst possible combination of any or all the items ‘a’ to ‘j’ & ‘k’ or ‘l’ listed in clause 3.1

3.2.3 In cases of bridges situated in seismic zone IV & V as given in Bridge Rules, the worst combination possible of any or all the items ‘a’ to ‘j’ and ‘k’ or ‘l’ listed in clause 3.1

3.2.4 The worst combination possible of loads and forces during erection.

3.2.5 In case of ballasted deck bridges, the combination of dead load and derailment load shall be considered as an occasional load.

3.3 Primary and Secondary Stresses

3.3.1 Primary Stress- The primary stresses in the design of triangulated structures are defined as axial stresses in members calculated on the assumption that

all members are straight and free to rotate at the joints;
all joints lie at the intersection of the centroidal axes of the members;
all loads, including the weight of the members are applied at the joints.

3.3.2. **Secondary Stresses**- In practice the assumptions made in clause 3.3.1 are not realized and consequently members are subjected not only to axial stress, but also to bending and shear stresses. These stresses are defined as secondary stresses, and fall into two groups.

(a) Stresses which are the result of eccentricity of connections and of off-joint loading generally (e.g. load rolling direct on chords, self-weight of members and wind loads on members).

(b) Stresses, which are the result of elastic deformation of the structure and the rigidity of the joints. These are known as deformation stresses.

3.3.3. Structures shall be designed, fabricated and erected in such a manner as to minimise as far as possible secondary stresses. In the case of truss spans, ratios of width of the members (in the plane of distortion) to their lengths between centres of inter-sections may preferably be not greater than 1/12 for chord members and 1/24 for web members, in order to minimize the deformation stresses.

3.3.4. Secondary stresses which are the result of eccentricity of connections and off-joint loading generally (see clause 3.3.2(a)) shall be computed and combined with the co-existent axial stresses in accordance with clause 3.11.1, but secondary stresses due to the self-weight and wind on the member shall be ignored in this case.

Note:-

In computing the secondary stress due to loads being carried direct by a chord, the chord may be assumed to be a continuous girder supported at the panel points, the resulting bending moments, both at the centre and at the supports being taken as equal to ¾ of the maximum bending moment in a simply supported beam of span equal to the panel length. Where desired, calculations may be made and the calculated bending moments may be taken. In computing such bending moments, the impact allowance shall be based on a loaded length equal to one panel length.

3.3.5. In all cases of truss members deformation stresses described under clause 3.3.2(b) shall be either computed or assumed in accordance with clause 3.3.6 and added to the co-existing axial stresses.

3.3.6. In non-pre-stressed girders, deformation stresses mentioned under clause 3.3.2 (b) shall in the absence of calculation, be assumed to be not less than 16 $^{2}/_{3}$ per cent of the dead load and live load stress including impact.

3.3.7. In the case of pre-stressed girders, deformation stresses may be ignored. Girders shall not be designed for prestressing unless it is assured that the standard of workmanship in the fabrication and erection of girders will be such that correct prestressing can be relied on. When this is not the case, alternative of partial prestressing, i.e complete prestressing of chords with no or partial prestressing of web members, may be considered and the girder designed accordingly.

3.3.8. The effectiveness of prestressing in the web members of spans below 60m (200ft) and in all members of spans below 45m (150ft) shall be ignored.

3.3.9. All open web girders for railway bridges of spans 30.5 m (100ft) and above shall be prestressed. Rules for prestressing are given in APPENDIX-A.

3.4 Relief of Stresses- In determining the maximum stress in any member of a bridge, it is permissible to take into account any relief afforded to the member by adjoining parts. In determining the amount of relief, the secondary stresses, if any in the member shall be taken into account and considered with other co-existent stresses. Such relief may be taken into account only if the relieving parts have been suitably designed and are effectively attached to the member.

In every such case it is necessary to consider whether the relief considered will be given by the adjacent member permanently or is liable to vanish owing to any change in the said adjacent member.

3.5 Allowable working stresses for Combinations of Loads and Forces

3.5.1 For the forces of combination 3.2.1 above, the allowable working stresses shall be those stresses given in clauses 3.7 to 3.18 inclusive. Where secondary stresses are taken into account, the allowable working stresses may be increased by $16^{2}/_{3}$ per cent.

3.5.2 For the forces of combination 3.2.2 and 3.2.3 above, the allowable working stresses shall be those given in clauses 3.7 to 3.18 inclusive increased by $16^{2}/_{3}$ per cent. Where secondary stresses are also taken into account in the case of triangulated trusses, the basic permissible stresses given in clauses 3.7 to 3.18 inclusive, may be increased by $33^{1}/_{3}$ per cent.

3.5.3 For the forces of combination 3.2.4 above, the allowable working stresses shall be those given in clauses 3.7 to 3.18 inclusive, increased by 25 per cent. Where secondary stresses are also taken into account in the case of triangulated trusses, the basic permissible stresses given in clauses 3.7 to 3.18 inclusive, may be increased by 40 per cent. Additional material shall be added or other provisions shall be made to keep stresses during erection within the limit specified.

3.5.4 ***Stresses while Lifting of Span during Maintenance***- The end cross girders or other members which are used for lifting the span shall be so proportioned that the maximum stress during lifting including the stress due to dead load or any other co-existing load shall not exceed the permissible stress by more than 25 per cent.

3.5.5 In no case, will the stress in any member exceed the yield stress specified for the material.

3.5.6 The total variation in allowable stresses after combining the provisions of clauses 3.3 and 3.5 are given in TABLE 1. The values given in the TABLE 1 do not allow for the effect of fluctuations in stress which must be dealt with under clause 3.6 while stress arising from combinations of bending moments and shear are subject to provisions of clause 3.11.

TABLE – 1 TOTAL VARIATION IN ALLOWABLE STRESSES

Type of Girder	Increase of allowable stresses for stress combinations as per clauses		
	3.2.1	3.2.2 & 3.2.3	3.2.4
(a) Solid Web Girder For calculated primary stress	No increase	16 $^{2}/_{3}$%	25%
(b) Triangulated Trusses -			
(i) for calculated primary stress	No increase	16 $^{2}/_{3}$%	25%
(ii) where primary stresses are combined with calculated secondary stresses of sub clause 3.3.2 (a) (self wt. and wind on member ignored) and with deformation stresses of sub clause 3.3.2 (b)	16 $^{2}/_{3}$ %	33 $^{1}/_{3}$%	40%

3.6 Fluctuations of Stress (fatigue)

3.6.1 Fluctuations of stresses may cause fatigue failure of members or connections at lower stresses than those at which they would fail under static load. Such failures would be primarily due to stress concentrations introduced by the constructional details.

3.6.2 All details shall be designed to avoid as far as possible stress concentrations likely to result in excessive reductions of the fatigue strength of members or connections. Care shall be taken to avoid a sudden reduction of the section of a member or a part of a member, especially where bending occurs.

3.6.3 Stresses due to dead load, live load and impact, stresses resulting from curvature and eccentricity of track and secondary stresses as defined in clause 3.3.2 (a) only shall be considered for effects due to fatigue. All other items mentioned in clause 3.1 and secondary stresses as defined in clause 3.3.2(b) shall be ignored when considering fatigue.

3.6.4 For any structural member or connection, the fatigue design shall be done as per Appendix 'G" (Re-revised) for a specified 'Design life' and 'Fatigue Load Model'.

3.6.5 The fatigue life assessment shall normally be made for a standard design life of 100 years for a standard annual GMT of 50. However, any other design life/annual GMT may be used for design with the approval of Chief Bridge Engineer.

Note:-
No allowance for fatigue need be made in the design of Foot over bridges.

3.6.6. **Connection riveted or bolted-** The number of rivets and bolts shall be calculated without any allowance for fatigue but rivets or bolts subjected to reversal of stress during passage of live load shall be designed for the arithmetical sum of the maximum load plus 50% of the reversed load. In the case of wind bracings, the connection shall be designed to resist the greater load only.

3.6.7. The welds shall be designed according to the permissible stresses given in IRS Welded Bridge Code.

3.7 Permissible Stresses- Subject to the provision of clauses 3.3, 3.5, 3.6, 3.8 to 3.11 of this Code, structures shall be so designed that the calculated stresses in structural steel do not exceed the basic values given in TABLE II.

3.8 Allowable Working stresses for Parts in Axial Compression- The calculated average working stress in compression member shall not exceed the value given in TABLE IV or IV (a) derived from the Formula given below (see also APPENDIX-B)

$$P_{ac} = \frac{P}{1+(0.18+0.0008\,l/r)Sec.(l/r\sqrt{mP_{ac}/4E})RADIANS}$$

Where,

P_{ac} = allowable working stress on effective cross section for compression member.

P = values depending upon the yield stress f_y of the material (see TABLE III).

m = load factor = 1.7.

E = Young's modulus = 21,100 kg/mm^2 =(13,400 tons/sq in).

r = least radius of gyration of compression member.

l = effective length of the compression member (See clause 4.2).

TABLE II -- BASIC PERMISSIBLE STRESSES IN STRUCTURAL STEEL

Description	Mild steel to IS: 226 and IS: 2062 with yield stress of				High tensile steel grade 58-HTC to IS: 961 with yield stress of					
	26 kg/ mm^2	16.5 ton/in^2	24 kg/ mm^2	15.2 ton/in^2	36 kg/ mm^2	22.9 ton/in^2	35 kg/ mm^2	22.0 ton/in^2	33 kg/ mm^2	21.0 ton/in^2
1	2	3	4	5	6	7	8	9	10	11
Parts in Axial Tension On effective sectional area …	15.4	9.8	14.2	9.0	21.3	**13.5**	20.7	13.1	19.5	12.4
Parts in Axial Compression on Effective gross section …					**See**	**Clause**	3.8			
Parts in bending (Tension or Compression). On effective sectional area for extreme fibre stress – (i)For plates, flats, tubes, rounds, square and similar sections.	17.0	10.8	15.7	10.0	**23.5 See**	**14.9 also**	22.8 **Clause3.9**	14.5	21.6	13.7
(ii) For rolled beams, channels, angles and tees, and for plate girders with single or multiple webs with d_1/t not greater than 85 for steel to IS:226 and IS:2062 d_1/t not greater than 75 for steel to IS:961.	16.3	10.4	15.0	9.5	22.5 **See**	**14.3 also**	21.9 **Clause 3.9**	13.9	20.6	13.1
ii) For plate girder with single or multiple webs with : d_1 /t greater than 85 for steel to IS:226 and IS:2062, d/t greater than 75 for steel to IS:961.	15.4	9.8	14.2	9.0	21.3 **See**	**13.5 also**	20.7 **Clause 3.9**	13.1	19.5	12.4

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

(contd.)

TABLE II – (Contd.)

1	2	3	4	5	6	7	8	9	10	11
Parts in Shear Maximum shear stress (Having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure) …	11.1	7.0	10.2	6.5	15.3	9.7	14.9	9.5	14.0	8.9
Average shear stress (on the gross effective sectional area of webs of plate girders, rolled beams, channels, angles, tees) …	9.4	6.0	8.7	5.5	13.0	8.3	12.7	8.1	12.0	7.6
← For stiffened webs see clauses 5.8 and 5.10. →										
Parts in Bearing On flat surfaces …	18.9 kg/mm²(12.0 Ton/in.²);				26.0 kg/mm² (16.5 Ton/in.²).					

Description	Mild steel to IS:226 and IS:2062 and carbon steel (class 2) to IS:1875		High tensile steel Grade 58-HTC to IS:961 and Carbon Steel (class 4) to IS:1875	
	Kg/mm²	Ton/in.²	Kg/mm²	Ton/in.²
1	2	3	4	5
Pins In shear …	10.2	6.5	14.2	9.0
In bearing …	21.3	13.5	29.9	19.0
In bending …	21.3	13.5	29.9	19.0
For turned and fitted knuckle pins and spheres in bearing: On projected area …	11.8	7.5	11.8	7.5

Contd….

TABLE – II – *(Contd.)*

Description	Material of bolts as per IS: 1367 - Property Class				Rivets bars to --------			
	4.6		6.6		IS: 1148*		IS: 1149	
	Kg/mm²	Ton/in.²	Kg/mm²	Ton/in.²	Kg/mm²	Ton/in.²	Kg/mm²	Ton/in.²
1	2	3	4	5	6	7	8	9
Bolts and Rivets								
Parts in Axial Tension								
(a) On net section of bolts and studs.								
(i) Over 38 mm (1.1/2") dia …	14.2	9.0	19.7	12.5	….	….	….	….
(ii) 28 mm (1.1/8") and over including 38 mm (1.1/2") dia …	12.6	8.0	18.9	12.0	….	….	….	….
(iii) Less than 28 mm (1-1/8") dia but not less than 22 mm (7/8") dia.	11.0	7.0	16.5	10.5	….	….	….	….
(iv) Less than 22 mm (7/8") dia …	9.4	6.0	14.2	9.0	….	….	….	….
(b) On rivets …	….	….	….	….	9.4	6.0	11.1	7.0
				See	Clause	7.9		
Parts in Shear								
Average shear stress –								
(a) On power driven shop rivets and turned and fitted bolts …	10.2	6.5	14.2	9.0	10.2	6.5	14.2	9.0
(b) On power driven field rivets …	…	…	…	…	9.4	6.0	13.4	8.5
(c) On hand driven rivets …	…	…	…	…	8.7	5.5	…	…
(d) On black bolts ...	7.9	5.0	…	…	…	…	…	…
(e) On precision bolts and semi-precision bolts. …	9.4	6.0	13.4	8.5	…	…	…	…
*NOTE – *Rivets from drawn wire not included.*			See	Clauses	7.6, 7.7	and 7.8		

Table II (Contd…)

1	2	3	4	5	6	7	8	9
Parts in Bearing								
(a) On power driven shop rivets and turned and fitted bolts …	23.6	15.0	32.3	20.5	23.6	15.0	32.3	20.5
(b) On power driven field rivets…	…	…	…	…	22.0	14.0	30.7	19.5
(c) On hand driven rivets …	…	…	…	…	18.9	12.0	…	…
(d) On black bolts …	15.8	10.0	…	…	…	…	…	…
(e) On precision bolts and semi-precision bolts. …	22.0	14.0	30.7	19.5	…	…	…	…
			See	Clauses	7.6, 7.7	and 7.8.		
Welds	See I.R.S Welded Bridge Code							

Note:- *For steels to IS:226, IS:2062 and IS:961 a summary of guaranteed yield stress for various thicknesses is given below. For beams and channels, the thickness of the web governs.*

Description	**Guaranteed yield stress**						
	Mild steel to IS:226 and IS:2062			**High tensile steel grade 58-HTC to IS:961**			
	26 kg/mm²	**24 kg/mm²**	**23 kg/mm²**	**36 kg/mm²**	**35 kg/mm²**	**33 kg/mm²**	**30 kg/mm²**
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Nominal thickness/ diameter of plates, sections (for example, angles, tees, beams, channels, etc.), and flats. …	6 mm up to and including 20 mm.	Over 20 mm up to and including 40 mm.	Over 40 mm.	6 mm up to and including 28 mm.	Over 28 mm up to and including 45 mm.	Over 45 mm up to and including 63 mm.	Over 63 mm.
Bars (rounds, square and hexagonal) …	10 mm up to and including 20 mm.	Over 20 mm.	…	-do-	-do-	-do-	-do-

TABLE III – VALUES OF 'P' FOR VARIOUS VALUES OF f_y, THE YIELD STRESS FOR MILD STEEL AND HIGH TENSILE STEEL

Steel	Yield Stress fy		P	
	Kg/mm^2	Ton/in^2	Kg/mm^2	Ton/in^2
Mild steel to IS:226 & IS:2062 …	26.0	16.5	17.8	11.3
	24.0	15.2	16.5	10.5
HTS to IS:961 …	36.0	22.9	24.8	15.8
	35.0	22.2	24.1	15.3
	33.0	21.0	22.6	14.4

TABLE IV – ALLOWABLE WORKING STRESSES P_{ac} IN Kg/mm² ON EFFECTIVE CROSS SECTION FOR AXIAL COMPRESSION

l/r	Mild steel to IS:226 and IS:2062		High tensile steel to IS:961		
	P=16.5	P=17.8	P=22.6	P=24.1	P=24.8
0 …	13.98	15.08	19.15	20.42	21.02
20 …	13.66	14.72	18.63	19.85	20.42
40 …	12.99	13.95	17.43	18.48	18.98
60 …	11.82	12.59	15.18	15.92	16.25
80 …	10.07	10.57	12.04	12.35	12.58
100 …	8.07	8.32	9.04	9.21	9.28
120 …	6.30	6.43	6.78	6.86	6.00
140 …	4.94	5.01	5.20	5.25	5.27
160 …	3.93	3.98	4.09	4.11	4.12

TABLE IV (a) – ALLOWABLE WORKING STRESSES P_{ac} IN Ton/in.² ON EFFECTIVE CROSS SECTION FOR AXIAL COMPRESSION

l/r	Mild steel to IS:226 and IS:2062		High tensile steel to IS:961		
	P=10.5	P=11.3	P=14.4	P=15.3	P=15.8
0 …	8.88	9.57	12.16	12.97	13.35
20 …	8.67	9.35	11.83	12.60	12.97
40 …	8.25	8.86	11.07	11.73	12.05
60 …	7.50	7.99	9.64	10.11	10.32
80 …	6.39	6.71	7.64	7.84	7.99
100 …	5.12	5.28	5.74	5.85	5.89
120 …	4.00	4.08	4.30	4.35	4.38
140 …	3.14	3.18	3.30	3.33	3.35
160 …	2.49	2.53	2.60	2.61	2.62

3.9 Allowable Working Stresses in Bending- For all sectional shapes the tensile and compressive bending stresses, f_{bt} and f_{bc}, calculated according to clauses 5.1 to 5.3, shall not exceed the appropriate basic permissible stresses in clause 3.7 Table II subject to the provisions in clause 3.9.1 for bending compression

3.9.1 ***Bending Compression*** - For sectional shape with I_Y smaller than I_x

where I_Y = moment of inertia of the whole section about the axis lying in the plane of bending (the y-y axis)

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

The bending compression stress, f_{bc} shall not exceed the value P_{bc} given in Table VIII, corresponding to Cs the critical stress in the compression element calculated as follows:-

3.9.1.1 *for sections with a single web:* (including I sections with stiffened or unstiffened edges, channels, angles, tees, etc but excluding I sections where the thickness of one flange is more than 3 times the thickness of the other flange):

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

$$C_s = \frac{267730}{\left(l/r_y\right)^2}\sqrt{\left\{1+\frac{1}{20}\left(\frac{lt_e}{r_y D}\right)^2\right\}} kg/mm^2 = A$$

Except that the value of Cs calculated above shall be increased by 20 per cent for rolled beams and channels, and for plate girders provided that:

t_e/t is not greater than 2

d_1/t is not greater than 85, for steel to IS:226 and IS:2062

d_1/t is not greater than 75, for steel to Grade 58-HTC of IS:961.

In the above,

l = effective length of compression flange (see clause 5.4)

r_y= radius of gyration about the y-y axis of the gross section of the whole girder, at the point of maximum bending moment.

D= overall depth of girder, at the point of maximum bending moment

t_e= effective thickness of the compression flange

= K_1 x mean thickness of the horizontal portion of the compression flange at the point of maximum bending moment.

(For rolled section, $t_e=k_1$ x thickness given in reference books)

The coefficient K_1 makes allowance for reduction in thickness or breadth of flanges between points of effective lateral restraint and depends on R_a, the ratio of the total area of both

flanges at the point of least bending moment to the corresponding area at the point of greater bending moment between such points of restraint.
(for flanges of constant area K_1=1).

d_1 & t are as defined in table II for parts in bending.

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

Note:-
To obtain C_s in ton/sq in replace the constant 267730 in the above formula by 1,70,000

Value of K_1 for different values of R_a, are given in the Table V

TABLE V – VALUES OF K_1

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

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

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

$$C_s = \frac{267730}{\left(l/r_y\right)^2}\sqrt{\left\{1+\frac{1}{20}\left(\frac{lt_e}{r_y D}\right)^2\right\}} + K_2\frac{267730}{\left(l/r_y\right)^2}\ kg/mm^2 = A + K_2B$$

Where *l*, r_y and D are as defined in (a) above, and

t_e = effective thickness of flange

= K_1 x mean thickness of the horizontal portion of the flange of greater moment of inertia about the y-y axis of the girder, at the point of maximum bending moment, where K_1 is obtained from Table V

K_2 = A coefficient to allow for inequality of tension and compression flanges, and depends on R_m, the ratio of the moment of inertia of the compression flanges alone to that of the sum of the moments of inertia of the compression and tension flanges, each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment

Note:
1. *For flanges of equal moment of inertia R_m=0.5 and K_2 = 0*
   For tees and angles R_m=1.0 and K_2=0.5
2. *To obtain C_s in ton/in^2 replace the constant 267730 in the above formula by 1,70,000*

Value of K_2 for different values of R_m, are given in the Table VI

TABLE VI – VALUES OF K_2

R_m	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
K_2	0.5	0.4	0.3	0.2	0.1	0.0	-0.2	-0.4	-0.6	-0.8	-1.0

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

$$C_s = \left[\frac{267730}{(l/r_y)^2} \sqrt{\left\{ 1 + \frac{1}{20} \left(\frac{lte}{r_y D} \right)^2 \right\}} + K_2 \frac{267730}{(l/r_y)^2} \right] x \frac{Y_c}{Y_t} \; kg / mm^2$$

$= (A + K_2B) \times y_c / y_t$

Where l, r_y, D, $t_{e,\&}$ K_2 are as defined above and

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

Y_t = distance from neutral axis of girder to extreme fibre in tension

To obtain C_s in ton/in^2, replace the constant 267730 in the above formula by 170000.
Values of K_2 for different values of R_m are given in table VI

For tees and angles, $R_m = 0$ and $K_2 = -1$

Note :-

1. *For values of 'A' and 'B' for different ratios of l/r_y and D/t_e to be used for calculating C_s in kg/mm^2 refer Table VII and [C_s in tons/in^2 refer Table VII (a)]*

2. *For values of allowable bending compressive stress P_{bc} for different values of C_s see Table VIII*

3.9.1.2 For sections other than those described in clause 3.9.1.1. above:

a) Where the section is symmetrical about the x-x axis, the value of C_s may be obtained from the basic equation in the APPENDIX C

b) Where the section is not symmetrical about the x-x axis, the exact value of C_s may be computed: but values obtained from the formulae 3.9.1.1. (b) and 3.9.1.1.(c) can be used with safety.

TABLE VII – VALUES OF A & B TO BE USED FOR CALCULATING VALUES OF C_s IN kg/ mm²

Where $A = \frac{267730}{(l/r_y)^2}\sqrt{\left\{1+\frac{1}{20}\left(\frac{lt_e}{r_y D}\right)^2\right\}}$

$$B = \frac{267730}{(l/r_y)^2}$$

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

l/r_y	A																B
	$\frac{D}{t_e}$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40		251.0	224.6	208.7	198.6	191.8	186.9	183.2	177.6	174.7	172.8	171.5	170.1	169.1	168.4	168.0	167.4
45		212.5	187.7	172.6	162.8	156.2	151.5	147.9	142.5	139.5	137.5	136.4	134.7	134.0	133.2	132.9	132.2
50		184.1	160.6	146.5	137.0	130.7	126.1	122.7	117.3	114.3	112.4	111.2	109.8	109.0	108.2	107.7	107.1
55		162.4	140.3	126.8	117.8	111.7	107.2	103.8	98.7	95.8	93.9	92.6	91.2	90.4	89.5	89.1	88.5
60		145.2	124.4	111.5	103.0	97.0	92.8	89.5	84.4	81.4	79.7	78.4	77.0	76.2	75.4	75.0	74.3
65		131.5	111.8	99.5	91.3	85.7	81.6	78.3	73.2	70.4	68.7	67.4	66.0	65.2	64.4	63.9	63.3
70		120.0	101.4	89.8	81.9	76.4	72.4	69.5	64.4	61.6	59.8	58.6	57.3	56.5	55.8	55.3	54.6
75		110.6	93.1	81.7	74.3	69.0	65.0	62.0	57.3	54.5	52.8	51.7	50.2	49.5	48.7	48.2	47.6
80		102.5	85.7	75.1	67.9	62.7	59.1	56.1	51.5	48.7	46.9	45.8	44.4	43.6	42.9	42.5	41.9
85		95.6	79.5	69.4	62.5	57.5	53.9	51.2	46.6	43.8	42.2	41.1	39.7	38.9	38.1	37.8	37.0
90		89.5	74.3	64.6	58.0	53.1	49.6	46.9	42.5	39.8	38.1	37.0	35.6	34.8	34.0	33.7	33.1
95		84.3	69.6	60.3	54.0	49.3	45.8	43.3	38.9	36.4	34.6	33.5	32.3	31.5	30.7	30.4	29.6
100		79.5	65.5	56.5	50.4	46.0	42.7	40.2	35.9	33.4	31.8	30.7	29.3	28.5	27.9	27.4	26.8
110		71.5	58.7	50.4	44.7	40.6	37.5	35.1	31.0	28.7	27.1	26.0	24.7	23.9	23.1	22.8	22.2
120		65.0	53.2	45.5	40.2	36.4	33.4	31.2	27.2	25.0	23.5	22.4	21.1	20.3	19.7	19.2	18.6
130		59.7	48.7	41.6	36.5	32.9	30.1	28.0	24.2	22.0	20.6	19.5	18.3	17.6	16.9	16.5	15.9
140		55.1	44.9	38.1	33.4	30.1	27.4	25.4	21.9	19.7	18.3	17.3	16.1	15.4	14.6	14.3	13.7

TABLE VII – VALUES OF A & B TO BE USED FOR CALCULATING VALUES OF C_s IN kg/ mm²

Where $$A = \frac{267730}{\left(l / r_y\right)^2} \sqrt{\left\{1 + \frac{1}{20}\left(\frac{l t_e}{r_y D}\right)^2\right\}}$$

$$B = \frac{267730}{\left(l / r_y\right)^2}$$

Note – Where flanges are equal and of constant cross section C_s = A.

l/r_y	A																B
	$\frac{D}{t_e}$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
150		51.3	41.7	35.4	30.9	27.7	25.2	23.3	20.0	17.8	16.5	15.6	14.3	13.7	12.9	12.6	12.0
160		47.9	38.9	32.9	28.7	25.7	23.3	21.4	18.3	16.2	15.0	14.0	12.9	12.1	11.5	11.2	10.4
170		45.0	36.4	30.7	26.8	23.9	21.6	19.8	16.9	15.0	13.7	12.8	11.7	11.0	10.2	9.9	9.3
180		42.4	34.3	29.0	25.2	22.4	20.2	18.6	15.6	13.9	12.6	11.7	10.6	9.9	9.3	9.0	8.2
190		40.2	32.4	27.2	23.8	21.1	19.1	17.5	14.6	12.9	11.7	10.9	9.8	9.1	8.3	8.0	7.4
200		38.0	30.7	25.8	22.4	19.8	18.0	16.4	13.7	12.0	10.9	10.1	9.0	8.3	7.7	7.4	6.8
210		36.2	29.1	24.6	21.3	18.9	17.0	15.4	12.9	11.3	10.2	9.4	8.3	7.7	7.1	6.8	6.1
220		34.5	27.7	23.3	20.2	18.0	16.1	14.6	12.3	10.6	9.6	8.8	7.7	7.1	6.5	6.1	5.5
230		32.9	26.5	22.2	19.2	17.0	15.3	14.0	11.5	10.1	9.0	8.2	7.2	6.6	6.0	5.7	5.0
240		31.5	25.4	21.2	18.4	16.2	14.6	13.4	11.0	9.6	8.5	7.7	6.8	6.3	5.7	5.4	4.7
250		30.2	24.4	20.5	17.6	15.6	14.0	12.8	10.6	9.1	8.0	7.4	6.5	5.8	5.2	4.9	4.3
260		29.1	23.3	19.5	16.9	15.0	13.4	12.1	10.1	8.7	7.7	6.9	6.1	5.5	4.9	4.6	3.9
270		28.0	22.5	18.9	16.2	14.3	12.9	11.7	9.6	8.2	7.4	6.6	5.8	5.2	4.6	4.3	3.6
280		26.9	21.7	18.1	15.6	13.7	12.4	11.2	9.1	7.9	6.9	6.3	5.5	4.9	4.4	4.1	3.4
290		26.0	21.0	17.5	15.1	13.2	12.0	10.9	8.8	7.6	6.8	6.1	5.2	4.7	4.1	3.8	3.2
300		25.2	20.2	16.9	14.6	12.8	11.5	10.4	8.5	7.2	6.4	5.8	5.0	4.4	3.9	3.6	3.0

TABLE VII (a) – VALUES OF A & B TO BE USED FOR CALCULATING VALUES OF C_s IN Ton/ in.²

Where $$A = \frac{170000}{\left(l / r_y\right)^2} \sqrt{\left\{1 + \frac{1}{20}\left(\frac{l t_e}{r_y D}\right)^2\right\}}$$

$$B = \frac{170000}{\left(l / r_y\right)^2}$$

Note – Where flanges are equal and of constant cross section C_s = A.

l/r_y	A																B
	$\frac{D}{t_e}$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40		159.4	142.6	132.5	126.1	121.8	118.7	116.3	112.8	110.9	109.7	108.9	108.0	107.4	106.9	106.7	106.3
45		134.9	119.2	109.6	103.4	99.2	96.2	93.9	90.5	88.0	87.3	86.6	85.5	85.1	84.6	84.4	84.0
50		116.9	102.0	93.0	87.0	83.0	80.1	77.9	74.5	72.6	71.4	70.6	69.7	69.2	68.7	68.4	68.0
55		103.1	89.1	80.5	74.8	70.9	68.1	65.9	62.7	60.8	59.6	58.8	57.9	57.4	56.6	56.6	56.2
60		92.2	79.0	70.8	65.4	61.6	58.9	56.8	53.6	51.7	50.6	49.8	48.9	48.4	47.9	47.6	47.2
65		83.5	71.0	63.2	58.0	54.4	51.8	49.7	46.5	44.7	43.6	42.8	41.9	41.4	40.9	40.6	40.2
70		76.2	64.4	57.0	52.0	48.5	46.0	44.1	40.9	39.1	38.0	37.2	36.4	35.9	35.4	35.1	34.7
75		70.2	59.1	51.9	47.2	43.8	41.3	39.4	36.4	34.6	33.5	32.8	31.9	31.4	30.9	30.6	30.2
80		65.1	54.4	47.7	43.1	39.8	37.5	35.6	32.7	30.9	29.8	29.1	28.2	27.7	27.2	27.0	26.6
85		60.7	50.5	44.1	39.7	36.5	34.2	32.5	29.6	27.8	26.8	26.1	25.2	24.7	24.2	24.0	23.5
90		56.8	47.2	41.0	36.8	33.7	31.5	29.8	27.0	25.3	24.2	23.5	22.6	22.1	21.6	21.4	21.0
95		53.5	43.2	38.3	34.3	31.3	29.1	27.5	24.7	23.1	22.0	21.3	20.5	20.0	19.5	19.3	18.8
100		50.5	41.6	35.9	32.0	29.2	27.1	25.5	22.8	21.2	20.2	19.5	18.6	18.1	17.7	17.4	17.0
110		45.4	37.3	32.0	28.4	25.8	23.8	22.3	19.7	18.2	17.2	16.5	15.7	15.2	14.7	14.5	14.1
120		41.3	33.8	28.9	25.5	23.1	21.2	19.8	17.3	15.9	14.9	14.2	13.4	12.9	12.5	12.2	11.8
130		37.9	30.9	26.4	23.2	20.9	19.1	17.8	15.4	14.0	13.1	12.4	11.6	11.2	10.7	10.5	10.1
140		35.0	28.5	24.2	21.2	19.1	17.4	16.1	13.9	12.5	11.6	11.0	10.2	9.8	9.3	9.1	8.7

TABLE VII (a) – VALUES OF A & B TO BE USED FOR CALCULATING VALUES OF C_s IN Ton/ in.²

Where $$A = \frac{170000}{\left(l / r_y\right)^2} \sqrt{\left\{1 + \frac{1}{20}\left(\frac{l t_e}{r_y D}\right)^2\right\}}$$

$$B = \frac{170000}{\left(l / r_y\right)^2}$$

Note – Where flanges are equal and of constant cross section C_s = A.

l/r_y	A																B
	$\frac{D}{t_e}$	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
150		32.6	26.5	22.5	19.6	17.6	16.	14.8	12.7	11.3	10.5	9.9	9.1	8.7	8.2	8.0	7.6
160		30.4	24.7	20.9	18.2	16.3	14.8	13.6	11.6	10.3	9.5	8.9	8.2	7.7	7.3	7.1	6.6
170		28.6	23.1	19.5	17.0	15.2	13.7	12.6	10.7	9.5	8.7	8.1	7.4	7.0	6.5	6.3	5.9
180		26.9	21.8	18.4	16.0	14.2	12.8	11.8	9.9	8.8	8.0	7.4	6.7	6.3	5.9	5.7	5.2
190		25.5	20.6	17.3	15.1	13.4	12.1	11.1	9.3	8.2	7.4	6.9	6.2	5.8	5.3	5.1	4.7
200		24.1	19.5	16.4	14.2	12.6	11.4	10.4	8.7	7.6	6.9	6.4	5.7	5.3	4.9	4.7	4.3
210		23.0	18.5	15.6	13.5	12.0	10.8	9.8	8.2	7.2	6.5	6.0	5.3	4.9	4.5	4.3	3.9
220		21.9	17.6	14.8	12.8	11.4	10.2	9.3	7.8	6.7	6.1	5.6	4.9	4.5	4.1	3.9	3.5
230		20.9	16.8	14.1	12.2	10.8	9.7	8.9	7.3	6.4	5.7	5.2	4.6	4.2	3.8	3.6	3.2
240		20.0	16.1	13.5	11.7	10.3	9.3	8.5	7.0	6.1	5.4	4.9	4.3	4.0	3.6	3.4	3.0
250		19.2	15.5	13.0	11.2	9.9	8.9	8.1	6.7	5.8	5.1	4.7	4.1	3.7	3.3	3.1	2.7
260		18.5	14.8	12.4	10.7	9.5	8.5	7.7	6.4	5.5	4.9	4.4	3.9	3.5	3.1	2.9	2.5
270		17.8	14.3	12.0	10.3	9.1	8.2	7.4	6.1	5.2	4.7	4.2	3.7	3.3	2.9	2.7	2.3
280		17.1	13.8	11.5	9.9	8.7	7.9	7.1	5.8	5.0	4.4	4.0	3.5	3.1	2.8	2.6	2.2
290		16.5	13.3	11.1	9.6	8.4	7.6	6.9	5.6	4.8	4.3	3.9	3.3	3.0	2.6	2.4	2.0
300		16.0	12.8	10.7	9.3	8.1	7.3	6.6	5.4	4.6	4.1	3.7	3.2	2.8	2.5	2.3	1.9

TABLE VIII – ALLOWABLE WORKING STRESS P_{bc}. FOR DIFFERENT VALUES OF CRITICAL STRESS C_s .

(SEE ALSO CLAUSE 3.7 TABLE II)

C_s	P_{bc} for steel to IS:226 & IS:2062	P_{bc} for steel to Grade 58-HTC of IS:961	C_s	P_{bc} for steel to IS:226 & IS:2062	P_{bc} for steel to Grade 58-HTC of IS:961
Kg/mm²	Kg/mm²	Kg/mm²	Ton/in.²	Ton/in.²	Ton/in.²
3	1.5	1.5	2	1.0	1.0
4	2.0	2.0	3	1.5	1.5
5	2.5	2.5	4	2.0	2.0
6	3.0	3.0	5	2.4	2.4
7	3.5	3.5	6	2.8	2.8
8	3.8	3.8	7	3.2	3.2
9	4.2	4.2	8	3.5	3.6
10	4.6	4.6	9	3.9	4.0
12	5.3	5.4	10	4.2	4.4
14	6.0	6.2	12	4.7	5.1
16	6.7	7.0	14	5.1	5.7
18	7.2	7.7	16	5.5	6.3
20	7.6	8.4	17	5.7	6.6
22	8.0	9.0	18	5.9	6.9
24	8.4	9.6	20	6.3	7.5
26	8.8	10.2	22	6.6	8.0
28	9.2	10.8	24	7.0	8.4
30	9.6	11.4	26	7.2	8.8
35	10.5	12.7	28	7.5	9.2
40	11.2	13.7	30	7.7	9.5
45	11.9	14.6	35	8.2	10.1
50	12.4	15.3	40	8.6	10.7
55	12.9	15.9	45	8.9	11.1
60	13.3	16.5	50	9.1	11.5
65	13.6	17.1	55	9.4	11.8
70	13.9	17.4	60	9.6	12.1
75	14.1	17.8	70	9.9	12.6
80	14.4	18.2	80	10.0	13.0
90	14.9	18.8	90	10.0	13.3
100	15.3	19.4	100	10.0	13.6
125	15.8	20.5	110	10.0	13.8
150	15.8	21.2	120	10.0	14.0
200	15.8	22.2	127	10.0	14.1
215	15.8	22.4	135	10.0	14.2

3.10 Allowable Shear Stress in solid Webs of Plate Girders- The calculated average shear stress f_s on the effective sectional area of the web (see clause 4.3.2.3) shall not exceed the value given in TABLE II, clause 3.7

3.11. **Combined Stresses**

3.11.1 *Bending and Axial Stresses*- Members subjected to both axial and bending stresses (compressive or tensile) shall be so proportioned that the quantity

$$\frac{f_a^1}{F_a} + \frac{f_b^1}{F_b}$$ does not exceed unity

Where,

f^1_a= calculated axial stress (compressive or tensile)

F_a= appropriate allowable working stress in axially loaded members.

f^1_b=calculated maximum bending (compressive or tensile) stresses about both principal axes including secondary stresses, if any

F_b= the appropriate allowable working stress in bending (compressive or tensile)

Note:-

Where an increase or reduction in permissible working stress is specified, both F_a and F_b shall be the increased or reduced working stress ns directed in the relevant clauses.

3.11.2 *Shear and Bending Stresses* – The equivalent stress (see clause 3.11.4) 'f_e', due to a combination of shear stress 'f_s' , bending stress 'f_b', tensile or compressive is calculated from:

$$f_e = \sqrt{f_b^{\,2} + 3f_s^{\,2}}$$

3.11.3 *Shear, Bending, and Bearing stresses*- The equivalent stress 'f_e', (see clause 3.11.4) due to a combination of shear stress 'f_s' bearing stress 'f_p and bending stress 'f_b' tensile or compressive is calculated from:

$$f_e = \sqrt{f_b^{\,2} + f_p^{\,2} + f_b f_p + 3f_s^{\,2}}$$

3.11.4 Irrespective of the permissible increase of stress given in clauses 3.3 to 3.5 the equivalent stress 'f_e' calculated in clauses 3.11.2 and 3.11.3 above shall not exceed the following values given in TABLE IX.

TABLE IX-MAXIMUM PERMISSIBLE VALUES OF THE EQUIVALENT STRESS f_e FOR MILD AND HIGH TENSILE STEEL

Quality of steel	Yield Stress		Maximum value of f_e	
	Kg/mm^2	Ton/in^2	Kg/mm^2	Ton/in^2
Mild steel to IS: 226 and IS: 2062 …	26.0	16.5	24.0	15.2
	24.0	15.2	22.0	14.0
High tensile steel to IS: 961 …	36.0	22.9	33.0	21.0
	35.0	22.2	32.0	20.3
	33.0	21.0	30.0	19.1

3.12 Allowable Working Loads on Cylindrical Roller and Spherical Expansion Bearings

3.12.1 Cylindrical and spherical bearings shall be of forged steel to class 3 of IS:2004 and IS:1875 steel or alternatively turned from carriage and wagon axles, and the allowable working load shall not exceed the value given below:

3.12.2 *Cylindrical rollers on curved surfaces*- The allowable working load per unit length of roller shall be:

(a) For single and double rollers,

$$0.8\left(\frac{1}{1/D_1 - 1/D_2}\right) \text{ kg per mm of length}$$

$$0.5\left(\frac{1}{1/D_1 - 1/D_2}\right) \text{ ton per inch of length}$$

(b) For three or more rollers,

$$0.5\left(\frac{1}{1/D_1 - 1/D_2}\right) \text{ kg per mm of length}$$

$$0.32\left(\frac{1}{1/D_1 - 1/D_2}\right) \text{ ton per inch of length}$$

Where D_1 and D_2 are diameters of the convex and concave contact surfaces respectively.

3.12.3 *Cylindrical Rollers on Flat Surfaces*- The allowable working load per unit length of roller shall be:

(a) For single and double rollers

0.8 D_3 kg per mm of length

0.5 D_3 ton per inch of length

(b) For three or more rollers

0.5 D_3 kg per mm of length

0.32 D_3 ton per inch of length

Where D_3 is the diameter of the roller

Note:- No roller shall be lesser of diameter than 102 mm (or 4 inches)

3.12.4 The allowable working load on spherical bearings shall be:

$$\frac{1}{127}\left[\frac{1}{1/D_1 - 1/D_2}\right]^2 kg$$

or $$\frac{1}{200}\left[\frac{1}{1/D_1 - 1/D_2}\right]^2 ton$$

Where D_1 and D_2 are defined in clause 3.12.2 above.

3.13 Allowable Working Pressure on Sliding Bearings- The allowable working pressure for steel sliding on hard copper alloys to IS: 1458 shall not exceed 3.2 kg/mm^2 (2ton/in^2)

3.14 Basic Permissible Stresses for Cast Steel in Bearings- The basic permissible stresses for cast steel to IRS M2, class 'C' large and important casting with a minimum tensile strength of 47.25 kg/mm^2 (30 ton/in^2) and with a minimum elongation of 20 per cent in bearings shall not exceed the basic permissible stresses specified in clause 3.7, TABLE II for mild steel to IS: 226 with yield stress of 24.0 kg/mm^2 (15.2 tons/in^2)

3.15 Cast Iron- Cast iron shall not be used in any portion of the structure of a bridge carrying a railway except when subject only to direct compression but may be used in other bridges when subject to bending or compression. The basic permissible stresses in the cast iron conforming to IS: 210-1962 shall not exceed the $\frac{Grade No.}{4} kg/mm^2$ ($\frac{Grade No.}{6.28} tons/in^2$) in tension and 15.7 kg/mm^2 (10 tons/in^2) in compression.

The Grade No. corresponds to the minimum tensile strength in kg/mm^2 of 30 mm dia cast test bar (Table I of IS: 210-1962)

3.16 Allowable Working Pressure under Bearings or Bed Plates – The area of bearings or bed plates shall be so proportioned that when the eccentricity of loads due to combination mentioned in Clause 3.2.1 the maximum pressure on material forming the bed shall not exceed the following limits: -

Granite 36 kg/cm^2 (33 tons/ft^2)

Sand Stone 29.5 kg/cm^2 (27 tons/ft^2)

Cement Concrete**:** As laid down for permissible bearing pressure in Plain concrete in Table III and III(a) of the IRS Concrete Bridge Code-1962.

Reinforced Concrete**:** As laid down for permissible stress in direct compression for the specified crushing strength at 28 days for ordinary Portland cement (or the equivalent period of time for other cement) given in Table III and III(a) of IRS Concrete Bridge Code-1962

The above-mentioned limits may be exceeded by $33^1/_3$ per cent for combinations mentioned in clauses 3.2.2 and 3.2.3

The centre of pressure under flat bearing plates attached to the girders shall be assumed to be at one-third of the length from the front edge.

3.17 Slab Bases for Bearings – The effective area for distributing the load to the foundation shall be taken as the contact area of the member communicating the load to the slab plus the area given by a projection of twice the thickness of the slab around the contact area of the member.

3.18 Basic Permissible Stresses in Wrought Iron and Mild Steel of Early Manufacture- Subject to the provisions in clauses 3.19 and 3.20 the basic permissible stresses in wrought iron and mild steel of early manufacture shall be the appropriate percentage given in terms of basic permissible stresses for mild steel to IS: 226 with the yield stress of 24.0 kg/mm^2 (15.2 tons/in^2) as given below:-

For parts in tension … $66\ ^2/_3$ per cent

For parts in compression … 60 percent subject to a maximum of 7.8 kg/mm^2 (5tons/in^2)

For parts in shear	…	75 per cent
For parts in bearing	…	66 $^{2}/_{3}$ per cent
Pins:		
In shear	…	66 $^{2}/_{3}$ per cent
In bearing	…	66 $^{2}/_{3}$ per cent
In bending	…	66 $^{2}/_{3}$ per cent
Knuckle pins in bearing	…	85 per cent

3.19 Special Notes on Working Stresses

3.19.1 Where there is any doubt as to the quality of steel, it should be treated as mild steel of early manufacture and the stresses given in clause 3.18 shall be adopted, unless tests are made as specified in APPENDIX D in which case the safe working stresses as defined therein shall be adopted. In general, steel manufactured prior to 1895 may be assumed as steel of early manufacture.

3.19.2 Where there is doubt as to the strength or quality of wrought iron, tests should be made as specified in APPENDIX D and working stresses determined by the method laid down therein.

3.20 Existing Bridges

3.20.1 *Rivets*-The stresses in the rivets connecting the flange angles to the web near the ends of plate girders may be calculated by the method given in APPENDIX E. The method of determining the permissible load on a rivet is equally applicable to bearing or shear.

3.20.2 *Mild Steel, Wrought Iron and Early Steel Girders*- Bridge spans other than open web girder spans may, if they are kept under regular observation by the Bridge Engineer and his staff, be retained in use, provided that if the impact effect-specified in clause 3 of the Bridge Rules (Revised 1964) for the maximum permissible speed over the bridges is allowed for the calculated stresses for various combinations of loads as laid down in relevant clauses do not exceed the working stresses specified for those combinations by more than 11 percent. Under the same conditions, permissible shear and bearing stresses on rivets may be increased by 25 per cent. This increase in rivet stresses shall not be allowed if the stresses are calculated by the method given in APPENDIX E.

Under the conditions specified above, open web girder spans may be retained in use, provided that the calculated tensile and compressive stresses do not exceed the specified working stresses by more than 5 per cent. The permissible shear and bearing stresses on rivets may be increased by 10 per cent.

3.20.3 Wrought Iron and Early Steel Girders- Where tests are carried out and working stresses determined by the method in APPENDIX D these may be increased by percentages laid down in clause 3.5 for the combination of forces and, under the conditions laid down in clause 3.20.2 by the percentages specified therein.

3.20.4 *Fatigue assessment for Existing Bridges*- The procedure given in appendix 'G' (Re-Revised) shall be followed for carrying out fatigue assessment of members of existing

bridges, using either actual field measurements or numerical models validated with field measurements. Traffic and repair history of the bridge shall be used as accurately as possible. In the absence of accurate data, conservative estimates shall be made.

4 DESIGN AND CONSTRUCTION – GENERAL

4.1 Effective Spans- The effective span shall be as given below:

(a) *For main girders*- The distance between centres of bearing plates or knuckle pins.

(b) *For cross girders*- The distance between the centres of the main girders or trusses.

(c) *For rail or road bearer*- The distance between the centres of the cross girders.

Note:-
Where a cross girder are bearer terminates on an abutment or pier, the centre of bearing thereon shall be taken as one end of the effective span.

(d) *For pins in bending:* The distance between the centre of bearings; but where pins pass through bearing plate having thickness greater than half the diameter of the pins, consideration may be given to the effect of the distribution of bearing pressures on the effective span.

4.2 Effective Length of Struts- For the purpose of calculating l/r (see clause 3.8) the effective length shall be taken as follows:

a) Effectively held in position and restrained in direction at both ends.
l = 0.7L

b) Effectively held in position at both ends and restrained in direction at one end.
l =0.85 L

c) Effectively held in position at both ends, but not restrained in direction.
l = L

d) Effectively held in position and restrained in direction at one end and at the other end partially restrained in direction but not held in position. l =1.5L

e) Effectively held in position and restrained in direction at one end but not held in position or restrained in direction at the other end
l =2.0 L

Where L=length of strut from centre to centre of intersection with supporting members or the cantilever length in case (e)

Note:-
For battened struts the effective length l given above shall be increased by 10 percent (see also clause 6.3)

4.3 Sectional Area

4.3.1 Gross Sectional Area-The gross sectional area shall be the area of the cross section as calculated from the specified sizes.

4.3.2 *Effective Sectional Area*

4.3.2.1 *Tension Members*- The effective sectional area of the member shall be the gross sectional area with the following deductions as appropriate-

(a) Deduction for rivet and bolt holes (see clause 7.2) :

Except as required by the following paragraph, the areas to be deducted shall be the sum of the sectional areas of the maximum number of holes in any cross section at right angles to the direction of stress in the member.

In the case of:

(i) all axially loaded tension members.

(ii) plate girders of steel to IS: 226 or IS:2062 and with d_1/t greater than 85.

(iii) plate girders of steel to IS:961 and with d_1/t greater than 75.

The area to be deducted, when the holes are staggered, shall be greater of:

(i) the sum of the sectional areas of the maximum number of holes in any cross section at right angles to the direction of stress in the member; and

(ii) the sum of the sectional areas of all holes on any zig-zag line extending progressively across the member or apart of the member, less $S^2 t_1/4G$ for each gauge space in the chain of holes, where d_1 and t are as defined in note in Table-II

where,

S= the staggered pitch, i.e., the distance, measured parallel to the direction of stress. In the member, centre-to-centre of holes in consecutive lines.

t_1= the thickness of the holed material and

G= the gauge, i.e., the distance, measured at right angles to the direction of stress in the member, centre-to-centre of holes in consecutive lines.

For sections such as angles, with holes in both legs, the gauge shall be measured along the centre of the thickness of the section.

The net section of the member shall be obtained from that chain which gives the least net area.

In a built-up member where the chains of holes considered in individual parts do not correspond with the critical chain of holes for the member as a whole, the value of any rivet or bolt joining the parts between such chains of holes shall be taken into account in determining the strength of the member.

(b) Deductions for a single angle connected through one leg-

To allow for eccentricity of connection, additional area to be deducted over that specified in (a) above shall be:

$$\frac{a_2^2}{3a_1 + a_2}$$

where,

a_1 = net area of connected leg;

a_2 = area of unconnected leg;

where lug angles are used (see clause 6.13) no additional deduction shall be made and the net area of the whole member shall be taken as effective

(c) Deductions for double angle tension member:

If a double angle tension member is connected with the angles back to back on opposite sides of a gusset plate, no additional deduction shall be made and full net area of the angles shall be considered as effective. Also, if the angles connect separate gusset plates (as in the case of double web truss) and the angles are connected by tie plates located as near the gusset as practicable, or by other effective means, no additional deduction shall be made and full net area of the angles shall be considered as effective. If the angles are not so connected,20% of the net area shall be deducted, in addition that specified in (a) above.

4.3.2.2 Compression members- The gross sectional area shall be taken for all compression members subject to relevant clauses.

4.3.2.3 Parts in shear- The effective sectional area for calculating average shear stress for parts in shear shall be as follows:

(a) *Rolled beams and channels* – The product of the thickness of the web and the overall depth of the section.

(b) *Plate girders* – The product of the thickness of the web and the full depth of the web plate.

Note:-

1. *Where webs are varied in thickness in the depth of the section by the use of tongue plates or the like and in the case of other sections, the maximum shear stress shall be computed from the whole area of the cross-section having regard to the distribution of flexural stresses.*

2. *Webs, which have openings larger than those used for rivets, bolts or other fastening require special consideration and the provisions of this clause are not applicable.*

4.4 Symmetry of Sections- All sections shall, as far as possible be symmetrical about the line of resultant stress, and all rivets shall be grouped symmetrically about the same line. The neutral axis of intersecting main members shall meet in a common point. If eccentric connections are unavoidable, the members shall be proportioned for the combined stress.

4.5 Minimum Sections

4.5.1 No flat, plate, angle or T-bar less than 8mm(5/16in) in thickness shall be used in the main members of the bridge when both sides are accessible for painting, nor less than 10mm (3/8 in) when only one side is accessible, except where it is riveted to another plate or bar. In other than main members of the bridge such as intermediate stiffeners, floor plates, parapets, etc, not designed to carry stresses, a minimum thickness of 6mm (1/4") may be used.

4.5.2 In riveted construction no angle less than 75x50mm (3x2 in) shall be used for the main members of girders.

4.5.3 No angle less than 65x45mm (2 ½ x 2 in) and no flat bar less than 50mm (2in) shall be used in any part of a bridge structure, except for hand railing.

4.5.4 End angles connecting longitudinal bearers to cross girders or cross girders to main girders shall be not less in a thickness than three-quarters of the thickness of the web plates of the stringers and floor beams (cross girders) respectively.

4.6 Spacing and Depth of Girders

4.6.1 The distance between centres of trusses or girders shall be sufficient to prevent overturning by the specified lateral forces. In no case shall it be less than 1/20 th of the span for open web girders nor $1/16^{th}$ of the span for solid web girders.

4.6.2 The depth between gravity axes of the top and bottom chords shall be not greater than three times the width between the centres of main girders. The depth of truss shall preferably be not less than $1/10^{th}$ of the span and that of the plate girders and rolled beams not less than $1/12^{th}$ of the span.

4.6.3 For road bridges and special cases of railway bridges the above limits may be exceeded with the approval of the competent authority.

4.7 Provision for Temperature, Stress and Deflection

4.7.1 Where provision for expansion and contraction, due to change of temperature and stress, is necessary, it shall be provided to the extent of not less than 25mm (1in) for every 30m (100ft) of span.

4.7.2 The expansion bearings shall be so designed as to permit of inspection and lubrication.

4.7.3 The expansion bearings shall allow free movement in a longitudinal direction and at the same time prevent any transverse motion. This provision shall not apply to the spans supported on spherical bearings.

4.7.4 Where the effective span exceeds 30m (100ft) bearings provided at both ends of the main girders shall be such as to permit deflection of the girders without unduly loading the face of the abutment or pier.

4.8 Anchorage – Anchorage shall be provided against longitudinal and lateral movement due to longitudinal and centrifugal loads together with wind or seismic loads, also to the extent of 50 percent in excess of any possible overturning moment of the span as a whole or of the bearings due to the same loads.

4.8.1 The superstructure of the bridge shall be properly secured to the substructures in Zone V, to prevent it from being dislodged off its bearing during earthquake.

4.9 Track Structures - The track structures and its fitting on the bridge shall be such as not to restrain expansion and contraction of the girder and the rail bearers. Guard rails should be provided on all bridges where derailment would likely to cause serious damage to the structures. Where cross sleepers are provided, the guardrails should be fastened to each cross sleeper.

4.10 Clevises and Turnbuckles-Clevises and turnbuckles shall in all cases develop the full strength of the bars of which they form a part and shall be designed to have the same factor of safety.

4.11 Composite Action of Steel and Concrete- Where steel construction is used in conjunction with concrete, and provision is made for adequate interaction between the two materials, they shall be treated as forming a composite member for the purpose of calculation.

4.12 Composite Use of Mild Steel and High Tensile Steel – Mild steel and High tensile steel may be used jointly in a structure or any member of a structure provided that the maximum stress in each element does not exceed the appropriate permissible stress.

4.13 Composite Connections

4.13.1 Connections made with more than one type of fastening transmitting a force direct, the following requirements shall be compiled with:

(a) *Rivets with precision or semi-precision bolts* – The force may be considered as share proportionately between the rivets and the bolts.

(b) *Welds with any other type of connection*- The welds shall be designed to transmit the entire force, except in case of strengthening of existing bridges, when the provisions of IRS Welded Bridge Code shall be followed.

4.14 End Cross Members- When a deck is carried by cross members it is generally preferable to provide end cross members rather than to support the deck on the abutments. When such members are provided, they shall be designed to resist forces from live load taken as not smaller than those for which the intermediate cross members are designed. End cross girders for truss spans preferably shall be designed to permit the use of jack for lifting the superstructure.

4.15 General Provision Against Corrosion – All details shall be designed to reduce to a minimum the incidence of corrosion. All parts should be accessible for inspection, cleaning and painting. Drainage shall be provided at all places where water is likely to collect so as to carry it clear of the surface of the underside of the member and other parts of the structure.

4.16 Camber

4.16.1 In Beam & Plate Girder subjected to Railway loading, spans upto and including 35m (115 ft) need not be cambered. However, in FOB, if required, deflection limitations due to dead load may be taken care of by providing suitable camber.

4.16.2 In unprestressed open web spans, the camber of the main girders and the corresponding variations in length of members shall be such that when the girders are loaded with full dead load plus 75 per cent of the live load without impact producing maximum bending moment, they shall take up the true geometrical shape assumed in their design.

4.16.3 Where girders are prestressed the stress camber change should be based on full dead load and live load including impact.

4.16.4 In Composite Steel girders, necessary camber may be adopted to offset the effect of all dead load and superimposed dead loads.

4.17 Deflection-

(i) For permanent installations other than foot-over-bridges, the ratio of deflection to the length of the girder shall not exceed 1/600.

(ii) In the case of foot-over bridges, the ratio of deflection to length of the girder shall not exceed 1/325.

(iii) In Plate girder & Composite Steel girders in which camber is provided for compensating the deflections due to dead load and superimposed dead load, the ratio of deflection due to live load and Impact Load measured from the soffit of a nominally straight span should not be exceeded by 1 in 800.

Note: With the specific sanctions of the Board, the limit of 1/600 & 1/800 as mentioned above may be exceeded for girders in permanent installations. **(ACS 23 dtd. 08.03.2024)**

5. SOLID WEB GIRDERS

5.1 **Plate Girders and Rolled Beams-** Solid web girders shall be proportioned on the basis of the moment of inertia of the gross cross section with neutral axis taken at the centroid of that section. In computing the maximum stress, the stresses calculated on this basis shall be increased in the ratio of gross to effective area of the flange section. For this purpose, the flange sectional area in riveted or bolted construction shall be taken to be that of the flange plates, flange angles, and the portion of the web and side plates, if any, between the flange angles. In welded construction, the flange sectional area shall be taken to be that of the flange plates and of the tongue plates (i.e., thick vertical plates connecting flange to web) if any, upto a limit of eight times their thickness, which shall not be less than twice that of the web (See clause 5.5)

5.2 Effective Sectional Area

5.2.1 *Compression Flange***-** The effective sectional area of compression flanges shall be the gross area with specified deductions for excessive width or projections of plates (see sub clauses 5.2.1.1 and 5.2.1.2) and the maximum deductions for open holes and holes for black bolts (see clause 7.2) occurring in a section perpendicular to the axis of the member.

5.2.1.1 For calculating the effective cross sectional area of a member in compression (see clause 6.2), the effective width 'b_e of a plate, in terms of its width 'b' measured between adjacent lines of rivets, bolts or welds connecting it to other parts of the section, unless effectively stiffened, shall be taken as:

(i) For riveted, bolted, or stress-relieved welded members in mild steel:

For b/t not above 45, b_e = b

For b/t above 45, b_e= 45t with a maximum value of b/t = 90.

(ii) For riveted or bolted members in high tensile steel:

For b/t not above 40, b_e = b

For b/t above 40, b_e = 40t with a maximum value of b/t=80.

(iii) For 'as-welded' members in mild steel:

For b/t not above 30, b_e = b

For b/t above 30, $b_e = 40\ t.\ \frac{(b/t - 18)}{(b/t - 14)}$ with a maximum value of b/t=80.

In the above, 't' is the thickness of a single plate, or the aggregate thickness of two or more plates, provided these are adequately tacked together (see clause 7.4 and 7.5)

5.2.1.2 The unsupported projection of any plate, measured from its edge to the line of rivets, bolts or weld connecting the plate to other parts of the section shall not exceed:

(a) 16 t for steel to IS: 226 and IS: 2062.

(b) 14t for steel to IS: 961.

Where t is as defined in sub-clause 5.2.1.1 (but see clause 5.5 for compression flanges).

5.2.2 *Tension Flange***-** The effective sectional area of the tension flange shall be the gross sectional area with deductions for all holes as specified for rivet and bolt holes in tension members (in clause 4.3.2.1).

5.2.3 *Webs in Shear***-** The effective sectional area of the web in shear shall be as given in clause 4.3.2.3.

5.3 Slenderness Ratio- The slenderness ratio l/r_y of a girder shall not exceed 300 and it shall not exceed 150 for cantilevers.

Where:

l = the effective length of the compression flange as specified in clause 5.4.

r_y= the radius of gyration of the whole girder about its y-y axis based on the gross moment of inertia and the gross sectional area.

5.4 Effective Length of Compression Flanges

5.4.1 The effective length l of the compression flange for buckling normal to the plane of the girder to be used in clause 3.9 shall be as given below, except that, when the load is applied to the compression flange and both the load and the flange are free to move laterally, the values given shall be increased by 20 per cent.

5.4.2 Simply Supported Girders with no Intermediate Lateral Support to Compression Flange.

5.4.2.1 For simply supported girders where there is no lateral bracing between compression flanges and no cross frames, but with each end restrained against torsion (see below).

(a) With ends of compression flanges unrestrained against lateral bending (i.e. free to rotate in plan at the bearing).

l = span.

(b) With ends of compression flanges partially restrained against lateral bending (e.g., securely cleated connection).

l =0.85 x span.

(c) With ends of compression flanges fully restrained against lateral bending (i.e., not free to rotate in plan at the bearing).

l =0.7 x span.

5.4.2.2 Restraint against torsion at the supports can be provided by web or flange cleats, by bearing stiffeners, by end frames or by lateral support to the compression flange. The restraint element shall be designed to resist in addition to the effects of wind and other applied lateral forces, the effects of a horizontal force F acting normal to the compression flange of the girder at the level of the centroid this flange, where:

$$F=\frac{1.4\times10^{-3}\,l}{\delta\left(\left(C_s / f_{bc}\right)-1.7\right)}$$

where l has the appropriate value given vide clause 5.4.2.1 above and C_s = the critical stress in the flange given by clause 3.9.

f_{bc}= the calculated bending stress in the flange.

δ = the virtual lateral displacement of the compression flange at the end restraint, calculated as explained in clause 5.4.3, except that where the girder rests on a transversely rigid bearing, the end stiffener shall be treated as a cantilever. In no case shall δ be taken as smaller than l^3/40 EI.

5.4.3 Simply Supported Girders with Compression Flanges Laterally supported by U-frames.

5.4.3.1 For simply supported girders where there is no lateral bracing of the compression flanges, but where cross members and stiffeners forming U-frames provide lateral restraint:

$$l=2.5\sqrt[4]{EIa\delta}$$

but not less than a.

Where,

E= Young's modulus.

δ = the virtual lateral displacement of the compression flange at the frame nearest mid-span of the girder, taken as the horizontal deflection of the stiffener at the point of its intersection with the centroid of the compression flange, under the action of unit horizontal force applied at this point to the frame only, except that in the case of very rigid U-frames where δ is less than a^3/40 E I, the horizontal force-F shall be obtained by putting $\delta = a^3/40$ EI & l=a.

This deflection shall be computed assuming that the cross member is free to deflect vertically and that the tangent to the deflection curve at the centre of the span remains parallel to the neutral axis of the unrestrained cross member.

In the case of existing bridges, the value of δ shall be determined experimentally.

a = distance between frames.
I = maximum moment of inertia of compression flange about the y-y axis of the girder.

a) When δ is not greater than a^3/40 E I.

l = a

b) In cases of symmetrical U-frames where cross members and stiffeners are each of constant moment of inertia throughout their own length

$$\delta = \frac{(d')^3}{3EI_1} + \frac{(d'')^2 b}{EI_2}$$

Where

d^{I}= distance of the centroid of the compression flange from the top the cross member.

d^{II}= distance of the centroid of the compression flange from the neutral axis of the cross member.

b = half the distance between centres of the main girders.

I_1 = the moment of inertia of a pair of stiffeners about the centre the web, or of a single stiffener about the face of the web.

I_2 = Moment of inertia of the cross member in its plane of bending.

U-frames shall have rigid connections and shall be designed to resist in addition to the effects of wind and other applied forces, the effect a horizontal force F acting normal to the compression flange of girder at the level of the centroid of this flange and having a value equal to that given by the formula in clause 5.4.2.2., *l* having the value $2.5\sqrt[4]{EIa\delta}$

5.4.4 Girders with Laterally Supported Compression Flanges

5.4.4.1 For all girders where there is effective lateral bracing to the compression flange,

l = the distance between centres of intersection of the bracing with the compression flange.

5.4.4.2 For all girders where the compression flanges are unbraced but supported laterally by members controlled by an effective bracing system or anchorage.

l= the distance between centres of lateral supports.

5.4.4.3 For existing deck type girder bridges, which have no effective lateral bracings between the top flanges but which have transverse sleepers, the effective length of the compression flanges may be taken as equal to the three quarters of the distance between centres of bearings.

5.4.5 *Cantilever Beams without Intermediate Lateral Support:*

for cantilever beams of projecting length L.

a) Built in at the support, free at the end, *l*=0.85 L.

b) Built in at the support, restrained against torsion at the free end by Continuous construction. *l*=0.75 L.

c) Built in at the support, restrained against lateral deflection and torsion at the end, *l*=0.5 L.

d) Continuous at the support, unrestrained against torsion at the support and free at the end. *l*=3.0 L.

e) Continuous at the support with partial restraint against torsion at the support and free at the end. *l*=2.0 L.

f) Continuous at the support, restrained against torsion at the support and free at the end *l*=L.

Where in cases (d), (e) and (f) there is a degree of fixity at the 'free' end the effective length shall be multiplied by 0.75/0.85 and 0.5/0.85 for degrees of fixity corresponding to cases (b) and (c) respectively.
Restraint against torsion at the supports can be provided as in clause 5.4.2.2. above.

5.4.6 *Compression Flange Supporting Continuous Deck* – A compression flange continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (i.e. *l*=0) if the frictional or positive connection of the deck to the flange is capable of resisting a lateral force of $2^1/_2$ per cent of the force in the flange at the point of maximum bending moment, distributed uniformly along its length.

5.5 Flanges

5.5.1 In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not less than 1/3) and the number of flange plates shall be kept to a minimum.

5.5.2 Where flange plates are used, they shall preferably be of equal thickness and at least one plate of the top flange shall extend the full length of the girder, unless the top edge of the web is finished flush with the flange angles.

5.5.3 Compression flange plates unstiffened at their edges shall not project beyond the outer lines of connections to the flange angles by more than 16 t' for steel to IS: 226 and IS: 2062 or 14 t' for steel to IS:961, where t' is the thickness of the thinnest flange plate or the aggregate thickness of two or more plates when the projecting portions of these plates are adequately tacked together.

5.5.4 *In Welded Construction* -Compression flange plates unstiffened at their edges shall not project beyond the line of connections to the web or tongue plates by more than 12 t'.

5.5.5 *In All Cases*-Tension flange plates, stiffened or unstiffened at their edges shall not project beyond the outer line of connections to the flange angles (or, where there are no flange angles, to the web or tongue plates) by more than 20 t'.

5.5.6 *For the Flanges of Girders with Vertical Stiffeners only (see clause at 5.10).* – Where d_1/t is greater than 130 in the case of mild steel to IS : 226 and IS : 2062 or 110 in the case of high tensile steel to IS :961 and when the average shear stress in the web is greater than 0.6 of the permissible stress given for mild steel in clause 3.7, the quantity, I/b^3t shall not be less than $2.5x10^{-4}$ in the case of mild steel and $3x10^{-4}$ in the case of high tensile steel

Where,

I= the moment of inertia of the compression flange about its axis normal to the web, taken as that of the flange angles and plates, and the enclosed portion of web in the case of riveted construction, and the case of welded construction as the flange plate together with a depth of web (adjacent to the flange plate) equal to 16 times the web thickness.

d_1= depth of girder as defined in clause 3.7, TABLE II.
b = spacing of stiffeners.
t = thickness of web.

5.5.7 The unsupported length of the compression flange between brackets riveted to deep cross girders in the case of semi through spans or between the cross frames in the case of open deck spans should not exceed 15 times its width.

5.6 Connection of Flanges to Web – The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the horizontal shear force combined with any vertical loads which are directly applied to the flange. Where a load is directly applied to a flange, it shall be considered as dispersed uniformly through the flange to the connection of the flange to the web at a slope of two horizontal to one vertical.

5.7 Curtailment of Flange Plates- Each flange plate shall be extended beyond its theoretical cut-off point, and the extension shall contain sufficient rivets, bolts or welds to develop the load in the plate calculated from the bending moment and girder section (taken to include the curtailed plate) at the theoretical cut-off point.

5.8 Web Thickness

5.8.1 The ratio of d_1 to t, where d_1 and t are as defined in the note below Table II, shall not be more than 175 for mild steel and 150 for high tensile steel. Where the actual extreme fibre stress in the compression flange is less than the permissible extreme fibre stress, the above figures may be increased in the ratio $\sqrt{K / f_c}$

Where,

f_c= the actual extreme fibre stress in the compression flange, and

K= the permissible extreme fibre stress for bending as given in Table II.

5.8.2 Where the ratio d_1/t exceeds the values stipulated in sub clause 5.8.1, a horizontal stiffener(s) shall be provided (see clause 5.10.3)

5.8.3 Whether horizontal stiffeners are used or not, the minimum thickness of web plate should be greater of 10mm or 1/180 of the smaller clear panel dimension or 1/270 of the greater clear panel dimension.

5.9 **Web Edges-** For girders of rivetted or bolted construction, the edges of the web shall be flush with the flange angles unless a plate is provided over the flange angles to the full length of the girder in which case, the flange angles may project by 3mm.

5.10 **Web Stiffeners:** Web plates shall have stiffeners on both sides over the points of support. Stiffener shall also be provided at all points of local and concentrated load, and when the thickness of the web is less than 1/75 for mild steel and 1/60 for high tensile steel, of the unsupported distance between the flanges, also at points throughout the length of the girder, at a distance apart not greater than 1.5 d_1 and not less than 0.33 d_1, where d_1 is as defined in note below Table II under clause 3.7.

5.10.1 ***Stiffeners over Point of Support and Load Bearing Stiffeners***

5.10.1.1 Stiffeners over the points of support and load bearing stiffeners shall have sufficient area to carry the entire reaction without exceeding the specified intensity of working stress for struts having a length equal to three-quarters of the depth of the girder. The radius of gyration shall be taken about the axis parallel to the web of the beam or girder. The section of the stiffener may be assumed to include a length of the web plate equal to the overall width of the stiffener.

5.10.1.2 The outstanding legs of each pair of stiffeners shall be proportioned to transmit 75 per cent of the entire reaction without exceeding the bearing stress specified in clause 3.7 TABLE II, on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds.

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

5.10.1.4 Stiffeners shall be fitted to provide a tight and uniform bearing upon the flange transmitting the load or reaction unless welds are provided between the flange and stiffener for this purpose. At points of support this requirement shall apply at both flanges.

5.10.1.5 Stiffeners shall not be joggled and shall be solidly packed throughout.

5.10.1.6 Stiffeners shall be so designed (with the addition of knee brackets if necessary) as to provide adequate support for 3/4th of the flanges over their widths.

5.10.1.7 When load bearing stiffeners at supports are the sole means of providing restraint against torsion (see clause 5.4.2.2) the moment of inertia I of the stiffener shall not be less than –

$$\frac{D^3 T R}{250 W}$$

Where
I = moment of inertia of the pair of end load bearing stiffeners, about the centre line of the web plate.
D= overall depth of girder.
T= maximum thickness of compression flange.
R= reaction of the bearing.
W= total load on girder.

In addition, the base of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to the horizontal force F specified in clause 5.4.2.2.

5.10.2 Intermediate Stiffeners

5.10.2.1 Intermediate stiffeners shall be designed so that

$$\text{I is not less than } 1.5\frac{d_1^{\,3} t_1^{\,3}}{S^2}$$

Where I = the moment of inertia of a pair of stiffeners about the centre of the web,
or
a single stiffener about the face of the web; d_1 is as defined in note below Table II under clause 3.7.

t_1 = minimum required thickness of web, and

S = maximum permitted clear distance between vertical stiffeners for thickness t_1.

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

5.10.2.2 Intermediate vertical stiffeners, when not acting as load bearing stiffeners may be joggled and may be in pairs placed one on each side of the web or single, and shall extend to the full depth between the flanges.

5.10.2.3 **EXTERNAL FORCES ON INTERMEDIATE STIFFENERS:** When vertical intermediate stiffeners are subjected to bending moment and shear due to the eccentricity of vertical loads, or the action of transverse forces, moment of inertia, I, of the stiffeners shall be not less than

$I_1 + I_2 + I_3$

I_1, I_2 and I_3 are to be calculated as under:

$$I_1 = 1.5 \times \frac{d_1^3 \; t_1^3}{S^2}$$

$$I_2 = 1.5 \frac{MD^2}{\pm t_2}$$

$$I_3 = 3 \frac{PD^3}{Et_2}$$

Where d_1, t_1 and S are defined in clause 5.10.2.1:

M= the applied bending moment;
P= the lateral forces to be taken by the stiffener and deemed to be applied at the compression flange of the girder;
D= overall depth of girder;
t_2 = actual thickness of web, and
E= young's modulus.

5.10.2.4 *Connection of intermediate stiffeners to web*- Intermediate vertical and horizontal stiffeners not subjected to external loads, shall be connected to the web by welds or rivets, in order to withstand a shearing force, between each component of the stiffener and the web, of not less than 12.6 t^2/h kg per mm run, where t equals web thickness in mm and h equals the projection, in mm of the stiffener component from the web ($8t^2$/h ton per inch run, where t and h are in inches)

5.10.3 HORIZONTAL STIFFENERS – Where horizontal stiffeners are used in addition to vertical stiffeners they shall be as follows:

One horizontal stiffener on one or both sides of the web shall be placed at a distance not more than d_2/5 from the compression flange when the thickness of the web is less than as specified in clause 5.8.1 or less than d_2/200 for mild steel or d_2/180 for high tensile steel, where d_2 is twice the clear distance from compression flange angles, or plate or tongue plate to neutral axis. This stiffener shall have a moment of inertia *l*, not less than 4 $S(t_1)^3$

Where,

I = moment of inertia of a pair of stiffeners about the center of the web or of a single stiffener about the face of web.

t_1= minimum required thickness of the web

S = actual distance between the vertical stiffeners

A second horizontal stiffener on one or both sides of the web shall be placed on the neutral axis of the girder when the thickness of the web is less than $d_2/250$ for mild steel or $d_2/225$ for high tensile steel. This stiffener shall have a moment of inertia. I, not less than $d_2(t_1)^3$.With this second horizontal stiffener, the thickness of web should not be less than $d_2/400$ for mild steel or $d_2/340$ for high tensile steel.

5.10.4. Outstand of All Stiffeners – Unless the outer edge of each stiffener is continuously stiffened the outstand of all stiffeners from the web shall be not more than the following:-

For sections.. 16 t for steel to IS: 226 and IS: 2062.
14 t for steel to IS: 961.

For flat. 12 t for all steels.

Where t is the thickness of the section or flat.

5.11 Flange Splices – Flange Joints Should preferably not be located at points of maximum stress.

Where cover-plates are used, their cross-sectional area shall be not less than the area of the flange element spliced, and be 5 per cent more than theoretical requirement of the spliced element and the calculated working stress in the outer covers shall not exceed the allowable working stress in the flange. There shall be enough rivets or bolts on each side of the splice to develop at least the load in the spliced element plus 5 per cent. For design of welds in flange plates in welded construction, provisions of IRS Welded Bridge Code shall apply.

5.12 Splices in Web – Splices in the web of plate girders and rolled sections used as beams shall be designed to resist the shearing forces and the moments in the web at the spliced section.

In riveted or bolted construction splice plates shall be provided on each side of the web.

5.13 Lateral Bracing

5.13.1 All spans shall be provided with end cross frames and a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings the wind or seismic, racking and centrifugal forces, if any, as specified in the Bridge Rules.

5.13.2 Deck type spans of over 20 m (or 65 ft) effective span shall be provided with end cross frames and a lateral bracing system between the top flanges, of sufficient strength to transmit to the bearings the total lateral load due to wind, or seismic racking and centrifugal forces and with a lateral bracing system between the bottom flanges of sufficient strength to transmit one-quarter of the total lateral loads.

5.13.3 The lateral bracing between compression flanges of all spans shall in addition be designed to resist a transverse shear at any section equal to $2^1/_2$ percent of the total compressive force carried by both the flanges at the section under consideration. Where, however, the transverse sleepers rest directly on compression flanges and offer resistance against buckling of these flanges, this additional transverse shear may be ignored.

Note:-
Existing deck type plate girder with transverse sleepers need not be condemned on account of the absence of lateral bracings, provided they show no sign of distress or undue internal oscillations.

6 OPEN WEB GIRDERS

6.1 Intersection at Joints – For triangulated trusses designed on the assumption of pin jointed connections, members meeting at a joint should, where practicable, have their centroidal axes meeting at a point, and wherever practicable the centre of resistance of a connection shall lie on the line of action of the load so as to avoid an eccentricity moment on the connections.

Where the design is based on non-intersecting members at a joint, all stresses arising from the eccentricity of the members shall be calculated and the stresses kept within the limits specified in the appropriate clauses of this Code.

6.2 General Requirements for Compression Members

6.2.1 The properties of a section shall be computed from the effective sectional area. When plates are provided solely for the purpose of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

6.2.2 The effective sectional area shall be the gross area less the specified deductions for excessive widths of plates (see clause 5.2.1) and the maximum deductions for open holes, including holes for black bolts (see clause 7.2) occurring in a section perpendicular to the axis of the member.

6.2.3 The ratio of the effective length to the least radius of gyration shall not exceed:

120 for main members, and
140 for wind bracing and subsidiary members.

6.2.4 The maximum unsupported width or projection of any plate forming part of a compression member shall be in accordance with sub clauses 5.2.1.1 and 5.2.1.2.

6.2.5 The open sides of built-up compression members of U or I sections shall be connected by lacing or battering where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand.

6.2.6 Lacing and battening plates shall be designed in accordance with clauses 6.5 and 6.6 and shall be proportioned to resist a total transverse shear force Q at any point in the length of the member equal to at least $2^1/_2$ percent of the axial force in the member together with all shear due to external forces, if any, in the plane of lacing. The shear force Q shall be considered as divided equally among all lacing, battening and plating in parallel lines.

6.2.7 Compression members composed of two or more components connected as described in clauses 6.4.6.5 and 6.6 may be designed as homogenous members.

6.3 Effective Length of Compression Members other than Lacings

6.3.1 In riveted, bolted or welded trusses the compression members act in a complex manner and the effective length to be used in clause 3.8 shall be taken as given in Table XI except that, for battened struts, all value given in table shall be increased by 10 percent.

TABLE XI – EFFECTIVE LENGTH OF COMPRESSION MEMBERS

Member		Effective length *l* of member		
		For buckling in the plane of the truss.	For buckling normal to the plane of the truss	
			Compression chord or compression member effectively braced by lateral system.	Compression chord or compression member unbraced.
Chords		0.85 x distance between centres of intersection with the web members	0.85 x distance between centres of intersection with the lateral bracing members or rigidly connected cross girders.	See clause 6.3.4.
Web	Single triangulated system	0.70 x distance between centres of intersection with the main chords.	0.85 x distance between centres of intersections.	Distance between centres of inter-sections.
	Multiple intersection system where adequate connections are provided at all points of intersection.	0.85 x greatest distance between centres of any two adjacent intersections.	0.70 x distance between centres of intersection with the main chords.	0.85 x distance between centres of intersection with the main chord

Note: *The intersections referred to are those of the centroidal axes of the members.*

6.3.2 For single-angle discontinuous struts connected to gussets or to a section either by riveting or bolting by not less than 2 rivets or bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially-loaded member provided that the calculated average stress does not exceed the allowable stresses given in Tables IV or IV (a), in which *l* is the length of the strut, centre-to-centre of fastenings at each end, and r is the minimum radius of gyration.

6.3.3 For single angle discontinuous struts intersected by, and effectively connected, to another angle in cross bracing the effective length in the plane of the bracing shall be taken as in Table XI. In the plane normal to the plane of the bracing, the effective length shall be taken as the distance between the points of intersection and the centroids of the main members. In calculating the ratio of slenderness the radius of gyration about the appropriate

rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration of buckling in the plane of the bracing.

6.3.4 *Effective Length of Unbraced Compression Chords*

6.3.4.1 For simply supported trusses with ends restrained at the bearings against torsion the effective length *l* of the compression chord for buckling normal to the plane of the truss, to be used in the equation given in clause 3.8, shall be taken as follows:

6.3.4.2 *With no lateral support to compression chord* – where there is no lateral bracing between compression chords and no cross frames, *l* = span.

6.3.4.3 *With compression chord supported by U-frames* – Where there is no lateral bracing of the compression chord, but where cross member and verticals forming U-frames provide lateral restraint:

$$l = 2.5 \sqrt[4]{EIa\delta}$$

but not less than a

Where,

δ = the virtual lateral displacement of the compression chord at the frame nearest mid-span of the truss, taken as the horizontal deflection of the vertical member at the point of its intersection with the centroid of the compression chord, under the action of a unit horizontal force applied at this point to the frame only.

This deflection shall be computed assuming that the cross member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unstrained cross member. In the case of existing bridges the value of δ should be determined experimentally.

a = distance between frames.

I = maximum moment of inertia of compression chord about the y-y axis of the truss.

E= Young's modulus.

a) when δ is not greater than $a^3 / 40\ E\ I$, l = a

b) In case of symmetrical U-frames where cross members and verticals are each of constant moment of inertia throughout their own length.

$$\delta = \frac{(d')^3}{3EI_1} + \frac{(d'')^2 b}{EI_2}$$

where,

d' = distance of the centroid of the compression chord from the top of the cross member.

d" = distance of the centroid of the compression chord from the neutral axis of the cross member.

b = half the distance between centre of the main trusses.

I_1 = moment of inertia of the vertical in its plane of bending.

I_2 = moment of inertia of the cross member in its plane of bending.

E = Young's modulus.

U-frames shall have rigid connections and shall be designed to resist, in addition to the effects of wind and other applied forces, the effect of a horizontal force F acting normal to the compression chord of the truss at the level of the centroid of this chord where

$$F = \frac{1.4 \times 10^{-3} l}{\delta(C_0 / f_{ac} - 1.7)}$$

In the above formula

C_o = Euler critical stress in the chord

$$= \frac{\pi^2 E}{(l/r)^2}$$

Where,

l = has the value 2.5 $\sqrt[4]{EIa\delta}$

f_{ac} the calculated working stress in the chord.

δ = the deflection of the chord under the action of unit horizontal force as defined above.

In case of very rigid U-frames where δ is less than a^3 / 40 EI, the horizontal force F shall be obtained by putting δ = a^3 / 40 EI and *l* =a.

6.3.4.4 *With compression chord supporting continuous deck* – A compression chord continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (i.e. *l*=0) if the frictional or positive connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length, of $2^1/_2$ per cent of the maximum force in the chord, in addition to other lateral forces.

6.4 Compression Members Composed of Two Components Back-to-Back

6.4.1 Compression members composed of two angles, channels or tees, back-to-back and separated by a distance not exceeding 50 mm (2 in.), shall be connected together by riveting, bolting or welding, so that the maximum ratio of slenderness *l*/r of each component of the member between such connections is not greater than 50 or 0.5 of the maximum ratio of slenderness of the member as a whole, whichever is the lesser, where *l* is the distance between the centres of connections.

The number of connections shall be such that the member is divided into not less than three approximately equal parts.

6.4.2 Where the members are separated back to back the rivets or bolts in these connections shall pass through solid washers or packings, and where the connected angles, legs or tables of tees are 127 mm (5 in.) wide or over, or where webs of channels are 152 mm (6 in.) wide or over, not less than two rivets or bolts shall be used in each connection, one on the line of each gauge mark.

6.4.3 Where these connections are made by welding, solid packings shall be used to effect the jointing unless the members are sufficiently close together to permit butt welding, and the members shall be connected by welding along both pairs of edges of the main components.

6.4.4 The rivets, bolts or welds in these connections shall be sufficient to carry the shear forces and the moments specified for battened struts, and in no case shall the rivets or bolts be less than 16 mm (5/8 in.)

6.4.5 Compression members connected by such riveting, bolting or welding shall not be subjected to transverse loading in a plane perpendicular to the riveted, bolted or welded surfaces.

6.4.6 Where the components are in contact back to back riveting, bolting or intermittent welding shall be in accordance with clauses as applicable.

6.5 Lacing of Compression Members

6.5.1 As far as practicable the lacing system shall not be varied throughout the length of the compression member.

6.5.2 Lacing bars shall be inclined at an angle of 50^0 to 70^0 to the axis of the member where a single intersection system is used, and at an angle of 40° to 50° where a double intersection system is used.

6.5.3 Except for tie plates as specified in clause 6.5.8 double intersection lacing systems shall not be combined with members of diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for in lacing and its fastenings.

6.5.4 Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

6.5.5 The maximum spacing of lacing bars, whether connected by welding, riveting or bolting, shall be such that the maximum slenderness ratio *l*/r of the components of the compression member between consecutive connections of the lacing bars to one component is not greater than 50 or 0.7 of the maximum ratio of slenderness of the member as a whole, whichever is the lesser, where *l* is the distance between the centres of the connections of the lacing bars to one component.

6.5.6 The required section of lacing bars shall be determined by using the permissible stresses for compression and tension members given in clauses 3.7 and 3.8. The ratio *l*/r of the lacing bars shall not exceed 140. For this purpose, the effective length shall be taken as follows:

6.5.6.1 *In riveted or bolted construction* – The length between the inner end rivets or bolts of the lacing bar in single intersection lacing, and 0.7 of this length for double intersection lacing effectively connected at intersections.

6.5.6.2 In welded construction the distance between the inner ends of effective lengths of welds connecting the bars to the components in single intersection lacings, and 0.7 of this length for double intersection lacing effectively connected at intersections.

6.5.7 Lacing bars shall be connected to the main member either by riveting or bolting by one or more rivets for bolts, in line along the lacings or by welding at each end sufficient to transmit the load to the bars. Any eccentricity of the connection with respect to the centroid

of the lacing bar may be ignored and the lacing designed as an axially-loaded strut provided that the calculated average stress does not exceed the allowable stress given in Table IV or IV(a). Where welded lacing bars overlap the main component, the amount of lap shall be not less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is lesser. Welding shall be provided at least along each side of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar.

Where lacing bars are fitted between the main components they shall be connected to each component by fillet welds on both side of the bar or by full penetration butt welds.

6.5.8 Laced compression members shall be provided with tie plates at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

6.5.9 The length of the end tie plates measured between end fastening along the longitudinal axis of the member shall not be less than the perpendicular distance between the lines of rivets connecting them to the flanges or the vertical side plates of the main chords, whichever is the greater, and shall be at least equal to the depth of the cross girders where these are directly attached to the struts and the length of intermediate tie plates shall be not less than ¾ of the distance between lines of rivets.

6.5.10 The thickness of the tie plates shall be not less than 1/50 of the distance between the innermost lines of rivets, bolts or welds, except when effectively stiffened at the free edges, in which case the minimum thickness may be 8 mm (5/16 in.) ; for this purpose the edge stiffener shall have a slenderness ratio not greater than 170.

6.5.11 Tie plates and their fastenings (calculated in accordance with the method described for battens) shall be capable of carrying the forces for which the lacing system is designed.

6.6 Battening of Compression Members

6.6.1 Battened compression members shall comply with the following requirements.

6.6.1.1 The battens shall be placed opposite each other at each end of the member and at points where the member is stayed in its length, and shall as far as practicable, be spaced and proportioned uniformly throughout. The number of battens shall be such that the member is divided into not less than three bays within its actual length centre to centre of connections.

6.6.1.2 In battened compression members in which the ratio of slenderness about the y-y axis (axis perpendicular to the battens) is not more than 0.8 times the ratio of slenderness about the x-x axis, the spacing of battens centre to centre of end fastenings shall be such that the ratio of slenderness *l*/r of the lesser main component over this distance shall be not grater 50 to 0.7 times the ratio of slenderness of the member as a whole, about its x-x axis (axis paralleled to the battens).

In battened compression members in which the ratio of slenderness about the y-y axis (axis perpendicular to the battens) is more than 0.8 times the ratio of slenderness about the x-x axis, the spacing of battens centre to centre of end fastenings shall be such that the ratio of slenderness *l*/r of the lesser main component over that distance shall not be greater than 50 or 0.6 times the ratio of slenderness of the member as a whole about its weaker axis.

6.6.1.3 Battens shall be plates, channels or *l* sections and shall be riveted, bolted or welded to the main components. Battens and their connections shall be so designed that they resist simultaneously a longitudinal shear force equal to QD/na and a moment equal to QD/2n where:

D= the longitudinal distance centre-to-centre of battens.

a= the minimum transverse distance between the centroids of rivet or bolt groups , or welding.

Q= the transverse shear force as defined in clause 6.2.6.

n = the number of parallel planes of battens

6.6.1.4 The length of end batten plates parallel to the axis of the member shall be not less than the perpendicular distance between the lines of rivet connecting them to the flanges or the vertical side plates of the main chord whichever is the greater, and shall be at least equal to the depth of the cross girders where these are directly attached to the struts; and the length of intermediate battens shall be not less than ¾ of the distance between lines of rivets but in no case shall the length of any batten plate be less than twice the width of the smaller component in the battens.

6.6.1.5 The thickness of batten plates shall be not less than 1/50 of the minimum distance between the innermost lines of connecting rivets, bolts or welds, except when effectively stiffened at the free edges, in which case the minimum thickness may be 8 mm (5/16 in.); for this purpose the edge stiffener shall have a slenderness ratio not greater than 170.

6.6.1.6 The length of weld connecting each longitudinal edge of the batten plate to a component shall in the aggregate be not less than half the length of the batten plate, and at least one-third of the weld shall be placed at each end of the longitudinal edge. In addition, the welding shall be returned along the ends of the plate for a length equal to at least four times the thickness of the plate.

Where tie or batten plates are fitted between main component they shall be connected to each component either by fillet welds on each side of the plate, at least equal in length to that specified in the preceding paragraph, or by complete penetration butt welds along the whole length of the plate.

6.6.1.7 Battened compression members not complying with these requirements, or those subjected to bending moments in the plane of the battens shall be designed according to the exact theory of elastic stability, or empirically with verification by tests, so that they have a load factor of not less than 1.7 in the actual structure.

6.6.1.8 Battened compression members composed of two angles forming a cruciform cross section shall conform to the above requirements except as follows:

(a) The battens shall be in pairs placed in contact one against the other unless they are welded to form cruciform battens.
(b) A transverse shear force of $Q/\sqrt{2}$ shall be taken as occurring separately about each rectangular axis of the whole member.
(c) A longitudinal shear force of $QD/a\sqrt{2}$ and the moment $QD/2\sqrt{2}$ shall be taken in respect of each batten in each of the two planes, except where the maximum

value of //r can occur about a rectangular axis, in which case each batten shall be designed to resist a shear force of $2^1/_2$ percent of the total axial force.

Q, D and a are as defined in clause 6.6.1.3.

6.7 General Requirements for Tension Members:

6.7.1 Tension members should preferably be of rigid cross section, and when composed of two or more components these shall be connected as described in clauses 6.8, 6.9 and 6.10.

6.7.2 The properties of the cross section shall be computed from the effective sectional area as given in clause 4.3.2.1.

When plates are provided solely for the purposes of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

6.7.3 For main members the ratio of unsupported length to the least radious of gyration shall not exceed:

(a) 250 for railway bridges,

(b) 300 for road and foot bridges.

6.7.4 The open sides of built-up tension members of U or I section shall be connected by lacing and battening , when the length of the outstand, towards the open side, exceeds 16 times the mean thickness of the outstand.

6.7.5 Lacing and battening shall be designed in accordance with clauses 6.9 and 6.10 and shall be proportioned to resist all shear forces due to external forces, if any, in the plane of lacing. The shear shall be considered as divided equally among all transverse systems and plating in parallel planes.

6.7.6 The net sectional area of a bolt or screwed tension rod shall be taken as the area at the root of the thread.

6.8 Tension Members Composed of two Components Back-to-Back

6.8.1 Tension members composed of two angles, tees or channels back-to-back, either in contact or separated by a distance not exceeding 50 mm (2 in.), shall be connected together in their length at regular intervals through solid washes or packing by riveting, bolting or welding so spaced that the maximum ratio of slenderness of each element is not greater than that specified for main members in clause 6.7.3.

6.8.2 Where the components are in contact back-to-back, riveting, bolting or intermittent welding shall be in accordance with clauses applicable.

6.9 Lacing of Tension Members

6.9.1 As far as practicable the lacing system shall not be varied throughout the length of the tension members.

6.9.2 Lacing bars shall be inclined at an angle of 50^o to 70^o to the axis of the member when a single intersection system is used and at an angle of 40^o to 50^o when a double intersection system is used.

6.9.3 Except for tie plates as specified in clause 6.9.7 below, double intersection lacing systems shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the member, unless all forces resulting from deformation of the member are calculated and provided for in the lacing and its fastenings.

6.9.4 Lacing bars shall be so connected that there is no appreciable interruption of the triangulation of the system.

6.9.5 The required section of lacing bars shall be determined by using the permissible stresses for compression and tension members given in clauses 3.7 and 3.8. the ratio *l*/r of the lacing shall not exceed 170. For this purpose the effective length shall be taken as follows:

6.9.5.1 In riveted or bolted construction, the length between the inner end rivets or bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersection.

6.9.5.2 In welded construction, the distance between the inner ends of effective lengths of welds connecting the bars to the components for single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersections.

6.9.6 The riveting, bolting or welding of lacing bars to the main members shall be sufficient to transmit the load to the bars. Where welded lacing bars overlap the main components, the amount of lap shall be not less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is the lesser. The welding shall be provided at least along each side the bar for the full length of the lap and returned along the ends of the plate for a length equal to at least 4 times the thickness of the plate.

6.9.7 Where lacing bars are fitted between main components, they shall be connected to each component by fillet welds on both sides of the bar or by full penetration butt welds.

6.9.8 Laced tension members shall be provided with tie plates at the ends of the lacing systems, at points where the lacing systems are interrupted and where the member is connected to another member.

6.9.9 The length of end batten plates parallel to the axis of the member shall be not less than the perpendicular distance between the lines of rivets connecting them to the flanges or the vertical side plates of the main chords whichever is the greater, and shall be at least equal to the depth of the cross girders where these are directly attached to the struts; and the length of intermediate tie plates shall be not less than ¾ of the distance, between the lines of rivets.

6.9.10 The thickness of all tie plates shall be not less than 1/60 of the distance between the innermost lines of rivets, bolts or welds attaching them to the main components, except when effectively stiffened at the edges, in which case the minimum thickness may be 8 mm (5/16 in.); for this purpose the edge stiffeners shall have a slenderness ratio not greater than 170.

6.10 Battening of Tension Members

6.10.1 Battened tension members shall comply with the following requirements:

6.10.1.1 The spacing of battens, measured as the distance between the middles of adjacent pitches of rivets or bolts or, for welded construction the clear distance between the battens, shall be such that the maximum ratio of slenderness of each element is not greater than that specified for main members in clause 6.7.3.

6.10.1.2 The effective length of the batten, parallel to the axis of the member, shall be taken as the longitudinal distance between end fastenings.

End battens shall have an effective length of not less than the perpendicular distance between the lines of rivets connecting them to the flanges or the vertical side plates of the main chords, whichever is the greater, and shall be at least equal to the depth of the cross girders where these are directly attached to the struts; and the length of intermediate battens shall have an effective length of not less than one-half of the distance between the lines of rivets.

6.10.1.3 Batten plates shall have a thickness of not less than 1/60 of the minimum distance between the connecting rivet or bolt groups or welds, except where they are stiffened at their edges.

6.10.1.4 Where battens are attached by rivets or bolts, not less than two rivets or bolts shall be used in each connection. Where battens are attached by welds the length of welds connecting each longitudinal edge of the batten plate to the component shall, in the aggregate, be not less than half the length of the batten plate, and at least 1/3 of the weld shall be placed at each end of the longitudinal edge. In addition, welding shall be returned along the base of the plate for a length at least equal to four times the thickness of the plate (But see clause 3.6.5)

Where the tie or batten plates are fitted between main components they shall be connected to each member either by fillet welds on each side of the plate equal in length to at least that specified in the preceding paragraphs or by full penetration butt welds.

6.11 Splices

6.11.1 *Compression Members*

6.11.1.1 Splices in compression members, located at or near effectively braced panel points, shall be designed to transmit the full load in the member. All other splices in compression members shall have a sectional area 5 per cent more than that required to develop the load in the member at the average working stress in the member. All cover material shall, as far as practical, be so disposed with respect to the cross section of the member as to communicate the proportional load in the respective parts of the section.

6.11.1.2 Both surfaces of the parts spliced shall be covered, or other means taken to maintain the alignment under load of the abutting ends.

6.11.1.3 Where flexural tension may occur in the member, the cover material shall be designed to resist such tension.

6.11.1.4 Rivets, bolts or welds shall develop the full load in the cover material as defined above, calculated on the cover area.

6.11.2 Tension Members

6.11.2.1 Splices in tension members shall have a sectional area 5 percent more than that required to develop the load in the member and, whenever practicable, the cover material shall be disposed to suit the distribution of stress in the various parts of the cross section of the member. Both surfaces of the parts spliced shall be covered wherever possible.

6.11.2.2 Rivets, bolts or welds shall develop the full load in the cover material as defined above, calculated on the cover area.

6.12 Connection at Intersections

6.12.1 Connections of members at an intersection shall develop at least the design loads and moments communicated by the members. Due regard to the nature and distribution of stress over the cross section of the members shall be given in determining the distribution of the fastenings. All members shall, where possible, be so connected that the load is appropriately distributed over the cross section; otherwise, consideration shall be given to the distribution of stress through the material to those parts of the section not directly connected, and for this purpose the angle of distribution may be taken as 45°.

6.12.2 Gusset shall be capable of sustaining the design loads and moments communicated by the members without exceeding the allowable working stresses.

6.12.3 Rivet, bolt and welding groups shall be as compact as practicable.

6.13 Lug Angles

6.13.1 Lug angles connecting a channel or similar member shall, as far as possible, be disposed symmetrically with respect to the section of the member.

6.13.2 In the case of angle members the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 20 per cent in excess of the force in the outstanding leg of the angle, and the attachment of the lug angles to the angle member shall be capable of developing a strength 40 per cent in excess of that force.

6.13.3 In the case of channel or similar members, the lug angles, and their connection to the gusset or other supporting member, shall be capable of developing a strength not less than 10 per cent in excess of the force not accounted for by the direct connection of the member, and the attachment of the lug angles to the member shall be capable of developing a strength of 20 per cent in excess of that force.

6.13.4 In no case shall less than two bolts or rivets be used for attaching the lug angle to the gusset or other supporting member.

6.13.5 The effective connection of the lug angle shall, as far as possible, terminate at the end of the member connected, and the fastening of the lug angle to the member shall preferably start in advance of the direct connection of the member to the gusset, etc.

6.14 Section at Pin Holes in Tension Members – In pin-connected tension members the longitudinal net section beyond the pin hole parallel with the axis of the member shall be not less than the required net section of the member. The net section through the pin hole transverse to the axis of the member shall be at least 33 per cent greater than the required net section of the member. In the case of members without stiffened edges the ratio of the net width of the members (through the pin hole transverse to the axis of the member) to its thickness shall be not more than 16. Where the thickness of main material is not sufficient to

resist the load from the pin in bearing; or where the net section through the pin hole requires reinforcement, pin plates (see clause 6.15) shall be provided and the total thickness shall comply with the above requirements.

6.15 Pin Plates – Pin plates shall be of sufficient thickness to make up the required bearing or cross-sectional area and shall be so arranged as to reduce the eccentricity to a minimum. Their length measured from the centre of the pin to the end (on the reaction side) shall be at least equal to their width and at least one plate on each side shall be as wide as the dimensions of the member will allow. Pin plates shall be connected with enough rivets, bolts or welds to transmit the bearing pressure on them and shall be so arranged as to distribute it uniformly over the full section of the member.

6.16 Diaphragms in Members – In addition to diaphragms required for the proper functioning of the structure, diaphragms shall be provided as necessary for fabrication, transport and erection.

6.17 Lateral Bracing

6.17.1 All spans shall be provided with a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings the wind or seismic, racking and centrifugal forces, if any, as specified in the Bridge Rules.

6.17.2 The bracing shall be so designed as to transmit to the main girders the longitudinal loads due to tractive effort and/or braking effect in order to relieve the cross-girders of horizontal bending stresses. The lateral bracing between compression chords shall be designed to resist a transverse shear at any section equal to 2½ per cent of the total compressive force carried by both the chords at the section under consideration. This force should be considered in addition to the wind, centrifugal and racking forces.

6.17.3 Where the depth permits, lateral diagonal bracing shall be fixed between the top chords of main girders of through span of sufficient rigidity to maintain the chords in line and of sufficient strength to transmit the wind or seismic forces to the portal bracing between the end posts.

6.18 Sway Bracings – The overhead cross bracings, or sway bracings between vertical web members shall be proportioned to transmit to the lower chords, through the web members, at least 50 per cent of the top panel wind or seismic load, and the vertical web members shall be designed to resist the resulting bending moments.

6.19 Portal Bracing – Portal bracing with knee braces, as deep as clearances will allow, shall be fitted to the end posts or rakers. The portal system at each end, i.e., the end posts or rakers together with the portal bracing, shall be designed to transmit to the bearings one half of the sum of the lateral forces on the top chords. The sway bracing so provided shall not be taken as affording any relief to the lateral bracing system or portal system. In addition, the portal system shall be designed to resist a lateral shear equal to 1¼ per cent of the total compressive force in the two end posts or in the two top chords in the end panel whichever is greater.

7. RIVETING, BOLTING AND WELDING

7.1 Effective area in Shear and Bearing of Rivets, Bolts and Pins (ACS 22 dtd 25.09.2019)

7.1.1 In calculating the number of rivets, bolts or pins required, the effective area shall be taken as :

For rivets: the area of hole.

For pins: the gross cross section area of pin.

For bolts: the gross cross section area of the bolt (where thread cannot occur in the shear plane) and the net tensile stress area (where thread can occur in the shear plane-). For bolts where the net tensile stress area is not defined, the net tensile stress area shall be taken as the area at the root of threads as given below:

7.1.2. The effective bearing area of a pin, rivet or bolt shall be the diameter multiplied by the thickness of the part transmitting or receiving the load, except that for rivets or bolts with countersunk heads half the depth of the countersunk shall be ignored in arriving the length in bearing.

7.2 Deductions for Holes for Rivets, Bolts and Pins - In calculating the area to be deducted for rivets, bolts or pins, the diameter of the hole and for the countersunk bolts or rivets, the diameter of the hole plus 3 mm (1/8 in.) shall be used.

Nominal bolt diameter (mm)	12	14	16	18	20	22	24	27	30	33
Net tensile stress area (mm^2)	84	115	157	192	245	303	353	459	561	694

Rivets, bolts or pins in double shear shall be considered as having twice the shear area of those in single shear

(ACS 22 dtd 25.09.2019)

7.3 Minimum Pitch of Rivets and Bolts The distance between centres of rivets or bolts shall be not less than 2½ times the nominal diameter of the rivet or bolt.

7.4 Maximum Pitch of Rivets and Bolts

7.4.1 The distance between centres of any two adjacent rivets or bolts connecting together elements in contact of compression or tension members shall not exceed 32 t or 300 mm (12 in.), whichever is the lesser, where t is the thickness of the thinner outside element.

7.4.2 The distance between centres of two adjacent rivets or bolts in a line lying in the direction of stress shall not exceed 16 t or 200 mm (8 in.), in tension members, or 12 t or 200 mm (8 in.) whichever is the lesser, in compression members. In the case of compression members in which forces are transferred through butting faces this distance shall not exceed 4½ times the diameter of the rivets or bolts for a distance from the abutting faces equal to 1½ times the width of the member.

7.4.3 The distance between centres of any two consecutive rivets or bolts in a line adjacent to and parallel to an edge of an outside plate shall not exceed 100 mm (4 in.) plus 4 t or 200 mm (8 in.), whichever is the lesser in compression or tension member.

7.4.4 When rivets or bolts are staggered at equal intervals and the gauge does not exceed 75 mm (3 in.) the distances between centres of rivets or bolts, as specified in clauses 7.4.2 and 7.4.3 above, may be increased by 50 per cent.

7.5 Edge Distance

7.5.1 The minimum distance from the centre of any rivet hole or bolt hole to a sheared edge or hand flame cut shall be 1¾ times the diameter of the hole and to a rolled, machine flame cut, sawn or planed edge, 1½ times the diameter.

7.5.2 Where two or more parts are connected together a line of rivets or bolts shall be provided at a distance of not more than 4 t + 40 mm (1½ in.) from the nearest edge, where t is the thickness of the thinner outside plate.

7.6 Hand Driven Rivets – The use of hand driven rivets shall be avoided as far as possible. If unavoidable, the number of rivets to be provided shall be determined in accordance with clause 3.7 (Table II).

7.7 Rivets or Bolts Through Packing – The number of rivets or bolts transmitting shear through packing shall be increased above the number required by normal calculations by 1¼% for each millimetre (2% for each 1/16 in.) thickness of packing except that for packing having a thickness of 6 mm (¼") or less, no increase shall be made. For double shear connections packed on both sides, the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing.

The additional rivets or bolts should be placed in an extension of the packing.

NOTE :-
If the packing is made of more than one plate, placed side by side, the thickness of packing shall be the over-all thickness of all such plates placed together.

7.8 Long Grip Rivets – The grip of rivets carrying calculated loads shall not exceed 8 times the diameter of the hole. Where the grip exceeds 6 times the diameter of the hole, the number of rivets required by normal calculation shall be increased by not less than half a percent for each additional millimetre of grip (¾% for each additional 1/16" of grip).

7.9 Rivets in Tension – The use of rivets in tension should be avoided wherever possible, but if their use is unavoidable, the stress shall not exceed that specified in clause 3.7.

7.10 Bolts – Turned and fitted, precision, semi precision and black bolts, when provided, shall conform to IRS B-I. Where there is risk of nuts becoming loose they shall be secured by check nuts or any other suitable means.

7.11 General Requirements for Welds – The design of welds in mild steel shall be in accordance with the requirements of the IRS Welded Bridge Code.

7.12 High Strength Friction Grip (HSFG) Bolts - High Strength Friction Grip (HSFG) bolts shall be provided as per para 28.9 of IRS Specification For Fabrication And Erection Of Steel Girder Bridges And Locomotive Turn Tables (Fabrication Specification) Serial No. B1-2001. These bolts shall be designed as per provisions of IS:4000-1992 (High Strength Bolts in Steel Structures - Code of Practice) with the following over-riding clauses:

7.12.1 Provisions not covered in IS:4000-1992 shall conform to IRS Codes. For edge distance, gauge distance and pitch etc., provisions of para 7 shall be followed.

7.12.2 Joints with HSFG bolts shall be designed only as friction type joints. Bearing type joints with HSFG bolts shall not be provided.

7.12.3 Short/long slotted holes shall not be allowed.

7.12.4 The diameter of hole shall be 1.5 mm more than the nominal diameter of the

HSFG bolt for bolt dia less than 25mm and 2mm more than the nominal diameter of the HSFG bolt for larger diameters.

7.12.5 In certain cases, HSFG bolts might be required to be provided in oversize holes. The maximum size of oversize holes shall not exceed 1.25 d or d + 4 mm, whichever is less where d is nominal diameter of HSFG bolt.

7.12.6 Design of Friction type joint shall be done as follows:

7.12.6.1 For joints subjected to shear force only- shear force applied should not exceed.

$$\frac{\text{Slip factor x Number of Effective Interfaces x Minimum Bolt Tension}}{\text{Factor of Safety}}$$

Minimum Bolt Tension shall be as given in table XII below:

Nominal size of Bolt	Minimum Bolt tension in KN	
	Property class 8.8	Property class 10.9
M16	94.5	130
M20	147	203
M22	182	251
M24	212	293
M30	337	466
M36	490	678

Table XII (Based on Table 3 of IS:4000-1992)

7.12.6.2 For joints subjected to shear and tensile force.

$$\frac{\text{Caculated Shear}}{\text{Slip Factor x No. of Effective Interface}} \leq \frac{(\text{Proof Load - Calculated Tension x F})}{\text{Factor of Safety}}$$

where factor F shall be taken as 2.0 if external tension is repetitive and 1.7 if non repetitive. Proof Load shall be worked out for the yield stress of the bolt used and the stress area of thread as given in Table 2 of IS:4000-1992.

7.12.6.3 In 7.12.6.1 and 7.12.6.2, the factor of safety shall be taken as follows:

(a) 1.4 for normal loads

(b) 1.2 when wind load is considered provided that (i) connections are adequate when wind forces are not considered (ii) wind load is not the primary loading for the purpose of design.

7.12.6.4 Slip factor for design shall be taken as per table XIII

S.No.	**Surface preparation of the interface between plies in a HSFG bolted joint**	**Slip factor**
1.	Surface blast cleaned and spray metallized with aluminium (thickness > 100µm), with no over coating	0.40
2.	Surfaces cleaned by wire brushing or flame cleaning, with loose rust and paint layers removed (only isolated patches of coatings/ rust can remain)	0.25
3.	Any other surface preparation	To be established as per procedure given in Annexure B of IS:4000-1992

Table XIII

7.12.6.5 **Limitation of Shear Transmitted to Plies:**

The bearing force transmitted between any bolt and any ply shall not exceed 1.2 *f_y *d*t where f_y is yield stress of the ply, d is nominal dia of HSFG bolt and t is the thickness of ply. In addition, the component of force acting on the edge of a bolt in the direction of the minimum distance toward the edge of a ply shall not exceed $e*f_y*t/1.4$ (where e is edge distance of bolt plus half the bolt diameter, in mm).

7.12.6.6 **Design of joints subject to pure tension:**

The tension in the bolts shall be limited to the values given in table 2 of IS:4000-1992. In case the bolts are subject to tension in fatigue conditions, the minimum tensile force in the bolt shall not exceed 50% of the minimum bolt tension values specified in table 3 of IS:4000-1992.

7.12.7 **Tension to be given in Bolts:**

The HSFG bolts shall be tightened by procedure given in IRS Specification For Fabrication And Erection Of Steel Girder Bridges And Locomotive Turn Tables (Fabrication Specification) Serial No. B1-2001 to impart the minimum tension specified in Table XII.

7.12.8 Detailing: The detailing of joints shall be done carefully ensuring that for all the bolts, access for fixing of torque wrench head (approximately 100 mm) is available on one side while the other side can be held with ordinary spanner.

APPENDIX 'A'

Rules for Pre-stressing Open Web Girder Spans

A-1 PREPARATION OF CAMBER DIAGRAM

A-1.1 Contract drawings are dimensioned for the main girder without camber and in order to ensure that its fabrication and erection shall be such as to eliminate deformation stresses in the loaded span, a camber diagram shall be prepared on which shall be clearly indicated the amounts by which the nominal lengths (i.e., the lengths which will give no camber) of members shall be increased or decreased in order that the outline of the girder under full load (dead load, live load and impact) in the case of pre-stressed girders or under full dead load plus 75 per cent of the live load without impact in the case of un-pre-stressed girders, shall be the nominal outline, enlarged (1 + K) times in the case of a through span and reduced (1 – K) times in the case of a deck span (see paragraph 4 below for definition of K).

A-1.2 The stress camber shall be calculated on gross area of the member and be equal to the change of the length of the member due to loading given in para A-1.1, but of opposite sign.

A-1.3 For the purpose of calculating the change in length of members under stress, the modulus of elasticity for both high tensile and mild steels shall be taken as 21,100-kg/sq mm (13,400 ton / sq. in.). The effective length shall be taken between the theoretical intersection points of adjacent members.

A-1.4 To ensure that the length of the floor system of a span shall be constructed to its nominal dimensions, i.e., to avoid changes in lengths of floor and loaded chord lateral system a further change in length shall be made in the lengths of all members equal to

$$\frac{\text{Loaded chord extension or contraction}}{\text{Loaded chord length}} \text{ x } \quad \text{Length of member} = (\text{KxL})$$

In through spans this change will be an increase in the lengths of all members while in the case of deck spans it will be a decrease in the lengths of all members.

A-1.5 The nominal girder lengths altered in accordance with paragraphs 1.2 and 1.4 give a girder correctly stress cambered but with the loaded chord length identical with that shown on the contract drawings, thus requiring no modifications to floor and loaded chord lateral systems.

A-1.6 The nominal lengths and camber lengths shall be rounded off to the nearest half a millimeters (1/64 in.).

A-1.7 The difference between nominal lengths and camber lengths thus modified is the practical camber change.

A-1.8 A williot diagram shall be drawn for the practical camber changes to obtain ordinates of the erection packing necessary to produce the required camber.

A-1.9 Adjustments of the lengths shall be made to top lateral bracing members to suit camber lengths of the top chords in the case of through girder spans and to the bottom lateral bracing members in the case of deck spans. The average value of the pre-stressed length of top or bottom lateral members as the case may be, adopted throughout.

A-2 FABRICATON

A-2.1 The actual manufactured lengths of the members are to be the lengths “with camber” given on the cambered diagram.

A-2.2 The positions and angular setting out lines of all connection holes in the main gussets and also the positions of the connection holes in the chord joints and the machining of the ends shall be exactly as shown on the contract drawings. This will permit the butts in the chord segments to be exactly as shown on the contract drawings.

A-2.3 The groups of connection holes at the ends of all the members are to be as shown on the contract drawings, i.e., without any allowance for camber but the distance between the groups at the ends of each member shall be altered by the amount of the camber allowance in the member.

A-3 ERECTION

A-3.1 The joints of the chords shall be drifted, bolted and preferably riveted to their geometric outline.

A-3.2 All other members are to be elastically strained into position by external forces, so that as many holes as possible are fair when filled with rivets.

A-3.3 Drifting of joints shall be avoided as far as possible, and when necessary, should be done with great care and under close expert supervision. Hammers not exceeding one kg. (2 lb.) in weight should be used with turned barrel, drifts and a number of holes drifted simultaneously, the effect of the drifting being checked by observation of adjacent unfilled holes.

A-3.4 The first procedure during erection consists of placing camber jacks in position on which to support the structure. The camber jacks should be set with their tops level and with sufficient run out to allow for lowering of panel points except the centre by the necessary amounts to produce the required camber in the main girders. It is essential that the camber is accurately maintained throughout the process of erection and it should be constantly checked. The jacks shall be spaced so that they will support the ends of the main girders and the panel points. The bottom chord members shall then be placed on the camber jacks, carefully levelled and checked for straightness and the joints made and riveted up.

A-3.5 The vertical and diagonal web members, except the end posts, shall then be erected in their proper positions on the bottom chords. It is recommended that temporary top gussets, the positions of the holes in which are corrected for the camber change of length in the members, should be used to connect the top ends of the members; this will ensure that the angles between the members at the bottom joints are as given by the nominal outline of the girders. The verticals and diagonals shall then be riveted to the lower chords.

A-3.6 All panel points, except the centre, shall now be lowered by amounts to produce the correct camber in the main girders as shown on the camber diagram.

A-3.7 The top chord should be erected piece-by-piece working symmetrically from the centre outwards, and the joint made by straining the members meeting at the joint and bringing the holes into correct registration.

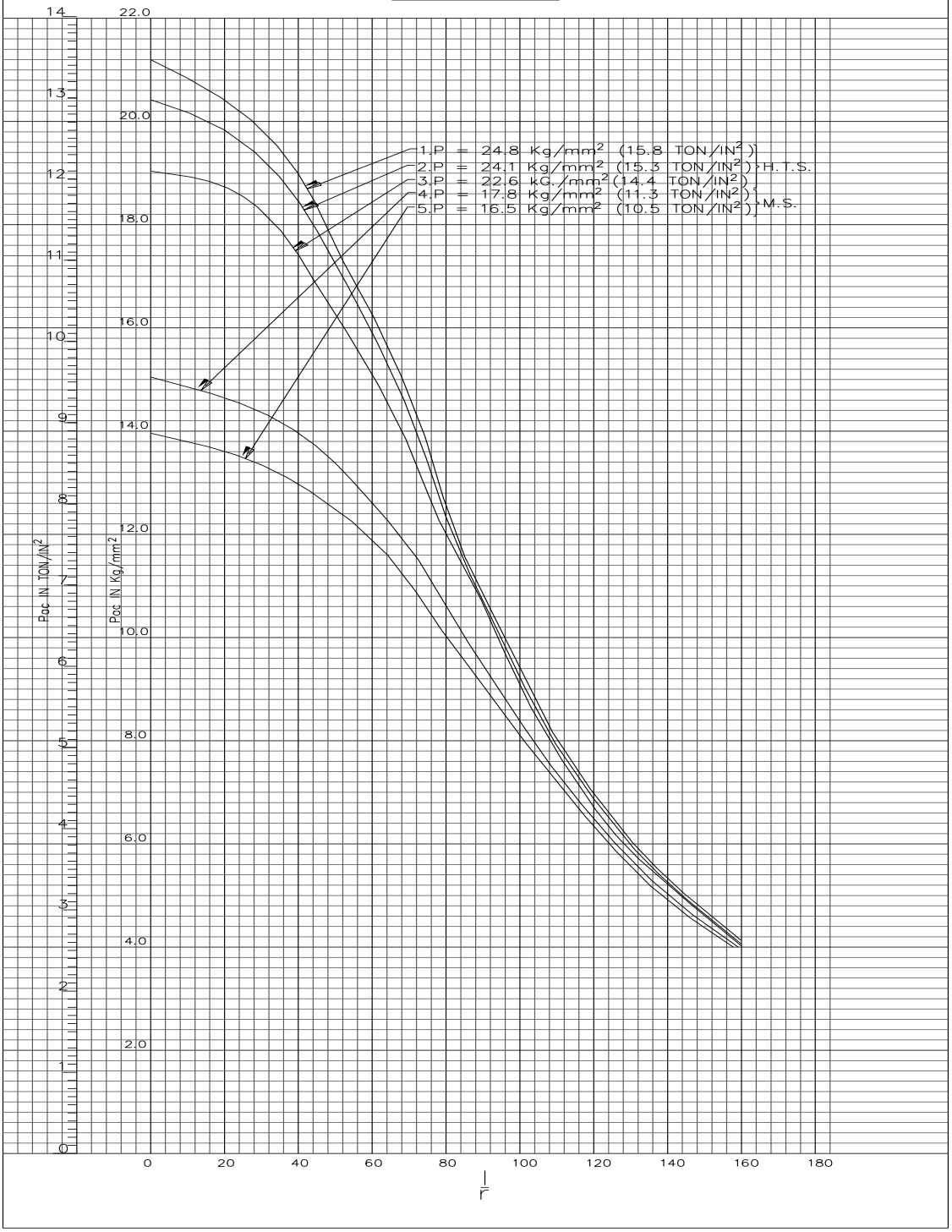
A-3.8 The temporary gussets, if used, shall be replaced by the permanent gussets in the same sequence as the erection of the top boom members.

A-3.9 The end posts shall be erected last. The upper end connection should preferably be made first and if there is no splice in the end raker, the final closure made at the bottom end connection. If there is a splice, the final closure should be made at the splice.

A-3.10 When cantilevered method of erection is used, the above procedure does not apply.

APPENDIX–B

CURVES SHOWING ALLOWABLE WORKING Pac ON EFFECTIVE CROSS SECTION FOR AXIAL COMPRESSION



APPENDIX 'C'
Critical Compression Stress C_s for Sections Symmetrical About the XX-Axis Formula

C-1. The critical compression stress C_s for sections symmetrical about the x-x axis may be calculated from:

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

Where,

Zx = gross section modulus about x –x axis.

l = effective length of compression flange.

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

I_x = moment of inertia of the whole section about x-x axis.

I_y = moment of inertia of the whole section about y-y axis.

E = Young's modulus taken as 21100 kg/mm^2 (13400 tons/in.^{2})

G = Modulus of rigidity taken as 0.4E.

K = Appropriate torsion constant.

W = warping constant = $\frac{EI_f h^2}{2}$ for I sections.

h = distance between flange centroids.

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

C-1.1 For I sections this formula reduces to

$$C_s = \frac{42200}{Z_x l}\sqrt{\left\{\frac{I_y K}{\mu}\left(1+12.3\frac{I_f h^2}{Kl^2}\right)\right\}} \text{ kg / mm}^2$$

to obtain C_s in tons/in^2, replace the constant 42200 in the above formula by 26800 for sections composed of approximately rectangular elements:

$K = \sum\left(\frac{bt^3}{3}\right)$ approximately,

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

C-1.2 For channel and Z sections, the formula in C-1.1 above gives conservative values

C-1.3 For box members, conservative values of C_s are obtained by substituting in formula in C-1.1 above.

K $= \dfrac{4A^2}{\sum(S/t)}$ approximately

A = total enclosed area of section.

S = length of each element of the periphery.

t = thickness of each element (in the case of curtailed flanges, the effective thickness).

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

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

C-1.4 For a plate or flat in bending in a plane parallel to its surface (substituting appropriate values of K, etc.).

$$C_s = 42200\frac{t^2}{lD} kg/mm^2$$

Or

$$C_s = 26800\frac{t^2}{lD} ton/in^2$$

Where,

t = thickness

D= depth,

l = effective length of part in compression.

APPENDIX 'D'

Method of Computing Permissible Stresses in Existing Wrought Iron or Early Steel Girder

D-1.1 In the case of early steel girders over which it is desired to run loads causing stresses in excess of those laid down in Clause 3.19 or where there is any doubt as to the quality of the material, the following tests should be made.

D-1.2 Test-pieces shall be cut from representative members carrying direct stress and the permissible stress shall be based on the results of tensile, tests made on these.

D-1.3 For open web spans 20 pieces of metal shall be cut from at least 4 different sections of one or more spans.

D-1.4 For plate web spans 8 pieces shall be cut from at least 4 different sections of one or more spans.

D-1.5 It should be ascertained from rolling marks on the metal whether the material for all the spans in a particular bridge is from the same source and, if possible, rolled in the same year. If this is not the case, the number of pieces of metal indicated above shall be obtained for each different source of supply of the material.

D-1.6 Test-pieces should, if possible, conform to Indian standard tensile test-pieces but it shall be permissible to use test-pieces of smaller size or miniature test-pieces, if desired, provided the elongation is measured on the appropriate gauge length.

D-1.7 In the case of abnormally low-test values, it shall be permissible to repeat the test on two additional specimens cut from the same position and to substitute the lower of the two additional results for the original one.

D-1.8 The permissible working stress in tension f in kg/ sq. mm (T/sq.in.) shall be determined by the following formula:

$$f = \frac{\text{ultimate stress}}{3} - \frac{(10 - \text{Elongation Percentage})}{1.907}$$

[For FPS units:

$$f = \frac{\text{ultimate stress}}{3} - \frac{(10 - \text{Elongation Percentage})}{3} \ ]$$

subject to a maximum value of $f = \frac{\text{ultimate stress}}{3}$

The above-mentioned formula shall be applied to each individual test and the permissible stress (f ') shall be the average of the worst 50% of the results thus obtained.

Note:-
The value of permissible working stress in tension (f ') thus obtained shall be subject to a maximum limit of 14.2 kg/sq mm (9.0T/sq in.) in the case of early steel and 9.5 kg/sq mm (6.0T/sq.in.) in the case of wrought iron.

D-1.9 When the permissible working stress in tension (f ') as determined in accordance with para D-1.8, lies between 9.5 kg/sq mm and 14.2 kg/sq mm (6.0T/sq in. and 9.0 T/sq. in.) in the case of early steel, the permissible stress in compression, shear, etc should be arrived at by modifying the respective percentages shown in clause 3.18 in the manner indicated below:-

$$p^1 = p + \frac{(100 - p)(f' - 9.5)}{(14.2 - 9.5)}$$

[For FPS units: -

$$p^1 = p + \frac{(100 - p)(f' - 6)}{(9 - 6)} \quad]$$

where p^1 is the modified percentage, and

p is the percentage indicated in clause 3.18.

D-1.10 When the permissible working stress in tension (f ') determined in accordance with D-1.8 is less than 9.5 kg/sq. mm (6.0T/sq.in.) the permissible stresses in compressions, shear, etc. for both wrought iron and early steel, shall be obtained by multiplying the respective percentages shown in clause 3.18 by the factor f '/9.5 (f '/6 for FPS units).

APPENDIX 'E'

Method of Computing Stresses in Rivets at the Ends of Existing Plate Girders.

E-1.1 The end shear due to dead load, live load and impact having been calculated, a parabola of bending moment due to a load equal to twice the end shear should be constructed on a base equal to the effective length of the span and from this the curve of flange force should be plotted, using the effective depth of the girder at the various points in the span.

E-1.2 If the web is in one length or, if jointed, is adequately covered for resisting bending force, the curve of flange stress may be reduced in the ratio of:

$$\frac{a}{a+\frac{W}{8}}$$

Where, a = gross area of one flange, and W = gross area of web.

E-1.3 A tangent should then be drawn to the curve of flange force, starting from the first rivet at extreme end of girder and the force (S) at the tangent point (X) ascertained. Then the number of rivets (N) required for the flange force increments in the length L between the first rivet and point x will be :-

$N = \frac{S}{R+F}$ and the pitch required will be $P_h = \frac{L}{N}$

Where

R= bearing value of a rivet,

F= assumed friction value for a rivet, viz.,

for a 25 mm (1 in.) dia rivet F = 1.27 tonnes (1.25 ton).

for a 22 mm (7/8 in.) dia rivet F = 1.01 tonnes (1.0 ton). and

for a 19 mm (¾ in.) dia rivet F = 0.76 tonnes (0.75 ton).

E-1.4 Beyond X the number of rivets required in any length L, equal to 1/12th effective span will be:-

$N = \frac{S}{R+F}$ and $P_h = \frac{L}{N}$

Where:

S represents the increase in flange force in the length, L beyond the flange force previously dealt with.

E-1.5 If there is vertical load of W per unit length on the rivets, the required pitch (P_v) for this load will be:-

$$P_v = \frac{R+F}{W}$$

And the final resultant pitch required will be $P = \frac{P_h \times P_v}{\sqrt{P_v^{\,2} + P_h^{\,2}}}$

Appendix 'G'
(Re-Revised)

Fatigue Assessment For Steel Bridges

1. General

"The process of progressive localized permanent structural change occurring in a material subjected to conditions which produce fluctuating stresses and strain at some point or points and which may culminate in cracks or complete fracture after a sufficient number of fluctuations."

The above definition of fatigue implies that fatigue in materials is the phenomenon which causes failure of any structural member, under the action of repetitive loads, to take place at stress levels that are substantially less than those associated with failure under static loading conditions. Railway bridges are dynamically loaded structures which are subjected to complex fluctuating loads of varying amplitudes. Therefore, an assessment for fatigue is required to be made if the bridges are to be designed for a definite service life.

2. Scope

2.1. The fatigue assessment shall be deemed to apply to structures which comply with all applicable codes of practice or regulations and have been analyzed and designed in accordance with accepted principles and practices.

2.2. The structural materials and fabrication procedures shall be deemed to comply with all applicable codes of practice or regulations.

2.3. This document is restricted in scope to the assessment of adequacy of members, connections and joints of railway bridges subjected to fatigue loading for a specified design life.

2.4. The assessment procedure contained herein shall be applicable to all grades of structural steel, conforming to applicable codes of practice or regulations.

2.5. The assessment procedure will not be applicable to the following:-

a. Corrosion fatigue

b. Low cycle(high stress) fatigue (No of cycles to failure < 10,000)

c. Thermal fatigue

d. Stress corrosion cracking

e. High temperatures $>150^0$C

f. Low temperatures (brittle transition temperature)

g. Aerodynamically induced vibrations

3. Limitations of provisions in this Appendix

3.1. For fatigue assessment, all nominal stresses, direct or shear, shall be within the elastic limits of the material. The range of the design values of such

stresses shall not exceed 1.5 f_y for normal stresses or 1.5 $f_y/\sqrt{3}$ for shear stresses.

3.2. The fatigue strengths specified in this document are applicable to structures where suitable corrosion protection measures have been undertaken and corrosion is not allowed to take place.

4. Terms and definitions

4.1. General

4.1.1 Fatigue: The phenomenon of damage in a structural part through crack initiation and/or gradual crack propagation caused by repeated stress fluctuations.

4.1.2 Nominal stress: in the parent material or in a weld adjacent to a potential crack location calculated in accordance with elastic theory excluding all stress concentration effects. The nominal stress as specified can be a direct stress, a shear stress, a principal stress or an equivalent stress as appropriate, unless explicitly specified.

4.1.3 **Geometric stress (hot spot stress):** The geometric stress is the maximum principal stress in the parent material adjacent to the weld toe, taking into account stress concentration effects due to the overall geometry of a particular constructional detail. (*Local stress concentration effects e.g. from the weld profile shape already included in the detail categories need not be considered separately.)*

4.1.4 Modified nominal stress: A nominal stress multiplied by an appropriate stress concentration factor k_f, to allow for a geometric discontinuity that has not been taken into account in the classification of a particular constructional detail.

4.1.5 Residual stress: Residual stress is a permanent state of stress in a structure that is in static equilibrium and is independent of any applied action. Residual stresses can arise from rolling stresses, cutting processes, welding shrinkage or lack of fit between members or from any loading event that causes yielding of part of the structure.

4.2. Loading and stress parameters

4.2.1 Load/Loading event: A defined sequence of loads (say, a train) which is passed over the structure a definite number of times during the life of a bridge. This shall usually consist of a sequence of axle loads, specified by the magnitude of the load and the interval between successive axles, or recommended equivalents to represent the passage a train.

4.2.2 Stress history: A record of the stress variation at a particular point in the structure during a load/loading event obtained either by analytical or experimental means.

4.2.3 Stress range: The algebraic difference between the two extremes of a particular stress cycle forming part of the stress history is denoted as a stress range.

4.2.4 Stress range spectrum: The stress range spectrum is a histogram of different stress ranges and their frequency of occurrence for a particular loading event.

4.2.5 Design Spectrum: The combination of all stress range spectra applicable to the fatigue assessment.

4.2.6 Rainflow method / Reservoir method: These are cycle counting techniques used to count the number of stress cycles corresponding to each stress range in a given stress history to derive a stress range spectrum.

4.2.7 Fatigue loading: The fatigue loading means a set of specific train loads and their daily frequency indicating the traffic density in terms of GMT (Gross million tons per annum). The traffic load models will consist of a combination of trains to which the bridge may be subjected within a specified time frame, usually specified by the passage of trains on a daily basis.

4.2.8 Design life: The specified period for which a structure is expected to perform safely with an acceptable probability that failure due to fatigue will not occur.

4.2.9 Fatigue life: The predicted period, expressed in years, preceding fatigue failure at a structural joint or detail based on analytical calculations or experimental observations.

4.2.10 Damage: Damage is the ratio of the actual number of cycles a member detail/connection is subjected to and the number of cycles to failure at a specific stress range. This is computed for various stress ranges and added up as specified.

4.2.11 Equivalent constant amplitude stress range: The constant amplitude stress range that would result in the same fatigue damage as the spectrum of actual variable amplitude stress ranges.

4.2.12 Equivalent constant amplitude fatigue loading: Simplified constant amplitude loading representing the fatigue effects of actual variable amplitude loading events based on the concept of equivalent damage.

4.3. Fatigue strength

4.3.1 Detail category: The designation given to a particular structural detail for a given direction of stress fluctuation to indicate which fatigue strength curve is applicable for fatigue assessment. This is denoted by a number which represents the magnitude in N/mm^2 of the stress range which is associated with an endurance of 2 million cycles for that particular category.

4.3.2 Fatigue strength curve(S - N curve): The fatigue strength curve or the S-N curve is a logarithmic relationship between stress range and the corresponding cycles to failure, based on the category of the detail under consideration. The S-N curves are defined separately for each detail category and may be modified in accordance with other provisions, as applicable.

4.3.3 Reference fatigue strength: The reference fatigue strength for a structural detail or connection is the magnitude in N/mm^2 of the constant amplitude stress range (direct or shear as applicable) associated with an endurance of 2 million cycles in S-N curve.

4.3.4 Endurance: Endurance is the duration of life to fatigue failure, expressed in cycles under the action of a constant amplitude stress history.

4.3.5 Limiting Stress Range: The limiting stress range for a particular structural connection or detail is the stress range for that particular detail category which corresponds to an endurance of 10000 cycles. If either the constant amplitude stress range or any of the variable amplitude stress ranges (direct

or shear as applicable) exceed the limiting stress range then the provisions contained herein will not be applicable.

4.3.6 Constant Amplitude Fatigue Limit: The constant amplitude fatigue limit is the limiting magnitude of the stress range (direct or shear, as applicable) below which no fatigue damage is deemed to accrue for constant amplitude loading of that particular detail. Under variable amplitude stresses, all stress ranges must be below this limit for no fatigue damage to occur.

4.3.7 Cut off limit: The limiting value of the stress range, below which, the stress ranges do not contribute to the accumulated damage in variable amplitude stress conditions.

5. List of symbols

Symbol	Definition
$\Delta\sigma$	Stress range (direct stress)
$\Delta\tau$	Stress range (shear stress)
$\Delta\sigma_E$, $\Delta\tau_E$	Equivalent constant amplitude stress range related to given number of cycles
$\Delta\sigma_{E,2}$, $\Delta\tau_{E,2}$	Equivalent constant amplitude stress range related to 2 million cycles
$\Delta\sigma_C$, $\Delta\tau_C$	Reference value of the fatigue strength at N_C = 2 million cycles
$\Delta\sigma_D$, $\Delta\tau_D$	Fatigue limit for constant amplitude stress ranges at N_D = 5 million cycles, unless otherwise specified
$\Delta\sigma_L$, $\Delta\tau_L$	Cut off limit for stress ranges at N_L = 100 million cycles
$\Delta\sigma_{SL}$, $\Delta\tau_{SL}$	Limiting stress range for a detail category at 10000 cycles
γ_{Ff}	Partial safety factor for equivalent constant amplitude stress range $\Delta\sigma_E$, $\Delta\tau_E$
γ_{Mf}	Partial safety factor for fatigue strength $\Delta\sigma_C$, $\Delta\tau_C$
m	Slope of fatigue strength curve
λ_i	Damage equivalence factors
Φ	Dynamic impact factor
log	Logarithm to the base 10
k_s	Reduction factor for fatigue stress to account for size effects
k_f	Stress concentration factor
N_R (or N)	Design life expressed as number of cycles related to a stress range i.e. $\Delta\sigma_R$ **or** $\Delta\tau_R$ **.**

Note: Other symbols occurring in text do not conform to universally accepted nomenclature and are defined wherever they first occur.

6. Fatigue loads

6.1. The fatigue loading specified in this Appendix shall be used for the determination of stresses at critical locations of the railway bridge, by appropriate and accepted methods of analysis. The stresses so determined will form the basis of fatigue assessment.

6.2. For fatigue life assessment, only live load and associated effects such as dynamic effects, centrifugal effects, longitudinal loads and racking loads specified in Bridge rules shall be considered subject to the following:

6.2.1 For fatigue assessment, 50% of the impact loads specified in Bridge Rules shall be considered.

6.2.2 The fatigue assessment can be done for traffic forecast on a bridge based on actual loading history of trains passed over the bridge and/or future projection of traffic. The future traffic models to be used shall be as specified by Chief Bridge Engineer. Alternately, it can be done for standard train combinations. The following standard train combinations have been considered while formulating the simplified provisions for design as per this code:

6.2.2.1 The standard traffic models for 25 t loading -2008 to be adopted for fatigue assessment shall be in accordance with Table - 1, Appendix G-I.

6.2.2.2 The standard traffic models for 32.5 t loading (DFC Loading) to be adopted for fatigue assessment shall be in accordance with Table - 2, Appendix G-I.

6.3. The loads shall be placed at different positions and the variation of stress with the same shall be worked out for carrying out fatigue assessment.

6.4. In case of bridges with multiple tracks, loading shall be done as follows:

(a) The train load shall be applied on a track which produces the greatest stress at the detail under consideration.

(b) 15%, unless otherwise specified, of the train loads scaled in magnitude shall be applied on any other track so as to produce the greatest stress at the detail under consideration.

(c) The train and its scaled counterpart, as described in (b) above shall be assumed to move in phase in the same direction.

7. Partial safety factors

7.1. General: The values of the partial safety factors to be used shall be as specified herein unless otherwise recommended by the Railway Board, taking into due consideration:

(a) the ease of access for inspection or repair and likely frequency of inspection and maintenance,

(b) the consequences of failure.

7.2. Partial safety factor for fatigue loading γ_{Ff} : To take account of uncertainties in the fatigue response analysis, the design stress ranges for the fatigue assessment procedure shall incorporate a partial safety factor γ_{Ff}**.** The partial safety factor γ_{Ff} covers the uncertainties in estimating:

(a) the applied load levels,
(b) the conversion of these loads into stresses and stress ranges,
(c) the equivalent constant amplitude stress range from the design stress range spectrum,
(d) the design life of the structure, and the evolution of the fatigue loading within the required design life of the structure.

7.3. Partial safety factor for fatigue strength γ_{Mf}: In the fatigue assessment procedure, in order to take account of uncertainties in the fatigue resistance, the design value of the fatigue strength shall be obtained by dividing by a partial safety factor γ_{Mf} .The factor γ_{Mf} covers the uncertainties due to the effects of:

(a) the size of the detail,
(b) the dimensions, shape and proximity of the discontinuities,
(c) local stress concentrations due to welding uncertainties.
(d) variable welding processes and metallurgical effects.

7.4. Values of partial safety factors: The values of the partial safety factor for fatigue loading (γ_{Ff}) and fatigue strength (γ_{Mf}) shall be taken as follows;

γ_{Ff} **= 1.00**

γ_{Mf} **= 1.15**

8. Methodology for Fatigue Assessment: The fatigue assessment shall be carried out as follows:

8.1. No fatigue assessment: Fatigue assessment is not required in the following cases:

8.1.1 The largest nominal stress range $\Delta\sigma$ satisfies

$$\gamma_{Ff} * \Delta\sigma \le 26/\gamma_{Mf} \text{ N/mm}^2$$

8.1.2 The total number of stress cycles N satisfies

$$N \le 2\times10^6 * \left[\frac{36/\gamma_{Mf}}{\gamma_{Ff} * \Delta\sigma_{E,2}}\right]^3$$

Where $\Delta\sigma_{E,2}$ is the equivalent constant amplitude stress range in N/mm^2 .

8.1.3 For a detail for which a constant amplitude fatigue limit $\Delta\sigma_D$ is specified, the largest stress range (nominal or geometric as appropriate) $\Delta\sigma$ satisfies the relation

$$\gamma_{Ff} * \Delta\sigma \le \Delta\sigma_D / \gamma_{Mf}$$

8.2. Classification of details: All details which are to be designed under fatigue shall first be classified so that standard curves known as S-N curves (explained in clause 10) shall be used wherever possible. The details shall be classified as per clause 9.

8.3. Determination of fatigue strength: Corresponding to the detail classification, the fatigue strength shall be worked out as per clause 10.

8.4. Determination of stress history: For each detail to be studied under fatigue, stress histories to be used for fatigue study have to be determined. If actual field measurements or detailed analytical analysis of stresses is being done, clauses 11 and 12 shall be followed. For new construction, especially where the accurate traffic details are not available, simplified method given in clause 14 may be followed.

8.5. Fatigue Assessment: The fatigue assessment of each detail shall be done as per clause 13.

8.6. Simplified fatigue analysis may be done as per clause 14 if actual field measurements or detailed analytical analysis of stresses are not done**.**

9. Classification of details: The structural connections and details, non-welded and welded, are divided into several detail categories, each corresponding to a specific S-N curve depending upon

- The geometrical arrangement of the detail.
- The direction of the fluctuating stress relative to the detail.
- The location of potential crack and direction of propagation.
- The method of fabrication and inspection of the detail.

9.1. In some welded joints, there are several locations at which fatigue cracks may develop, e.g. at the weld toe in each of the parts joined, at the weld ends, and/or in the weld itself. Each such location should be classified separately and assessed independently for fatigue performance.

9.2. The detail categories of structural connections and details has been given in tables in Appendix G-II as follows :-

9.2.1 Table G-II.1: Non-welded details

9.2.2 Table G-II.2: Welded built-up sections

9.2.3 Table G-II.3: Transverse butt welds

9.2.4 Table G-II.4: Welded attachments and stiffeners

9.2.5 Table G-II.5: Load carrying welded joint

9.2.6 Table G-II.6: Fatigue resistance against geometric stress for cracks initiating from toes of welds.

NOTE: Table G-II.*6 does not cover effects of misalignment. The effect of misalignment has to be considered explicitly in determination of stress. Further, it does not cover fatigue initiation from the root followed by propagation through the throat.*

10. Determination of fatigue strength

10.1. **S-N Curves:** The fatigue strength for nominal stresses is defined by a series of S-N curves(log $\Delta\sigma_R$ – log N, or, log $\Delta\tau_R$ – log N), each corresponding to a specific detail category. Each curve is a log-log plot of the stress range against the number of cycles to failure at that stress range where the logarithms are to the base 10. A typical fatigue strength curve is shown in figure 10.1.

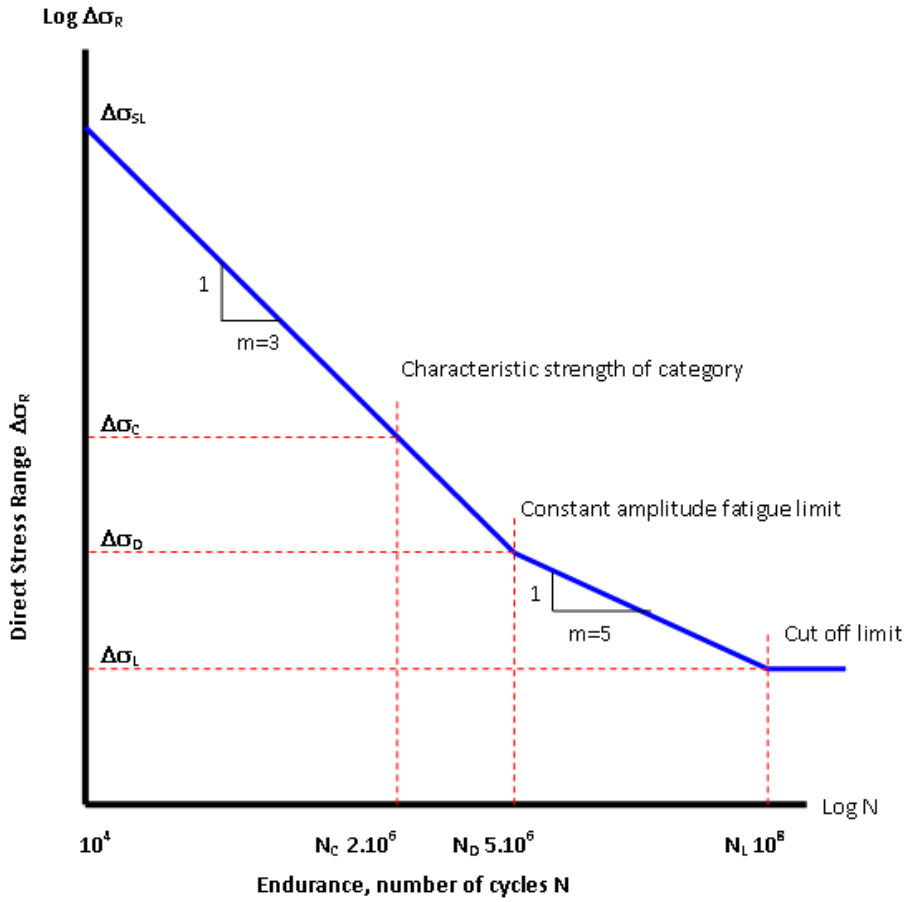


Figure 10.1: Typical fatigue strength (S-N) curve

10.2. Parameters of S-N curves

10.2.1 Each detail category is characterized by a number which represents, in N/mm^2, the reference value $\Delta\sigma_C$ or $\Delta\tau_C$ as applicable for the fatigue strength at 2 million cycles.

10.2.2 The S-N curves include the effects due to :-

- Local stress concentration
- Size and shape of acceptable discontinuities
- The stress directions
- Residual stresses
- Metallurgical conditions

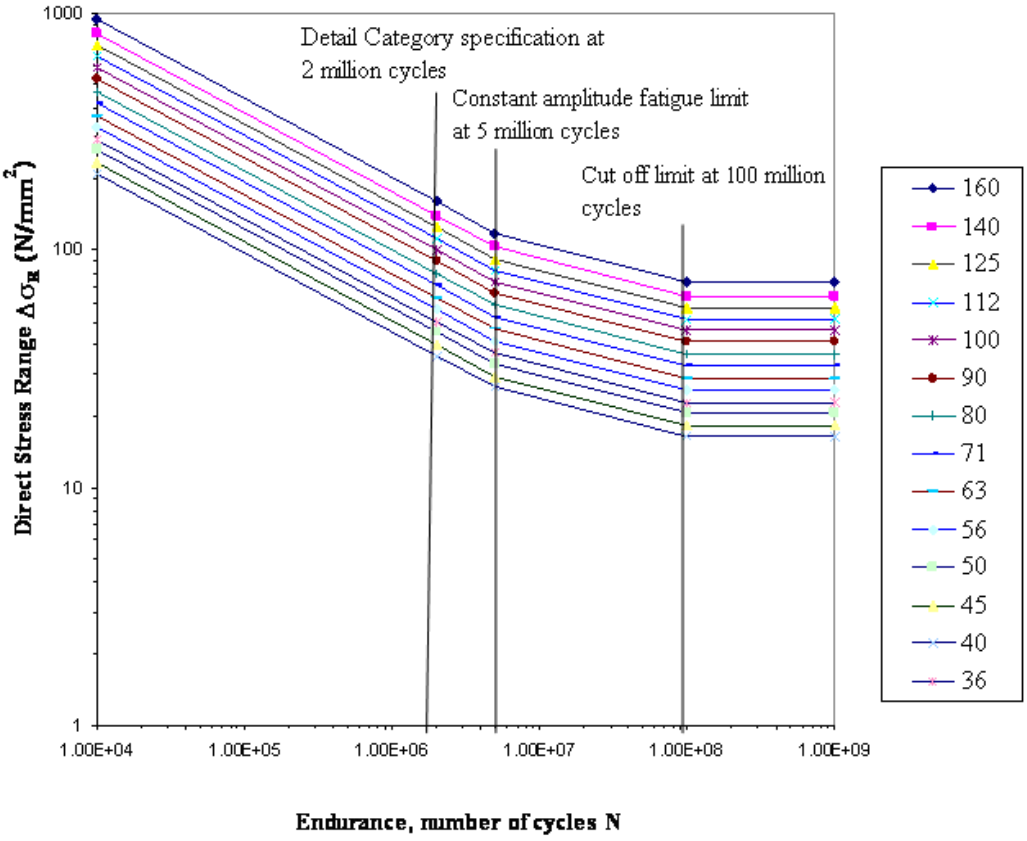


Figure 10.2: Fatigue Strength curves for normal stress ranges

10.2.3 S-N curve for constant amplitude normal stress ranges: These curves for different fatigue categories are shown in figure 10.2. Each curve is described as below:

10.2.3.1 **From 10^4 cycles to 5 x 10^6 cycles**, the curve has a negative slope of 3. The value of $\Delta\sigma_D$ at 5 x 10^6 cycles is called constant amplitude fatigue limit. The fatigue strength in this part is defined by:

$N_R * (\Delta\sigma_R)^m = 2 * 10^6 * (\Delta\sigma_C)^m$, with m=3 for $N_R \leq 5*10^6$

Where $\Delta\sigma_D = \left(\frac{2}{5}\right)^{1/3} * \Delta\sigma_C = 0.7368\Delta\sigma_C$

and N_R is the number of cycles to failure corresponding to $\Delta\sigma_R$ read from the appropriate S-N curve.

10.2.3.2 **From 5 x 10^6 cycles to 1 x 10^8 cycles**, the curve has a negative slope of 5. The value of $\Delta\sigma_L$ at 100 million cycles is called cut off limit. The fatigue strength in this part is defined by:

$N_R * (\Delta\sigma_R)^m = 5 * 10^6 * (\Delta\sigma_D)^m$ with m=5 for $5*10^6 < N_R \leq 10^8$

Where $\Delta\sigma_L = \left(\frac{5}{100}\right)^{1/5} * \Delta\sigma_D = 0.5493 * \Delta\sigma_D$

10.2.3.3 **Beyond 1 x 10^8 cycles**, the curve has NIL slope and there is no fatigue damage.

10.2.4 S-N curve for constant amplitude shear stress ranges: These curves for different fatigue categories are shown in figure 10.3. Each curve is described as below:

10.2.4.1 **From 10^4 cycles to 1 x 10^8 cycles**, the curve has a negative slope of 5. The value of $\Delta\tau_L$ is the cut off limit at 100 million cycles. The fatigue strength curves for shear stress are defined as:

$N_R * (\Delta\tau_R)^m = 2 * 10^6 * (\Delta\tau_C)^m$, with m=5 for $N_R \leq 10^8$

Where $\Delta\tau_L = \left(\frac{2}{100}\right)^{1/5} * \Delta\tau_C = 0.4573\Delta\tau_C$ is the cut off limit at 100 million cycles

10.2.4.2 **Beyond 1 x 10^8 cycles**, the curve has NIL slope and there is no fatigue damage.

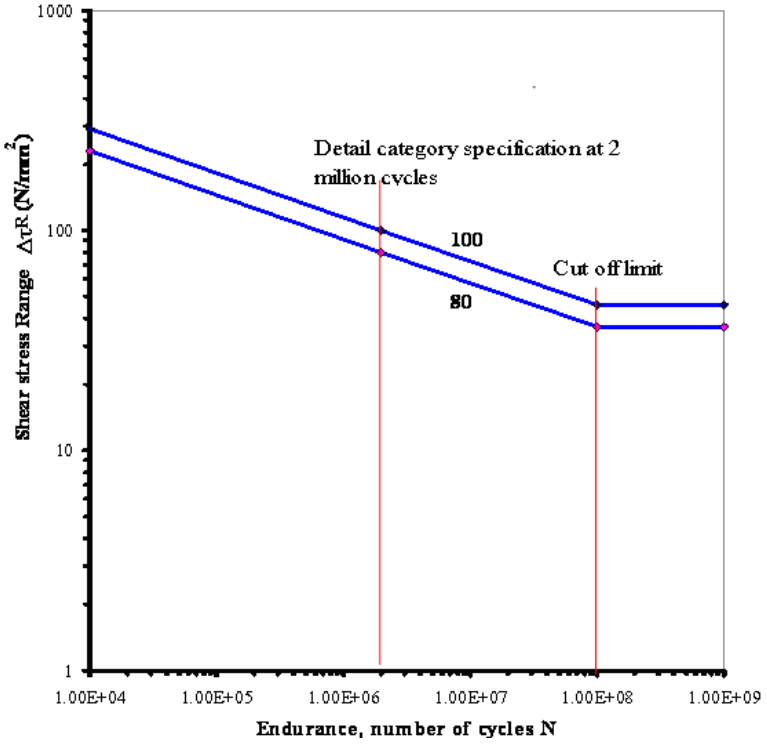


Figure 10.3: Fatigue Strength curves for shear stress ranges

10.2.5 Equations defining S-N curves: The fatigue strength curves for nominal normal/shear stresses are also defined by

$\log N_R = \log a - m * \log \Delta\sigma_R$ or $\log N_R = \log a - m * \log \Delta\tau_R$

where $\Delta\sigma_R$ or $\Delta\tau_R$ is the fatigue strength

N_R is the corresponding number of cycles to failure of stress range $\Delta\sigma_R$ or $\Delta\tau_R$

m is the constant slope of the fatigue strength curves

log a is a constant which depends on the specific segment of the fatigue curve

The numerical values for the fatigue strength curves for normal and shear stress ranges as defined by above are given in Tables 10.1 and 10.2 respectively.

Table 10.1: Numerical values for fatigue strength curves for normal stress ranges

Detail Category $\Delta\sigma_C$ (N/mm^2)	log a for $N_R \leq 10^8$		Stress Range at Constant amplitude Fatigue limit ($N_R = 5*10^6$) $\Delta\sigma_D$(N/mm^2)	Stress Range at Cut off limit ($N_R = 10^8$) $\Delta\sigma_L$(N/mm^2)	Limiting Stress Range ($N_R = 10^4$) $\Delta\sigma_{SL}$(N/mm^2)
	$N_R \leq 5 * 10^6$ m = 3	$N_R \geq 5 * 10^6$ m = 5			
160	12.913	17.056	118	65	936
140	12.739	16.766	103	57	819
125	12.592	16.520	92	51	731
112	12.449	16.282	83	45	655
100	12.301	16.036	74	40	585
90	12.164	15.807	66	36	526
80	12.010	15.551	59	32	468
71	11.855	15.292	52	29	415
63	11.699	15.032	46	25	368
56	11.546	14.777	41	23	327
50	11.398	14.531	37	20	292
45	11.261	14.302	33	18	263
40	11.107	14.046	29	16	234
36	10.970	13.817	27	15	211

Table 10.2: Numerical values for fatigue strength curves for shear stress ranges

Detail Category $\Delta\tau_C$(N/mm^2)	log a for $N \leq 10^8$ m = 5	Stress Range at Cut off limit($N = 10^8$) $\Delta\tau_L$(N/mm^2)	Limiting Stress Range($N = 10^4$) $\Delta\tau_{SL}$(N/mm^2)
100	16.301	46	289
80	15.816	37	231

10.3. Modifications to the fatigue strength

10.3.1 To account for reversal of stresses: In non-welded or stress relieved details the effective stress range to be considered for fatigue assessment shall be determined by adding the tensile portion of the stress range with 70% of the compressive portion of the stress range as shown in figure 10.4.

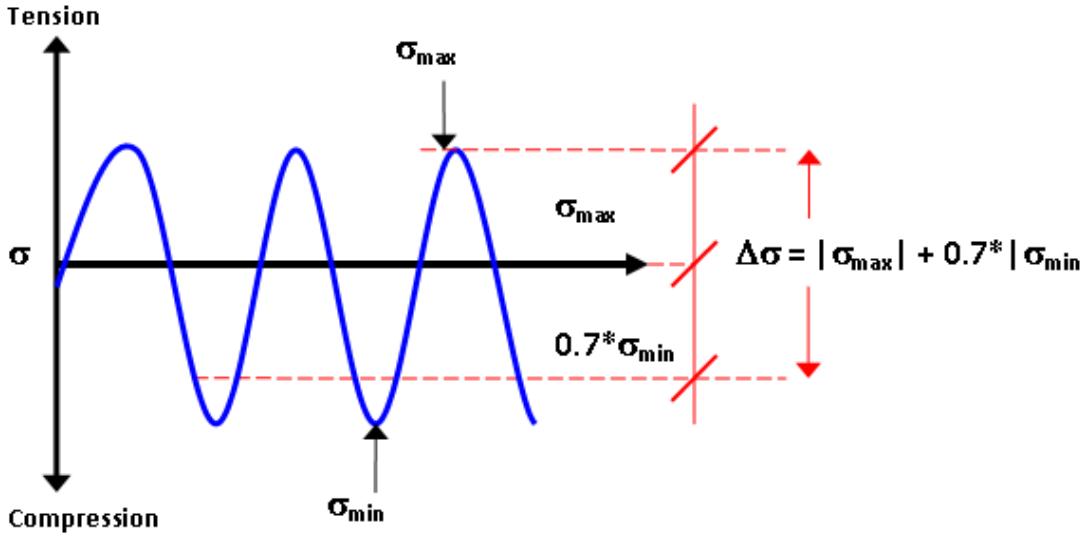


Figure 10.4: Modified stress range for non-welded or stress relieved details

10.3.2 To account for the influence on the fatigue strength of the thickness of the parent metal in which the potential cracks may initiate and propagate:

10.3.2.1 The reduction in the fatigue strength will be applicable only to those structural details with welds transverse to the direction of the normal stress.

10.3.2.2 Where the material thickness of the structural detail is greater than 25 mm, the effect of thickness shall be accounted for by reducing the fatigue strength as :-

$$\Delta\sigma_{R,t} = \Delta\sigma_R * (25/t)^{0.20}, \text{ where } 25/t \leq 1.$$

10.3.2.3 Where the detail category in the classification tables indicates a specific variation in the fatigue strength with thickness then 10.3.2.1 will not be applicable.

10.3.3 Modified fatigue strength is applicable to structural details duly marked with an asterisk in the detail classification table G.II.5. Such details have been allocated a category lower than the stress range corresponding to 2 million cycles. The classification of such details may be upgraded by one category provided that fatigue strength curves are adopted such that the constant amplitude fatigue limit is at 10 million cycles for a slope of m = 3 as shown in figure 10.5 and the numerical values for the modified fatigue strength curves are as indicated in table 10.3.

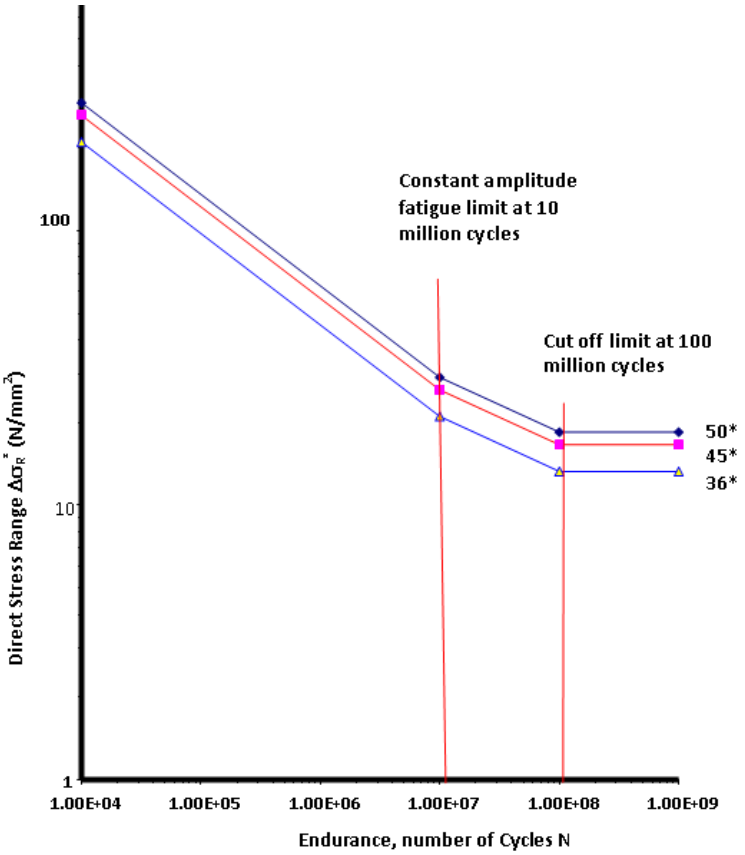


Figure 10.5: Modified fatigue Strength Curves

Table 10.3: Numerical values for modified fatigue strength curves for normal stress ranges

Detail Category $\Delta\sigma_C$ (Nominal)	log a for $N \leq 10^8$		Stress Range at Constant amplitude Fatigue limit ($N = 10^7$) $\Delta\sigma_D$ (N/mm^2)	Stress Range at Cut off limit ($N = 10^8$) $\Delta\sigma_L$ (N/mm^2)	Limiting Stress Range ($N = 10^4$) $\Delta\sigma_{SL}$ (N/mm^2)
	$N \leq 10^7$ m = 3	$N \geq 10^7$ m = 5			
50*(56)	11.546	14.576	33	21	327
45*(50)	11.398	14.330	29	18	292
36*(40)	11.107	13.845	23	15	234

NOTE: Values in parentheses indicate the next higher category for which the constants are evaluated as per clause 10.3.3.

11. **Determination of stresses to be used for fatigue design:** For each structural detail or joint being assessed for fatigue, typical load event (or train) produces a stress history plot, depending on position of the train at different time intervals. A typical stress history with time plot is shown in Figure A.1 of Appendix G-III. These stresses for different positions of train(s) shall be obtained for member(s) as follows:

11.1. Field Measurements: The stresses measured on members while actual trains/ test trains pass over the bridge and the plot of variation of stresses with position of train can be used for fatigue assessment of existing bridges. Based on these plots, stress history plots shall then be obtained for the other trains plying/ likely to ply on the bridge. For parameters difficult to replicate/ measure in field, such as impact, suitable modifications shall be made as per Bridge rules. This method captures the actual behaviour of girders. However, if the actual plot shape/ magnitude of stresses measured in field vary too much as compared with the theoretical expected stresses, reasons for the same shall be studied and designer shall decide if the measured stresses are reliable for fatigue assessment studies or not.

11.2. Theoretical Computations: Alternately, the theoretical plot of stress with position of actual/expected moving train loads shall be determined for the fatigue loads specified in clause 6 above. The stresses due to the moving train loads shall be determined on the basis of static linear elastic analysis carried out in accordance with accepted principles and practices, unless otherwise stated or implied, taking into account all axial, bending and shear stresses occurring under the prescribed fatigue loading.

11.3. Stresses for Fatigue Assessment:

11.3.1 For a particular class of construction detail, the stresses to be considered may be nominal stresses or shear stresses or both.

11.3.2 When a constructional detail is defined in the detail classification tables (Table G.II.1 to G.II.5), the nominal stress range shall be used.

11.3.3 The effects of geometric discontinuities which are not part of the constructional detail itself, such as holes, cut-outs or re-entrant corners shall be taken into account separately, either by a special analysis or by the use of appropriate stress concentration factors, to determine the modified nominal stress range.

11.4. Modification in measured/ computed stresses:-

11.4.1 The nominal stresses should be calculated at the location of potential fatigue initiation. No redistribution of loads or stresses is permitted from any consideration whatsoever.

11.4.2 Where applicable, effect of the following should be incorporated in the stress calculations:-

(a) Shear lag, restrained torsion and distortion, transverse stresses and flange curvature

(b) Effective width of steel plates

(c) Load application away from joints, member eccentricities at joints and rigidity of joints in triangulated skeletal structures.

(d) Stress concentration effects, when specifically stated as a requirement for a detail or joint, which shall be accounted for by using an appropriate stress concentration factor.

11.4.3 The effects of the following need not be included in the stress calculations

(a) Residual stresses.

(b) Eccentricities arising in a standard detail.

(c) Standard stress concentration associated with a detail as given in tables G-II.1 to G-II.5 which has already been considered in the fatigue detail category.

11.5. The stresses to be determined for purpose of assessment of fatigue shall consist of the stresses as defined in clauses 11.7 and 11.8 for stresses in parent material and stresses in welds respectively.

11.6. Modification of stress ranges based on geometric stress range:

11.6.1 Where abrupt changes of section occur close to the potential crack locations (for details not covered in tables G.II.1 to G.II.5), high stress gradients occur close to a weld in toe joints (covered in Table G-II.6), geometric stress range shall be used.

11.6.2 The geometric stress is the maximum principal stress in the parent material adjacent to the weld toe taking into account only the overall geometry of the joint, excluding local stress concentration effects due to the weld geometry and discontinuities at the weld toe.

11.6.3 The maximum value of the geometric stress range shall be found, investigating various locations at the weld toe around the welded joint or the stress concentration area.

11.6.4 The geometric stresses may be determined using stress concentration factors obtained from parametric formulae within their domains of validity, a finite element analysis or an experimental model.

11.6.5 A fatigue assessment based on the geometric stress range, shall be treated similarly to the assessment methodology given in clause 12, but replacing the nominal stress range by the geometric stress range.

11.7. Stresses for details in the parent material: Depending upon the fatigue assessment to be carried out, either the nominal stresses or geometric stresses shall be evaluated.

- Nominal direct stresses σ
- Nominal shear stresses τ

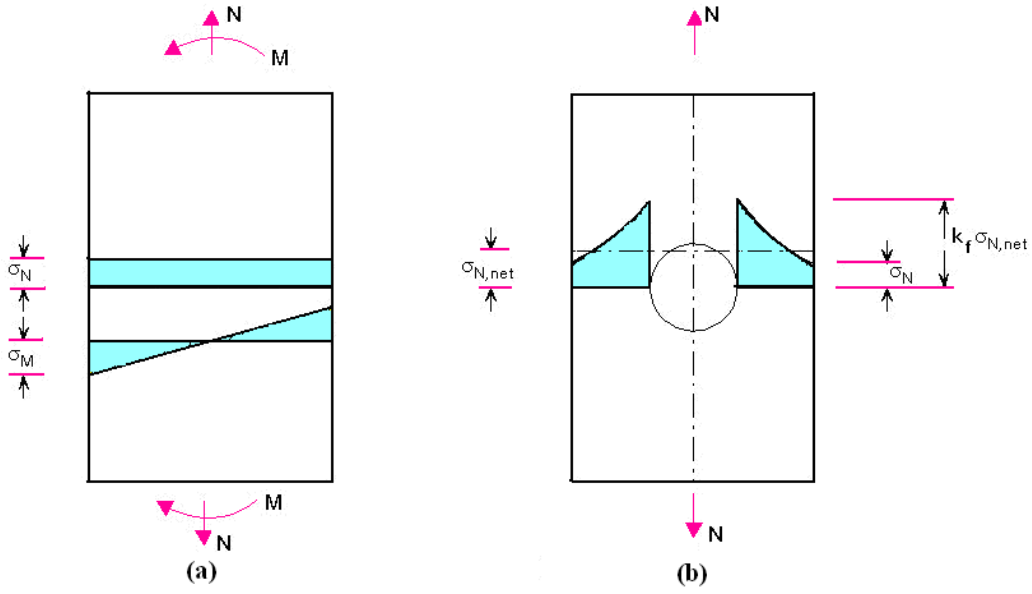


Figure-11.1 Nomination stress and geometric stress concentration

The nominal normal or direct stress when a member is under uni-axial and bending stresses, (Refer figure-11.1(a)), is as calculated according to basic strength of materials theory

$$\sigma_N = \frac{N}{A} \text{ and } \sigma_M = \frac{M * y}{I}$$

Where N and M are the axial force and bending moment at the section

A and I are the cross sectional area and moment of inertia, and

y is the distance from the neutral axis to the extreme fiber.

When geometric stress concentration (such as shown in figure-11.1 (b) other than that already considered in fatigue category) occurs, the nominal stress should be determined as follows

$\sigma_G = k_f * \sigma_{N,net}$

Where σ_G is the effective stress

k_f is the stress concentration factor

and $\sigma_{N,net}$ is the stress calculated on the net area

the fatigue requirements for the structural joint or detail, as applicable.

11.8. Stresses in the welds: In load carrying partial penetration or fillet welded joints, the forces transmitted by a unit length of weld shall be resolved into components transverse and parallel to the longitudinal axis of the weld.

The fatigue stresses in the weld will consist of the following:

- Normal stresses σ_w transverse to the axis of the weld
- Shear stresses τ_w longitudinal to the axis of the weld

The stresses σ_w and τ_w may be obtained by dividing the relevant component of the force transmitted per unit length of the weld, by the throat size 'a'.

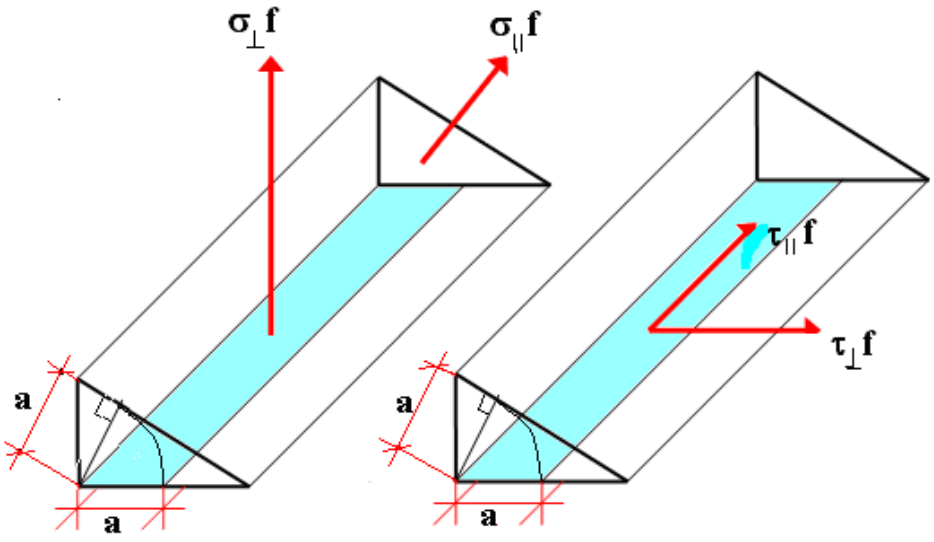


Figure-11.2 stresses in fillet welds

Alternatively, σ_w and τ_w may be obtained by the following, (Refer figure 11.2).

- $\sigma_w = \sqrt{\sigma_{\perp f}^2 + \tau_{\perp f}^2}$
- $\tau_{w} = \tau_{||f}$

It will be necessary to ensure that the effects of the stresses considered individually and in conjunction satisfy the fatigue requirements for the structural joint or detail, as applicable.

12. **Determination of stress ranges and cycles for fatigue life assessment:**

12.1. General: Typical load events analyzed as per clause 11 produce a stress history, with respect to the leading train axle, depending on the location of the structural detail or joint being assessed for fatigue. This variation of stress in the stress history can be highly irregular. The stress history as stated above cannot be used directly to assess the damage and cycle counting techniques are required to be used. The purpose of cycle counting is to reduce a complex stress history to a sequence of stress ranges and the corresponding number of cycles of occurrence in the stress history.

12.2. Methods of cycle counting: There are two established methods of cycle counting namely the “Rainflow method” and the “Reservoir method”, both yielding identical results provided that rainflow counting begins with the highest peak in the loading event.

Generally, rainflow counting is more suited to computer analyses of long stress histories, whereas the reservoir method is most convenient for graphical analyses of short histories.

12.2.1 Determination of stress ranges and cycles by the reservoir method: The method consists of imagining the stress history as the section of a reservoir which is drained successively from each of the lowest points till the reservoir is empty. Each draining operation is considered to be equivalent to one cycle of a stress range equal in magnitude to the maximum height of water drained in that particular operation (see Appendix G-III).

12.2.2 Determination of stress ranges and cycles by the rainflow method: The rainflow method as the name suggests counts half cycles based on the visualization of the complex stress history as a sequence of pagoda roofs over which rain tickles down. In order to achieve the above the stress history is rotated by 90^0 (see Appendix G-III). Counting of cycles shall be done as per rules given in Appendix G-III.

12.3. The values of stress ranges for which cycles are thus counted might be quite variable in magnitude. For further computations, the values of stress ranges are grouped together in different stress range slabs to get the stress histogram. The fatigue assessment is done using this stress histogram. Stress histogram for the stress history has been worked out in clause A3.3 of Appendix G-III.

12.4. For each stress range slab in stress histogram, the corresponding fatigue life can be worked out for the appropriate SN curve applicable to the member detail.

13. Fatigue assessment:-Fatigue assessment refers either to checking if a member has been designed with adequate fatigue life or to verifying if the residual fatigue life of a member is adequate. Stresses determined/ modified in accordance with clause 12 shall be used for this purpose.

13.1. The assessment for fatigue shall be carried out either

- in terms of cumulative damage by comparing the applied damage to the limiting damage, or
- in terms of the equivalent stress range by comparing it with the fatigue strength for a given number of stress cycles.

13.2. Fatigue assessment based on nominal stress ranges:

13.2.1 Constant amplitude loading

For constant amplitude loading the fatigue assessment criterion is:

$$\gamma_{Ff} * \Delta\sigma \leq \Delta\sigma_R/\gamma_{Mf} \text{ or } \gamma_{Ff} * \Delta\tau \leq \Delta\tau_R/\gamma_{Mf}$$

where $\Delta\sigma$ (or $\Delta\tau$) is the nominal stress range.

$\Delta\sigma_R$ (or $\Delta\tau_R$) is the fatigue strength for the relevant detail category for the total number of cycles N during the required design life.

and γ_{Ff} and γ_{mf} shall be as per clause 7.4

Variable amplitude loading

13.2.1.1 For variable amplitude loading defined by a design spectrum, the fatigue assessment shall be based on **Palmgren-Miner rule of cumulative damage given in 13.2.2.3 below.**

13.2.1.2 If the maximum stress range due to the variable amplitude loading is higher than the constant amplitude fatigue limit, then one of the following types of fatigue assessment shall be made:

- Cumulative damage
- Equivalent constant amplitude

13.2.1.3 A cumulative damage assessment may be made using:

$D_d \leq 1$ where $D_d = \sum \frac{n_i}{N_i}$

Where n_i is the number of cycles of stress range $\Delta\sigma_I$ (or $\Delta\tau_i$) during the required design life, taken from stress range spectrum histogram.

and N_i is the number of cycles of stress range $\gamma_{Ff}*\gamma_{Mf}*\Delta\sigma_i$ (or $\gamma_{Ff}*\gamma_{Mf}*\Delta\tau_i$) to cause failure for the relevant detail category, read from the appropriate S-N curve given in clause 10.

13.2.2 For nominal stress ranges, N_i may be calculated as follows ;

(a) if $\gamma_{Ff} \cdot \Delta\sigma_{i} \geq \Delta\sigma_D / \gamma_{Mf}$

$$N_i = 5*10^6 * \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff} * \Delta\sigma_i}\right]^3$$

(b) if $\Delta\sigma_D / \gamma_{Mf} \geq \gamma_{Ff} \cdot \Delta\sigma_{i} \geq \Delta\sigma_L / \gamma_{Mf}$

$$N_i = 5*10^6 * \left[\frac{\Delta\sigma_D/\gamma_{Mf}}{\gamma_{Ff} * \Delta\sigma_i}\right]^5$$

(c) if $\Delta\sigma_L / \gamma_{Mf} \geq \gamma_{Ff} * \Delta\sigma_i$ then N_i may be taken as infinite

13.2.3 For shear stress ranges, N_i may be calculated as follows :

(a) if $\gamma_{Ff} * \Delta\tau_{i} \geq \Delta\tau_{L} / \gamma_{Mf}$

$$N_i = 2*10^6 * \left[\frac{\Delta\tau_D / \gamma_{Mf}}{\gamma_{Ff} * \Delta\tau_i} \right]^5$$

(b) if $\gamma_{Ff} * \Delta\tau_{i} \leq \Delta\tau_{L} / \gamma_{Mf}$ then N_i may be taken as infinite

13.2.4 An equivalent constant amplitude fatigue assessment may be made by checking the criterion:

$$\gamma_{Ff} * \Delta\sigma_E \leq \Delta\sigma_R / \gamma_{Mf} \text{ or } \gamma_{Ff} * \Delta\tau_E \leq \Delta\tau_R / \gamma_{Mf}$$

Where,

$\Delta\sigma_E$ or $\Delta\tau_E$ is the equivalent constant amplitude stress range which, for the given number of cycles leads to the same cumulative damage as the design spectrum, and

$\Delta\sigma_R$ or $\Delta\tau_R$ is the fatigue strength for the relevant detail category for the same number of cycles as used to determine $\Delta\sigma_E$.

13.2.5 An equivalent constant amplitude fatigue assessment may be made alternatively by the following criteria;

$$\gamma_{Ff} * \Delta\sigma_{E,2} \leq \Delta\sigma_C / \gamma_{Mf} \text{ or } \gamma_{Ff} * \Delta\tau_{E,2} \leq \Delta\tau_C / \gamma_{Mf}$$

Where, $\Delta\sigma_{E,2}$ or $\Delta\tau_{E,2}$ is the equivalent constant amplitude stress range for 2 million cycles worked out as per clause 14,

And $\Delta\sigma_C$ or $\Delta\tau_C$ is the reference value of the fatigue strength for the relevant detail (also at 2 million cycles).

13.3. Combination of normal and shear stress ranges:

13.3.1 In the case of a combination of normal and shear stresses the fatigue assessment shall consider their combined effects.

13.3.2 If the equivalent nominal shear stress range is less than 15 % of the equivalent nominal normal stress range, the effects of the shear stress range may be neglected.

13.3.3 At locations other than weld throats, if the normal and shear stresses induced by the same loading event vary simultaneously, or if the plane of the maximum principal stress does not change significantly in the course of a loading event, the maximum principal stress range may be used.

13.3.4 If, at the same location, normal and shear stresses vary independently, the components of damage due to normal and shear stresses shall be determined separately in accordance with the Palmgren-Miner rule and then combined in accordance with

$$D_{d,\sigma} + D_{d,\tau} \leq 1$$

where $D_{d,\sigma} = \sum(n_i / N_i)$ for normal stress ranges $\Delta\sigma_i$

and $D_{d,\tau} = \sum(n_i / N_i)$ for shear stress ranges $\Delta\tau_i$

13.3.5 The criteria specified in 13.3.4 for equivalent constant amplitude stress ranges assumes the form

$$\left[\frac{\gamma_{Ff} * \Delta\sigma_E}{\Delta\sigma_R / \gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff} * \Delta\tau_E}{\Delta\tau_R / \gamma_{Mf}}\right]^5 \leq 1$$

13.3.6 An equivalent constant amplitude fatigue assessment may, alternatively, be made by the following criterion

$$\left[\frac{\gamma_{Ff} * \Delta\sigma_{E,2}}{\Delta\sigma_C / \gamma_{Mf}}\right]^3 + \left[\frac{\gamma_{Ff} * \Delta\tau_{E,2}}{\Delta\tau_C / \gamma_{Mf}}\right]^5 \leq 1$$

13.3.7 Stress ranges in welds shall be determined as specified in Clause 11.6. The components of damage for normal and shear stresses shall be assessed in accordance with the Palmgren-Miner rule and then combined in accordance with

$$D_{d,\sigma} + D_{d,\tau} \leq 1$$

where $D_{d,\sigma} = \sum(n_i / N_i)$ for normal stress ranges σ_{wf}.

and $D_{d,\tau} = \sum(n_i / N_i)$ for shear stress ranges $\Delta\tau_{wf}$.

14. Simplified Approach if actual field measurements or detailed analytical analysis of stresses is not done: For the simplified fatigue assessment, the following procedure may be adopted to determine the design stress spectrum:

14.1. For a detail or structural connection being assessed for fatigue, the maximum stress $\sigma_{P,max}$ or $\tau_{P,max}$ and the minimum stress $\sigma_{P,min}$ or $\tau_{P,min}$ should be determined for the live loads specified in clause 6. Instead of actual train loads, the Equivalent Uniformly distributed loads as specified in IRS Bridge Rules may be used.

14.2. The reference stress range $\Delta\sigma_P$ (or$\Delta\tau_P$) for determining the damage due to the stress spectrum should be obtained from:

$$\Delta\sigma_P = \left|\sigma_{P,\max} - \sigma_{P,\min}\right| \text{ or } \Delta\tau_P = \left|\tau_{P,\max} - \tau_{P,\min}\right|$$

14.3. Modification of the above stress range may be done in accordance with clause 10.3, 11.3 and 11.4, if applicable.

14.4. Design value of equivalent constant amplitude stress range: The design value of equivalent constant amplitude stress range (related to N_C i.e. 2 x 10^6 cycles, $\Delta\sigma_{E,2}$ or $\Delta\tau_{E,2}$)

shall be worked out by multiplying the modified stress range worked out as per clause 14.3 above by damage equivalent factor for railway bridges, λ worked out as per clause 14.6 below.

14.5. Fatigue assessment

The fatigue assessment shall be carried out by ensuring the satisfaction of the following criteria:

$\gamma_{Ff} * \Delta\sigma_{E,2} \leq \Delta\sigma_C / \gamma_{Mf}$

and $\gamma_{Ff} * \Delta\tau_{E,2} \leq \Delta\tau_C / \gamma_{Mf}$

Where $\Delta\sigma_C$ or $\Delta\tau_C$ is the reference value of the fatigue strength for the relevant detail (at 2 million cycles).

14.6. Damage equivalence factors

14.6.1 The damage equivalent factor for railway bridges should be determined from:

$$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4$$

subject to the condition that $\lambda \leq \lambda_{max}$ where $\lambda_{max} = 1.4$

Where,

λ_1 is a factor that takes into account the damaging effect of traffic and depends on the base length of the longest loop of the influence line diagram

λ_2 is a factor that takes into account the annual traffic volume in million tons

λ_3 is a factor that takes into account the design life of the bridge in years

λ_4 is a factor to be taken into account when the bridge structure is loaded on more than one track

λ_{max} is the maximum λ value taking into account the fatigue limit

14.6.2 The value of λ_1 may be obtained from tables 14.6.2 (1) and 14.6.2 (2) 25t loading – 2008 and 32.5t loading (DFC) respectively as a function of the loaded length for the train types included in respective traffic models. The loaded length shall depend upon the influence line diagram of the structural element or detail/connection under consideration. For simplified analysis the maxima for each length category could be adopted. For routes with train types other than those considered above, the designer may use alternative values of λ_1.

Table 14.6.2 (1): λ_1 for 25 T Loading

Span (m)	Train-1	Train-2	Train-3	Train-4	Train-5	Train-6	Train-7	Train-8	Train-9	Train-10	Train-11
0.50	1.30	1.34	1.45	1.28	1.45	1.44	1.48	1.53	1.36	1.09	0.88
1.00	1.29	1.32	1.43	1.28	1.43	1.43	1.48	1.53	1.35	1.08	0.89
1.50	1.28	1.31	1.42	1.27	1.42	1.42	1.47	1.52	1.35	1.07	0.90
2.00	1.27	1.30	1.40	1.27	1.40	1.41	1.47	1.52	1.35	1.06	0.90
2.50	1.26	1.29	1.38	1.26	1.37	1.39	1.46	1.51	1.34	1.05	0.91
3.00	1.24	1.28	1.36	1.25	1.34	1.38	1.45	1.50	1.34	1.04	0.92
3.50	1.22	1.26	1.34	1.23	1.32	1.36	1.44	1.49	1.33	1.03	0.93
4.00	1.20	1.25	1.32	1.22	1.30	1.34	1.43	1.47	1.31	1.02	0.94
4.50	1.18	1.23	1.30	1.21	1.28	1.31	1.42	1.46	1.30	1.00	0.95
5.00	1.17	1.21	1.28	1.19	1.26	1.27	1.41	1.45	1.28	0.99	0.96
6.00	1.12	1.17	1.24	1.15	1.23	1.21	1.39	1.42	1.24	0.96	0.97
7.00	1.08	1.14	1.19	1.11	1.18	1.16	1.37	1.39	1.19	0.93	0.98
8.00	1.05	1.12	1.17	1.09	1.16	1.12	1.38	1.37	1.14	0.91	1.00
9.00	1.03	1.11	1.15	1.07	1.13	1.08	1.38	1.36	1.09	0.90	1.02
10.00	0.96	1.03	1.12	1.15	1.08	1.10	1.37	1.37	1.08	0.89	0.99
12.50	0.89	0.98	1.05	1.06	1.03	1.02	1.32	1.32	1.00	0.86	1.01
15.00	0.87	0.92	1.00	1.01	1.05	1.00	1.30	1.31	0.99	0.83	1.02
17.50	0.82	0.86	0.94	0.93	1.04	0.94	1.24	1.24	0.93	0.78	0.98
20.00	0.83	0.86	0.99	0.94	1.07	0.89	1.13	1.09	0.89	0.79	1.01
25.00	0.76	0.86	0.93	0.85	1.08	0.87	1.13	1.11	0.87	0.75	0.99
30.00	0.77	0.82	0.84	0.80	1.09	0.88	0.98	1.10	0.87	0.69	0.96
35.00	0.73	0.75	0.78	0.76	0.87	0.86	0.93	1.09	0.86	0.66	0.90
40.00	0.66	0.68	0.70	0.73	0.85	0.78	0.84	1.07	0.78	0.63	0.75
45.00	0.64	0.66	0.68	0.77	0.81	0.77	0.82	1.01	0.78	0.63	0.65
50.00	0.62	0.63	0.65	0.77	0.80	0.75	0.79	0.91	0.76	0.62	0.66
60.00	0.59	0.60	0.62	0.77	0.77	0.74	0.77	0.78	0.74	0.60	0.67
70.00	0.58	0.59	0.60	0.75	0.76	0.75	0.77	0.78	0.75	0.61	0.64
80.00	0.56	0.58	0.59	0.66	0.76	0.74	0.76	0.77	0.75	0.60	0.63
90.00	0.56	0.57	0.58	0.64	0.76	0.72	0.75	0.76	0.75	0.58	0.63
100.00	0.55	0.56	0.58	0.61	0.76	0.73	0.74	0.75	0.74	0.55	0.63

Table 14.6.2 (2): λ_1 for 32.5 T Loading

Span (m)	Train-1	Train-2	Train-3	Train-4	Train-5	Train-6	Train-7
0.50	1.53	1.60	1.39	1.59	1.30	1.11	0.86
1.00	1.52	1.59	1.38	1.58	1.28	1.10	0.87
1.50	1.52	1.58	1.37	1.57	1.27	1.09	0.87
2.00	1.52	1.58	1.36	1.57	1.25	1.09	0.88
2.50	1.51	1.57	1.36	1.56	1.24	1.08	0.89
3.00	1.51	1.56	1.35	1.55	1.23	1.08	0.89
3.50	1.51	1.55	1.34	1.54	1.21	1.07	0.90
4.00	1.50	1.54	1.34	1.53	1.20	1.07	0.91
4.50	1.49	1.53	1.33	1.51	1.19	1.06	0.92
5.00	1.47	1.52	1.32	1.50	1.17	1.05	0.93
6.00	1.43	1.50	1.29	1.46	1.14	1.03	0.95
7.00	1.38	1.48	1.28	1.44	1.11	1.02	0.97
8.00	1.34	1.46	1.27	1.42	1.09	1.01	0.99
9.00	1.31	1.45	1.25	1.39	1.06	1.00	1.01
10.00	1.31	1.47	1.30	1.39	1.18	1.06	1.06
12.50	1.22	1.40	1.25	1.33	1.12	1.03	1.09
15.00	1.16	1.34	1.20	1.28	1.08	1.00	1.08
17.50	1.10	1.28	1.15	1.23	1.05	0.97	1.04
20.00	0.96	1.18	1.04	1.15	1.17	1.08	1.08
25.00	0.89	1.11	0.97	0.97	1.10	0.88	0.99
30.00	0.88	1.10	0.88	0.89	0.95	0.75	0.93
35.00	0.84	1.06	0.85	0.84	0.89	0.75	0.90
40.00	0.84	0.89	0.84	0.84	0.84	0.76	0.81
45.00	0.81	0.84	0.80	0.81	0.80	0.73	0.68
50.00	0.80	0.82	0.79	0.80	0.77	0.73	0.70
60.00	0.74	0.75	0.73	0.74	0.70	0.66	0.67
70.00	0.72	0.74	0.70	0.73	0.69	0.63	0.65
80.00	0.72	0.77	0.70	0.76	0.69	0.63	0.63
90.00	0.74	0.76	0.70	0.75	0.70	0.61	0.63
100.00	0.74	0.75	0.70	0.75	0.69	0.60	0.64

14.6.3 The loaded length for determination of appropriate λ_1 should be taken as follows:

(a) **for moments:**

- For a simply supported span, the span length, L
- For cross girders supporting rail bearers (or stringers), the sum of the spans of the rail bearers (or stringers) carried by the cross girder.

(b) **for shear for a simply supported span**

- For the support section, the span length.
- For the mid-span section, 0.4 * the span under consideration.

(c) **for axial force in members of a triangulated truss**

- Base length of loop containing the largest ordinate (+ve or -ve) in member being assessed for fatigue as per the Influence Line Diagram (ILD) of the member (see Fig. 14.1).

(d) **In other cases**

- the same as for moments.

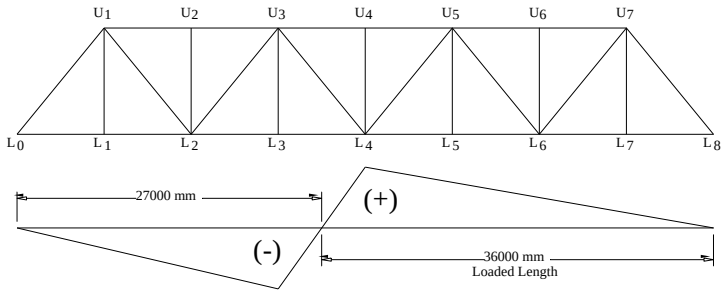


Figure 14.1 Loaded Length for Diagonal U_3-L_4

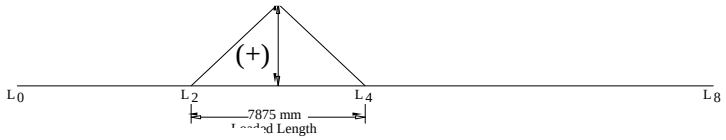


Figure 14.1 (b) Loaded Length for Verical U_3-L_3

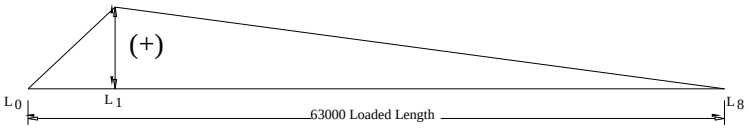


Figure 1(c) Loaded Length for Bottom Chord L_0-L_1 & L_1-L_2

Note :- (+) For Tension and (-) For Compression

Figure 14.1: Loaded lengths for finding λ_1.

14.6.4 The value of λ_2, in terms of the annual volume of traffic may be obtained from the following expression where T_a is the annual volume of traffic expressed in million tons:

$$\lambda_2 = 0.5193 * T_a^{0.2036}$$

14.6.5 The value of λ_3, in terms of the design life may be calculated from the following expression where L_D is the design life in years:-

$$\lambda_3 = 0.3899 * L_D^{0.2048}$$

14.6.6 The value of λ_4, assuming 15% of the total traffic on both tracks crosses whilst on the bridge, shall be obtained from

$$\lambda_4 = 0.7926 * a^2 - 0.7280 * a + 0.9371$$

Where $a = \Delta\sigma_1 / \Delta\sigma_{1+2}$

$\Delta\sigma_1$ = Stress range at the section being checked due to train on one track.

$\Delta\sigma_{1+2}$ = Stress range at the same section due to train load on two tracks.

The values of λ_4 may be calculated for other proportions of crossing traffic from

$$\lambda_4 = \sqrt[5]{n + (1-n)\left[a^5 + (1-a)^5\right]}$$

Where, n is the proportion of traffic that crosses simultaneously on the bridge.

Appendix G-I

Table 1(a) – Traffic Models for 25t Loading

Type of Train	Train No.	Train Composition	Weight per train (t)	GMT/ Train	Class of Traffic							
					Heavy Freight Traffic (100 GMT)		Mixed Traffic Lines with Heavy Traffic (70 GMT)		Sub Urban Traffic (60 GMT)		Mixed Traffic Lines with Light Traffic (40GMT)	
					No. of Trains	GMT	No. of Trains	GMT	No. of Trains	GMT	No. of Trains	GMT
Passenger	1	1+15ICF COACH NON AC	900	0.33	3	1.0	6	2.0	-	-	5	1.7
	2	2+22 ICF COACH NON AC	1400	0.51	2	1.0	10	5.1	7	3.57	5	2.6
	3	2+26 COACH AC	1700	0.62	-	-	14	8.7	7	4.34	-	-
	4	EMU12	700	0.26	-	-	-	-	200	52.0	-	-
Freight	5	2(22.5T)+40 BOXN	4270	1.56	2	3.1	-	-	-	-	4	6.24
	6	2(25T)+55 BOXN	5800	2.12	8	16.96	4	8.48	-	-	9	19.08
	7	2E(2+55 BOXN)	11540	4.21	10	42.1	6	25.21	-	-	1	4.21
	8	2D(2+55 BOXN)	11600	4.23	8	33.84	5	21.15	-	-	1	4.23
	9	Bo-BO +40 BOXN	4200	1.53	2	3.06						
Freight empty	10	2(25T)+55BOXN	1686	0.61	-	-	-	-	-	-	1	0.61
	11	2(22.5T)+40 BOXN	1278	0.47	-	-	-	-	-	-	2	0.9
Total					35	101.06	45	70.64	214	59.91	28	39.57

Table 1 (b) Train Formation details in traffic Model for 25t. Loading–2008

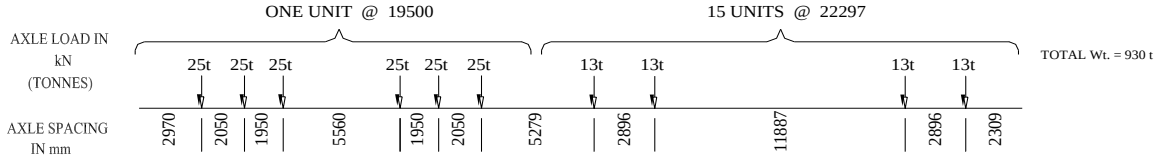
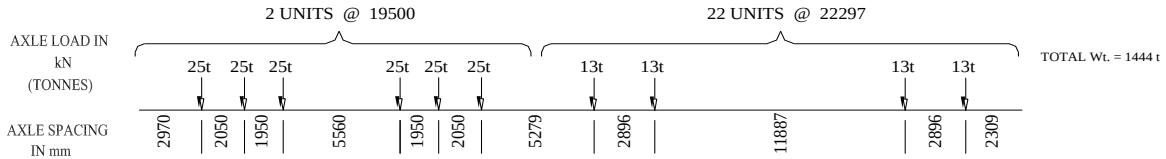
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type - 1	1L+15 ICF COACH (Non AC)	348.676	930	1. PASSENGER TRAIN ONE 25 t. LOCO + 15 ICF COACH NON AC 
Type - 2	2+22 ICF COACH (Non AC)	524.255	1444	2. PASSENGER TRAIN TWO 25 t. LOCO + 22 ICF COACH NON AC 

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Table 1 (b) Train Formation details in traffic Model for 25t. Loading–2008

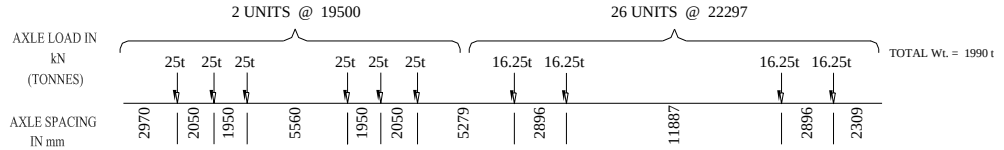
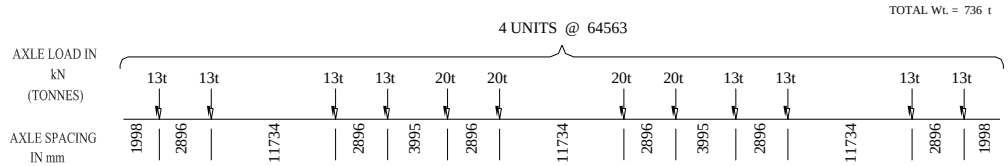
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type - 3	2+26 COACH (AC)	613.443	1990	3. PASSENGER TRAIN TWO 25 t. LOCO + 2 COACH AC 
Type - 4	EMU 12	254.257	736	4. PASSENGER TRAIN EMU 12 (3x4 UNITS) 

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Table 1 (b) Train Formation details in traffic Model for 25t. Loading–2008

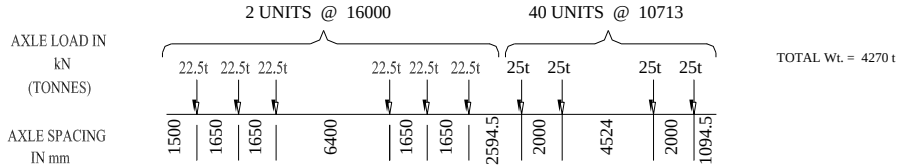
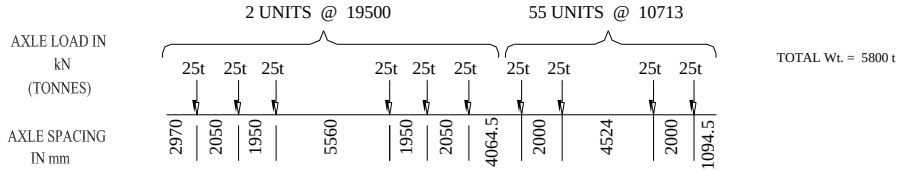
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type - 5	2(22.5t)+40 BOXN	457.925	4270	5. FREIGHT TRAIN TWO 22.5T LOCO + 40 BOXN 
Type - 6	2(25T)+55 BOXN	624.15	5800	6. FREIGHT TRAIN TWO 25T LOCO + 55 BOXN 

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Table 1 (b) Train Formation details in traffic Model for 25t. Loading–2008

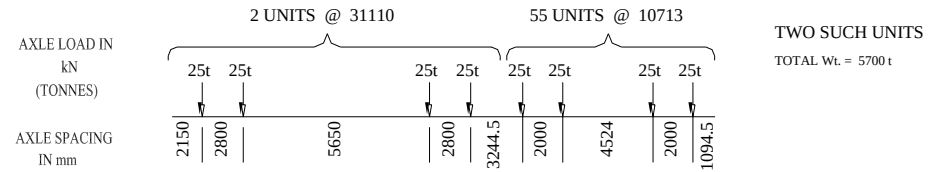
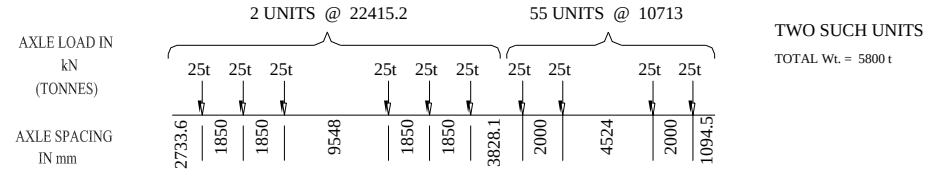
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type - 7	2E(2+55 BOXN)	1236.33	11400	7. FREIGHT TRAIN 2 (TWO ELECTRIC LOCO + 55 BOXN) 
Type - 8	2D(2+55 BOXN)	1262.62	11600	8. FREIGHT TRAIN 2 (TWO DIESEL LOCO + 55 BOXN) 

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Table 1 (b) Train Formation details in traffic Model for 25t. Loading–2008

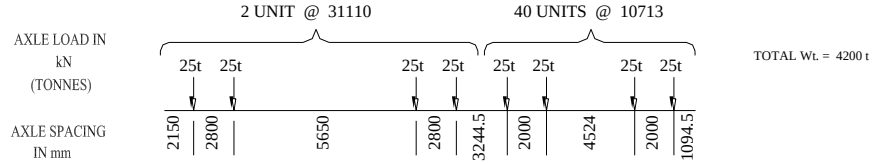
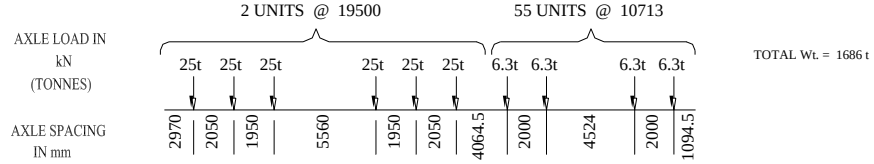
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type - 9	BO-BO+40 BOXN	456.375	4200	9. FREIGHT TRAIN TWO BO-BO + 40 BOXN 
Type - 10	2(25T)+55 BOXN	624.15	1686	10. FREIGHT EMPTY TRAIN TWO 25T LOCO + 55 BOXN 

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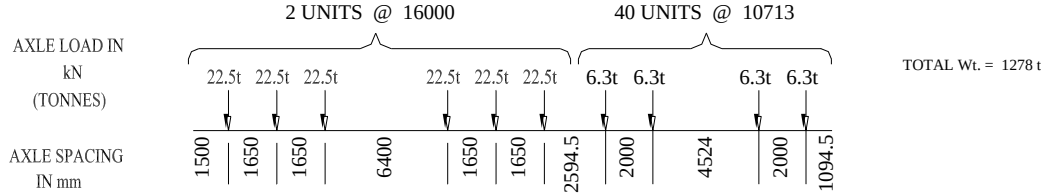
Table 1 (b) Train Formation details in traffic Model for 25t. Loading–2008

Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type - 11	2(22.5T)+40 BOXN	457.925	1278	11. FREIGHT EMPTY TRAIN TWO 22.5T LOCO + 40 BOXN 

Appendix G-I

Table 2 (a) – Traffic Models 32.5t (DFC) Loading

Type of Train	Train for-mation No.	Train Composition	Total length of Train (m)	Weight per Train (t)	GMT per train	Class of Traffic					
						Heavy (150 GMT)		Medium (100 GMT)		Light (50 GMT)	
						No. of Trains per day	GMT	No. of Trains per day	GMT	No. of Trains per day	GMT
Freight trains loaded (Gondola Type Wagon)	1	Two 6 axle loco as proposed + 40	473.35	5500	2.07	3	6.12	2	4.08	4	8.16
	2	2(2WDG2 Type +55)	1256.43	14900	5.44	4	21.988	2	10.994	1	5.497
	3	One 8-axle loco + 40	459.62	5400	1.97	15	29.895	11	21.923	2	3.986
	4	3WDG2 Type+75	861.975	10200	3.72	6	22.536	4	15.024	2	7.512
	5	3WAG6C +75	865.161	10155	3.7	18	66.474	12	44.316	6	22.158
Freight trains empty	6	2WDG2 Type+40	467.52	1332	0.486	3	1.524	2	1.16	3	1.524
	7	2MBG Type loco +55	621.215	1689	0.616	2	1.232	4	2.464	2	1.232

Table 2 (b) Train Formation details in traffic Model for 32.5t DFC Loading

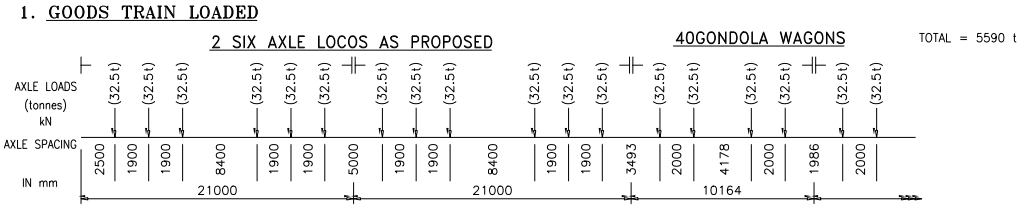
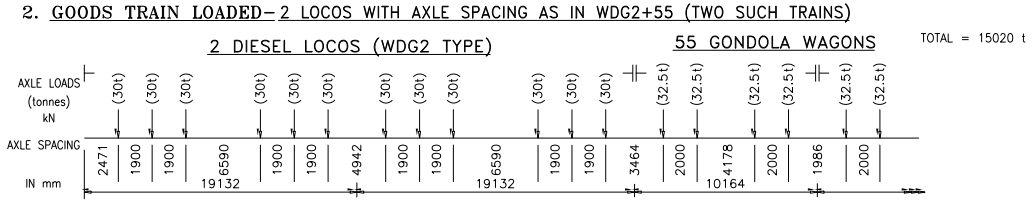
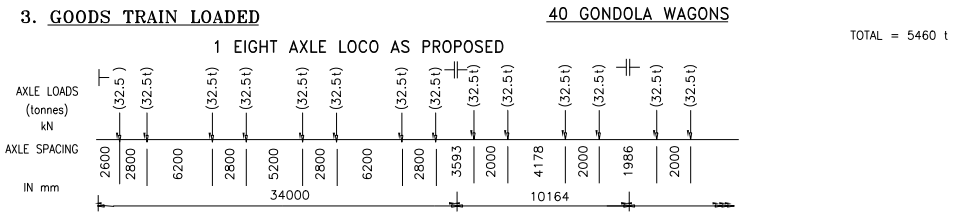
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type – 1	Two 6 Axle Loco as Proposed+40	445.067	5590	
Type – 3	2(2WDG2 Type+55)	1189.628	15020	
Type – 3	One 8–Axle Loco+40	436.967	5460	

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Table 2 :(b) Train Formation details in traffic Model for 32.5t DFC Loading

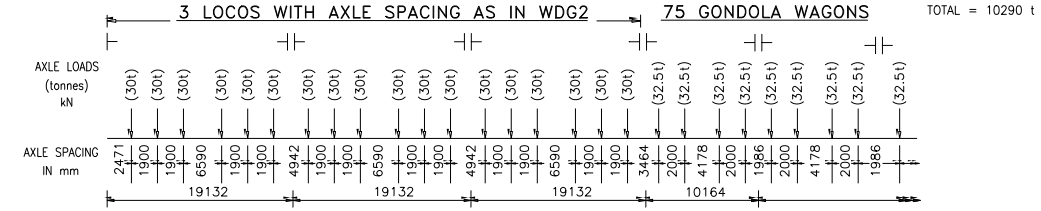
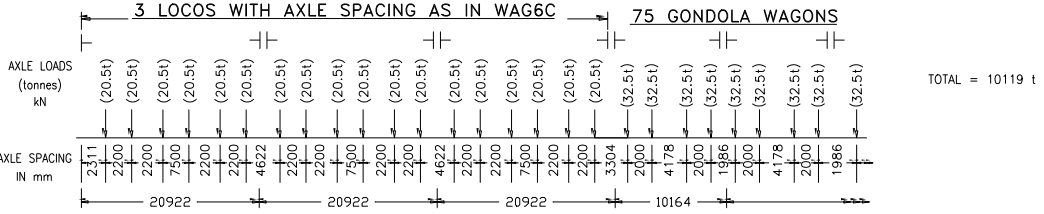
Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type – 4	3WDG2 Type+75	816.232	10290	4. GOODS TRAIN LOADED 
Type – 5	3WAG6C +75	821.762	10119	5. GOODS TRAIN LOADED 

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Table 2 (b) Train Formation details in traffic Model for 32.5t DFC Loading

Train type	Composition	Train Length Ist. to Last axle (m)	Total Weight (t)	Diagram
Type – 6	2WDG2 Type+40	441.360	1392	6. GOODS TRAIN EMPTY 2 LOCOS WITH AXLE SPACING AS IN WDG2 — 40 GONDOLA WAGONS AXLE LOADS (tonnes) kN: (30t) (30t) (30t) (30t) (30t) (30t) (30t) (30t) (30t) (30t) (30t) (30t) (6.45t) (6.45t) (6.45t) (6.45t) (6.45t) (6.45t) AXLE SPACING IN mm: 2471 1900 1900 6590 1900 1900 4942 1900 1900 6590 1900 1900 3464 2000 4178 2000 1986 2000 19132 19132 10164 TOTAL = 1392 t
Type – 7	2MBG Type LOCO+55	588.527	1689	7. GOODS TRAIN EMPTY 2 LOCOS WITH AXLE SPACING AS IN MBG — 55 GONDOLA WAGONS AXLE LOADS (tonnes) kN: (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (22.5t) (6.45t) (6.45t) (6.45t) (6.45t) (6.45t) (6.45t) (6.45t) (6.45t) AXLE SPACING IN mm: 1500 1650 1650 6400 1650 1650 3000 1650 1650 6400 1650 1650 2493 2000 4178 2000 1986 2000 4178 2000 1986 16000 16000 10164 TOTAL = 1689 t

NOTE :-

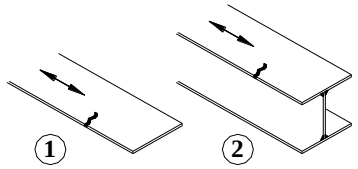
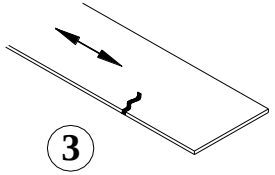
1. BRAKING FORCES

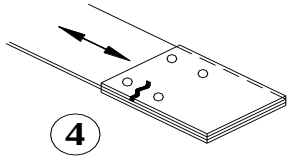
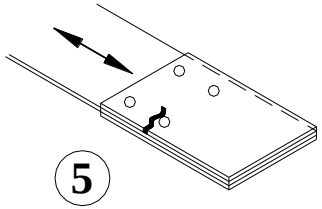
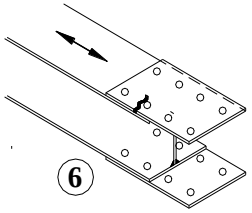
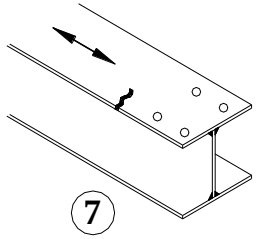
(a) TRACTIVE EFFORT ..... FOR TWO 8 AXLE LOCO 220 t @ 110 t PER LOCOS.
(b) TRACTIVE EFFORT ..... FOR TWO 6 AXLE LOCO 180 t @ 90 t PER LOCOS.
(c) TRACTIVE EFFORT ..... FOR THREE STANDARD LOCOS 180 t @ 60 t PER LOCOS.
(d) BRAKING FORCE OF TRAIN LOAD ..... 13.4% OF TRAIN LOAD.
(e) BRAKING FORCE OF LOCO ..... 25 % OF WEIGHT OF LOCO.

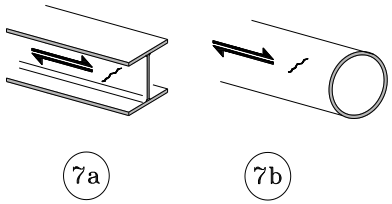
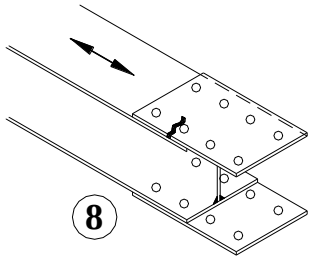
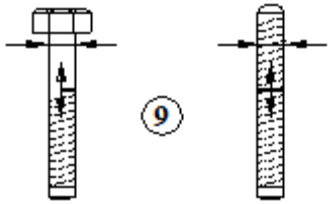
2. TRANSVERSE HORIZONTAL FORCE.

RAKING FORCE @ 1400 Kg/m

Table G-II.1 Non-welded details

Detail category	Constructional Detail	Description	Requirements
160	NOTE: The fatigue strength curve associated with category 160 is the highest. No detail can reach a better fatigue strength at any number of cycles.  	Rolled and extruded products: 1) Plates and flats 2) Rolled sections	Details 1) to 2) Sharp edges, surface and rolling flaws to be improved by grinding until removed and smooth transition achieved.
125	 	3) <u>Sheared or gas cut plates:</u> Machine gas cut or sheared material subsequently dressed to remove all edge discontinuities.	3) All visible signs of edge discontinuities to be removed. The cut areas are to be machined or ground and all burrs to be removed. Any machinery scratches for example from grinding operations can only be parallel to the stresses. Detail 3 - Re-entrant corners to be improved by grinding (slope < ¼) or evaluated using the appropriate stress concentration factors. - No repair by weld refill.

For detail 1-3 made of weathering steel use the next lower category			
112		4) Double covered symmetrical joint with preloaded high strength bolts.	4) $\Delta\sigma$ to be calculated on the gross cross-section.
90		5) Double covered joint with fitted or non preloaded bolts.	5) $\Delta\sigma$ to be calculated on the net cross section.
		6) One sided connection with preloaded H.S.B.	6) $\Delta\sigma$ to be calculated on the gross cross-section.
		7) Structural element with holes subject to bending and axial forces.	7) $\Delta\sigma$ to be calculated on the net cross- section.

100 M=5	 	7a) & 7b) Rolled and extruded products, as in details 1) and 2) above	7a) & 7b) $\Delta\tau$ calculated from: $\tau = \frac{V\,S(t)}{I\,t}$
80	 	8) One sided connection with fitted bolts or rivets.	8) $\Delta\sigma$ to be calculated on the net cross-section.
50	 	9) Bolts and rods with rolled or cut thread in tension. For large diameters (anchor bolts) the size effect has to be taken into account with ks Size effect for ϕ>30mm $k_s=(30/\phi)^{0.25}$. Where, ϕ is the nominal diameter of the bolt or rod.	9) $\Delta\sigma$ to be calculated using the tensile stress area of the bolt. Bending and tension resulting from prying effects and bending stresses from other sources must be taken into account. For preloaded bolts, the reduction of the stress range may be taken into account.

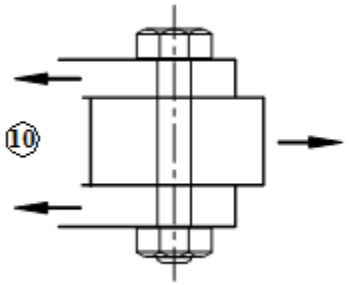
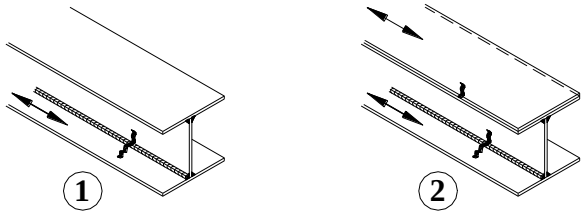
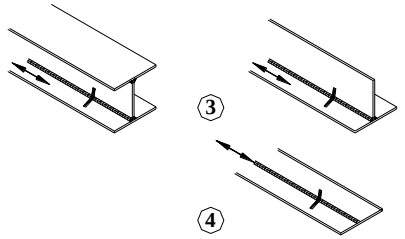
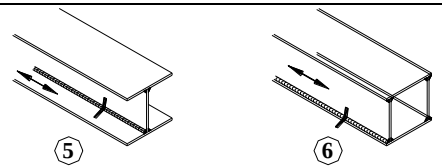
100 m = 5		10) Rivets or Bolts in single or double shear. Thread not in the shear plane - Fitted bolts - Normal bolts without load reversal (bolts of property class 6.6, 8.8 or 10.9)	10) $\Delta\tau$ calculated on the shank area of the bolt.

Table G-II. 2 Welded built-up sections

Detail category	Constructional Detail	Description	Requirements
125		Continuous longitudinal welds: 1) Automatic butt welds carried out from both sides. 2) Automatic fillet welds Cover plate ends to be checked using detail 5) or 6) Table G-II.5.	Details 1) to 2) No stop/start position is permitted except when the repair is performed and inspection is carried out to verify the proper execution of the repair.
112		3) Automatic fillet or butt weld carried out from both sides but containing stop/start positions. 4) Automatic butt welds made from one side only, with a continuous backing bar, but without stop/start positions.	4) When this detail contains stop/start positions category 100 to be used.
100		5) Manual fillet or butt weld. 6) Manual or automatic butt weld carried out from one side only, particularly for box girders.	6) A very good fit between the flange and web plates is essential. The web edge to be prepared such that the root face is adequate for the achievement of regular root penetration without break-out.

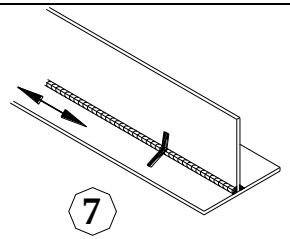
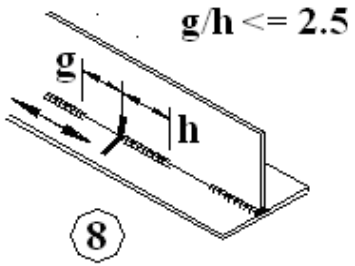
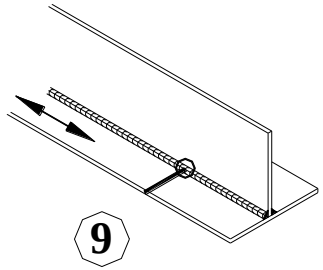
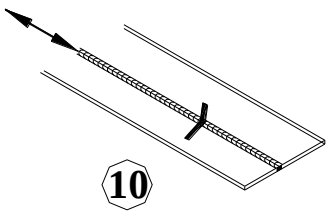
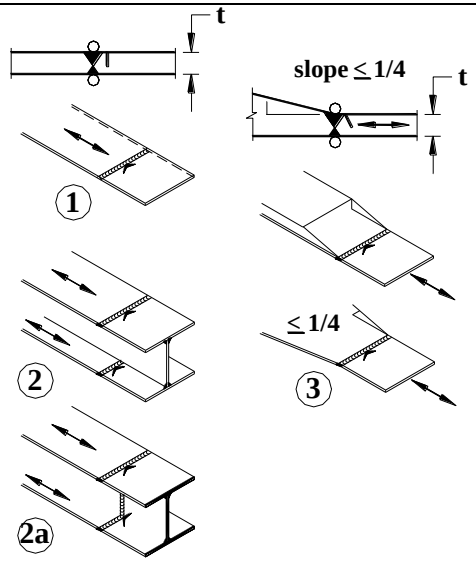
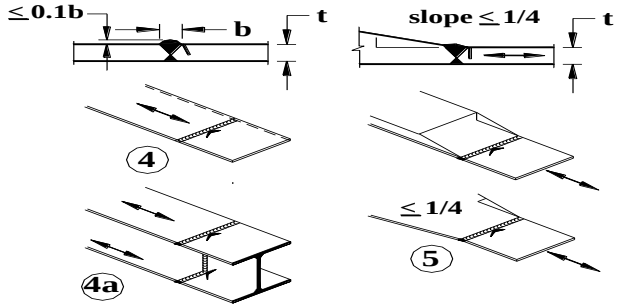
100		7) Repaired automatic or manual fillet or butt welds for categories 1-6 above.	7) Improvement by grinding performed by specialist to remove all visible signs and adequate verification can restore the original category.
80		8) Intermittent longitudinal fillet welds.	8) $\Delta\sigma$ based on normal stress in flange.
71		9) Longitudinal butt weld, fillet weld or intermittent weld with cope holes, cope holes not higher than 60mm.	9) $\Delta\sigma$ based on normal stress in flange.
125		10) Longitudinal butt welds, both sides ground flush parallel to load direction, 100% NDT.	
112		10) No grinding and no start/stop.	
90		10) With start/stop positions.	

Table G-II.3 Transverse butt welds

Detail category	Constructional Detail		Description	Requirements
112	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		Without backing bar: 1) Transverse splice in plate and flats 2) Flange and web splices in plate girders before assembly. 2a) Full cross-section butt welds of rolled sections without cope holes. 3) Transverse splices in plates or flats tapered in width or in thickness, with a slope ≤ ¼.	Details 1, 2 and 3: - All welds ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edge to be ground flush in direction of stress. - Welded from both sides: checked by NDT Detail 2 a) Rolled sections with the same dimensions without tolerance differences or cut and rewelded.
90	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		4) Transverse splices in plates or flats. 4a) Full cross-section butt welds of rolled sections without cope holes. 5) Transverse splices in plates or flats tapered in width or in thickness with a slope < ¼. Translation of welds to be machined notch free.	- The height of the weld convexity to be not greater than 10% of the weld width, with smooth transition to the plate surface. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT.

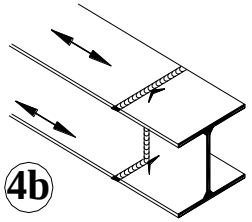
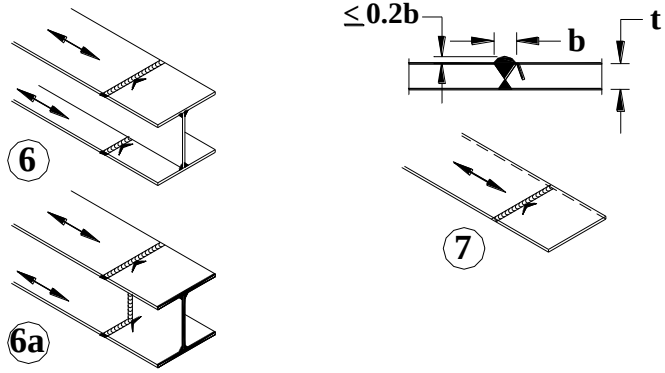
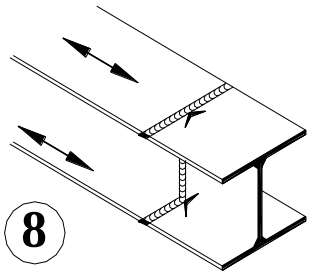
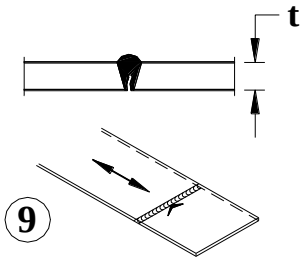
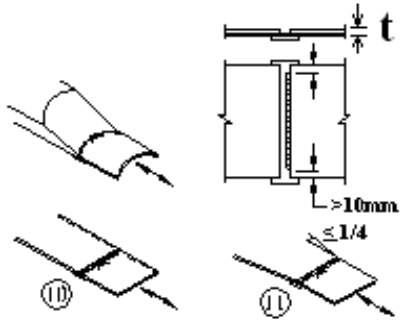
				Details 4 and 5 Welds made in flat position.
90	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		4b)Full cross-section butt welds of rolled sections with cope holes.	- All weld ground flush to plate surface parallel to direction of the arrow. - Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides; checked by NDT. - Rolled sections with the same dimensions without tolerance differences.
80	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		6) Transverse splices in welded plate girders without cope hole. 6a) Full cross-section butt welds of rolled sections with cope holes. 7) Transverse splice in plates, flats, rolled sections or plate girders.	- The height of the weld convexity to be not greater than 20% of the weld width, with smooth transition to the plate surface. - Weld not ground flush. - Weld run-on and run-off pieces to be used and subsequently removed, plate edge to be ground flush in direction of stress. - Welded from both sides; checked by NDT. Detail 6a: The height of the weld convexity to be not greater than 10% of the weld width, with smooth transition to the

				plate surface.
63			8) Full cross-section butt welds of rolled sections without cope hole.	- Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. - Welded from both sides.
36			9) Butt welds made from one side only	12) Without backing strip.
71	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		9) Butt welds made from one side only when full penetration checked by appropriate NDT.	
71	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		With backing strip: 10) Transverse splice. 11) Transverse butt weld tapered in width or thickness with a slope < ¼. Also valid for curved plates.	Details 10) and 11): Fillet welds attaching the backing strip to terminate > 10mm from the edges of the stressed plate.

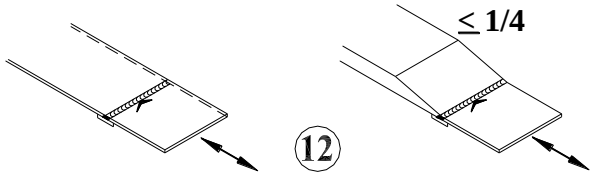
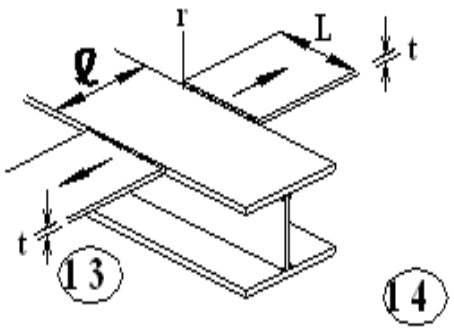
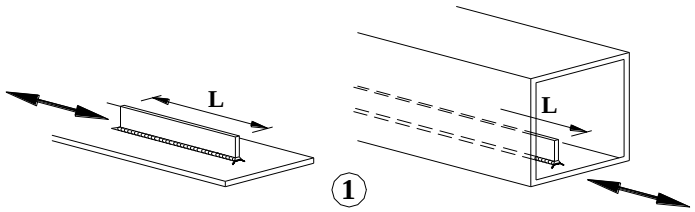
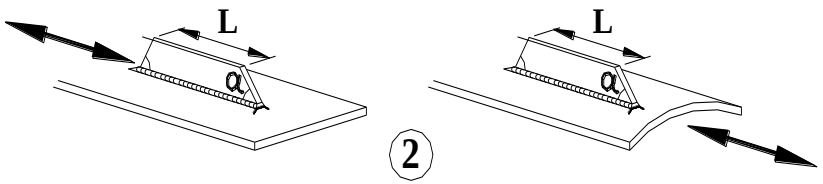
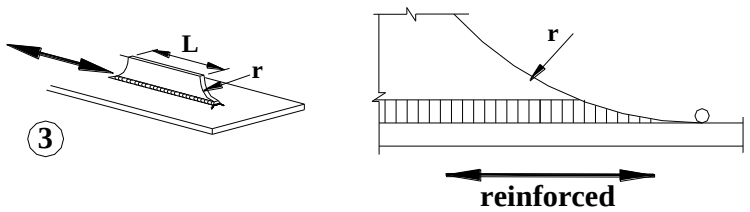
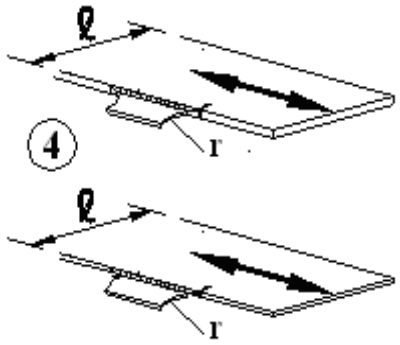
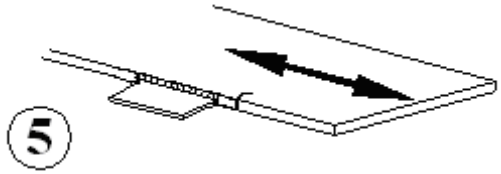
50	Size effect for t>25mm: $k_s=(25/t)^{0.2}$		12) Transverse butt weld on a permanent backing strip tapered in width or thickness with a slope < ¼. Also valid for curved plates.	12) Where backing strip fillet welds end < 10 mm from the plate edge, or if a good fit cannot be guaranteed.
As detail 1 in Table G-II.5			13) Transverse butt weld at crossing flanges.	Details 13) and 14) The fatigue strength in the perpendicular direction has to be checked with Table G-II.4 detail 4 or detail 5.
As detail 4 in Table G-II.4			14) With transition radius according to Table G-II.4, detail 4.	

Table G-II. 4 Welded attachments and stiffeners

Detail category	Constructional Detail		Description	Requirements
80	L≤50mm		Longitudinal attachments: 1) The detail category varies according to the length of the attachment L.	The thickness of the attachment must be less than its height. If not see Table G-II.5, details 5 or 6.
71	50<L≤80 mm			
63	80<L≤100mm			
56	L>100mm			
71	L>100mm $\alpha<45^{o}$		2) Longitudinal attachments to plate or tube.	

80	r>150mm		3) Longitudinal fillet welded gusset with radius transition to plate or tube; end of fillet weld reinforced (full penetration); length of reinforced weld >r.	<u>Detail 3) and 4)</u> Smooth transition radius r formed by initially machining or gas cutting the gusset plate before welding, then grinding subsequently the weld area parallel to the direction of the arrow so that the transverse weld toe is fully removed.
90	$\frac{r}{\ell} \geq \frac{1}{3}$ r>150mm		4) Gusset plate, welded to the edge of a plate or beam flange.	
71	$\frac{1}{6} \leq \frac{r}{\ell} \leq \frac{1}{3}$			
50	$\frac{r}{\ell} < \frac{1}{6}$			
40			5) As welded, no radius transition.	

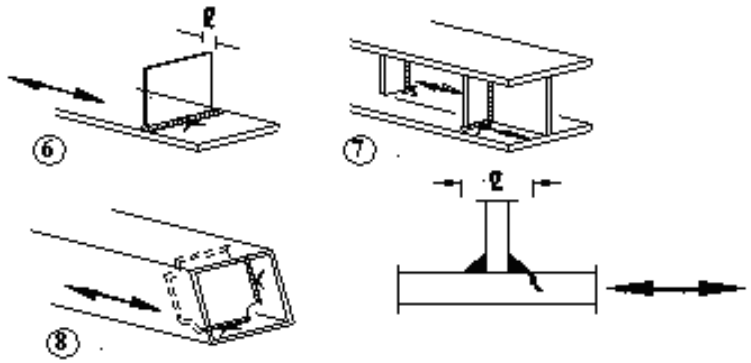
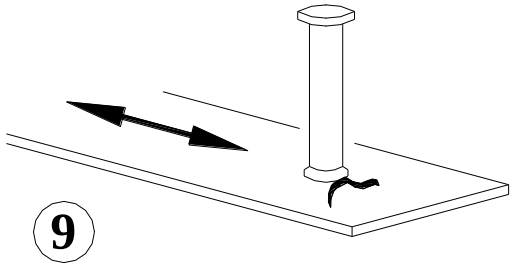
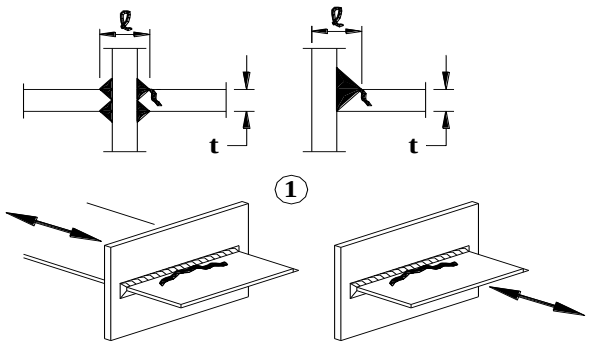
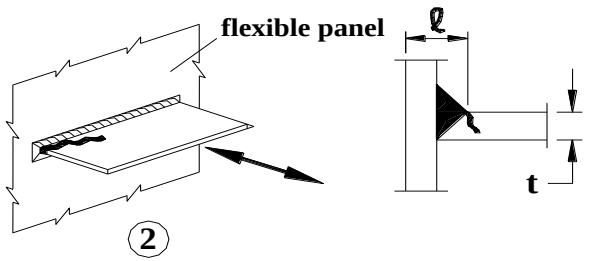
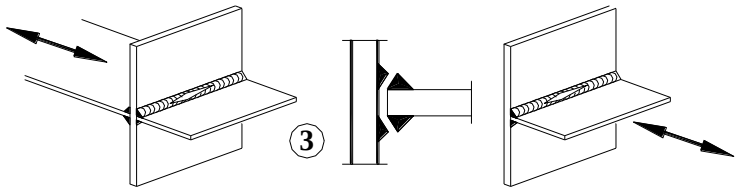
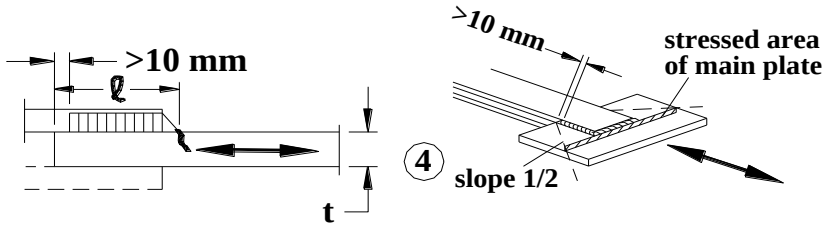
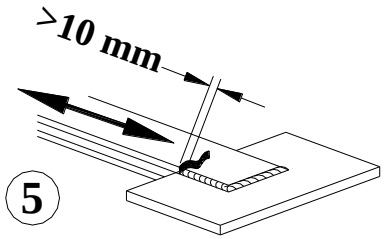
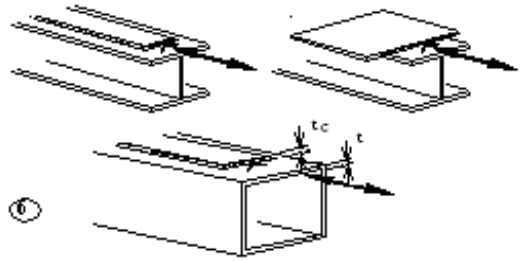
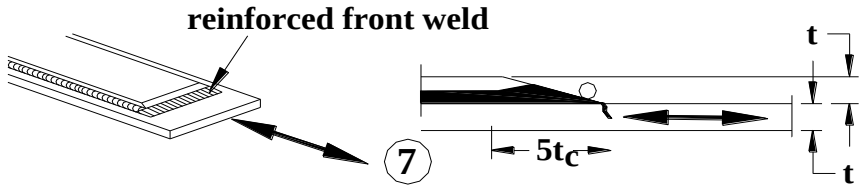
80	$\ell \leq 50$mm		**Transverse attachments** 6) Welded to plate. 7) Vertical stiffeners welded to a beam or plate girder. 8) Diaphragm of box girders welded to the flange or the web. Not possible for hollow sections. The values are also valid for ring stiffeners.	<u>Details 6) and 7):</u> Ends of welds to be carefully ground to remove any undercut that may be present. 7) $\Delta\sigma$ to be calculated using principal stresses if the stiffener terminates in the web.
71	$50 < \ell \leq 80$ mm			
80			9) The effect of welded shear studs on base material.	$\Delta\tau$ to be calculated on the nominal cross section of the stud.

Table G-II. 5 Load carrying welded joint

Detail category	Constructional Detail			Description	Requirements
80	$\ell \leq 50$mm	all t		Cruciform and Tee joints 1) Toe failure in full penetration butt welds and all partial penetration joints.	1) Inspected and found free from discontinuities and misalignments outside the recommended tolerances. 2) For computing $\Delta\sigma$, use modified nominal stress. 3) In partial penetration joints two fatigue assessments are required. Firstly, root cracking evaluated according to stresses defined in section 7.3, using category 36* for $\Delta\sigma_w$ and category 80 for $\Delta\tau_w$. Secondly, toe cracking is evaluated by determining $\Delta\sigma$ in the load carrying plate.
71	$50 < \ell \leq 80$	all t			
63	$80 < \ell \leq 100$	all t			
56	$100 < \ell \leq 120$	all t			
56	$\ell > 120$	$t \leq 20$			
50	$120 < \ell \leq 200$ $\ell > 200$	$t > 20$ $20 < t \leq 30$			
45	$200 < \ell \leq 300$ $\ell > 300$	$t > 30$ $30 < t \leq 50$			
40	$\ell > 300$	$t > 50$			

As detail 1 in Table G-II.5		2) Toe failure from edge of attachment to plate, with stress peaks at weld end due to local plate deformations.	<u>Details 1) to 3)</u> The misalignment of the load-carrying plates should not exceed 15% of the thickness of the intermediate plate.
36*		3) Root failure in partial penetration Tee-built joint or fillet welded joint and effective full penetration in Tee-butt joint.	
As detail 1 in Table G-II.5		**<u>Overlapped welded joints</u>** 4) Fillet welded lap joint.	4) $\Delta\sigma$ in the main plate to be calculated on the basis of area shown in the sketch.

<table>
<tr><td>45*</td><td colspan="3"> 
</td><td><u>Overlapped</u>
5) Fillet welded lap joint.</td><td>5) $\Delta\sigma$ to be calculated in the overlapping plates.

<u>Details 4) and 5)</u>
- Welded terminations more than 10 mm from plate edge.
- Shear cracking in the weld should be checked using detail 8).</td></tr>
<tr><td></td><td>t_c<t</td><td>$t_c \geq t$</td><td rowspan="6">  </td><td rowspan="6"><u>Cover plates in beams and plate girders</u>
6) End zones of single or multiple welded cover plates, with or without frontal weld.</td><td rowspan="6">6) If the cover plate is wider than the flange, a frontal weld is needed. This weld should be carefully ground to remove undercut.
The minimum length of the cover plate is 300mm. For shorter attachments see detail 1).</td></tr>
<tr><td>56*</td><td>$t \leq 20$</td><td>-</td></tr>
<tr><td>50</td><td>$20 < t \leq 30$</td><td>$t \leq 20$</td></tr>
<tr><td>45</td><td>$30 < t \leq 50$</td><td>$20 < t \leq 30$</td></tr>
<tr><td>40</td><td>t>50</td><td>$30 < t \leq 50$</td></tr>
<tr><td>36</td><td>-</td><td>t>50</td></tr>
<tr><td>56</td><td colspan="3"> 
</td><td>7) Cover plates in beams and plate girders.
5 t_c is the minimum length of the reinforcement weld.</td><td>7) Front weld ground flush. In addition, if tc>20mm, front of plate at the end ground with a slope < ¼.</td></tr>
</table>

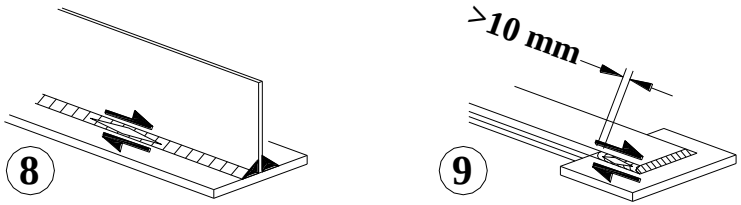
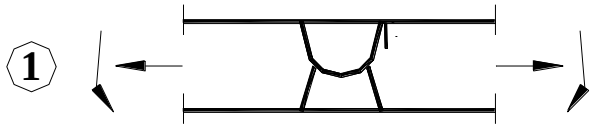
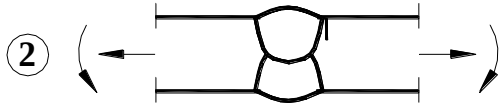
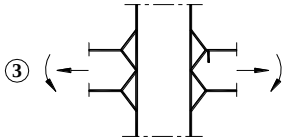
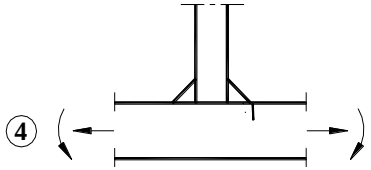
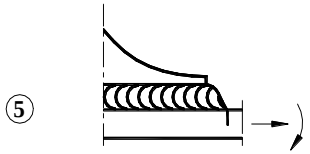
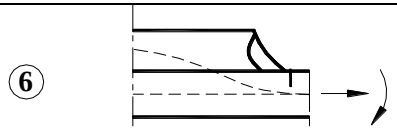
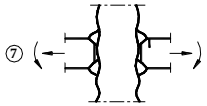
80 m=5		8) Continuous fillet welds transmitting a shear flow, such as web to flange welds in plate girders. 9) Fillet welded lap joint.	8) $\Delta\tau$ to be calculated from the weld throat area. 9) $\Delta\tau$ to be calculated from the weld throat area considering the total length of the weld. Weld terminations more than 10mm from the plate edge.

Table G-II. 6 : Fatigue resistance against geometric stress for cracks initiating from toes of welds.

Detail category	Constructional details	Description	Requirements
112		1) Full penetration butt joint.	1) – All welds ground flush to plate surface parallel to direction of the arrow. – Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. – Welded from both sides, checked by NDT. – For misalignment see note 1 below.
100		2) Full penetration butt joint.	2) – Weld not ground flush – Weld run-on and run-off pieces to be used and subsequently removed, plate edges to be ground flush in direction of stress. – Welded from both sides. – For misalignment see note 1 below
100		3) Cruciform joint with full penetration K-butt welds.	3) – Weld toe angle < 60^{o} – For misalignment see note 1 below.

100		4) Non load-carrying fillet welds.	4) Weld toe angle < 60° (see note 2 below)
100		5) Bracket ends, ends of longitudinal stiffeners.	5) Weld toe angle < 60° (see note 2 below)
100		6) Cover plate end and similar joints.	6) Weld toe angle < 60° (see note 2 below)
90		7) Cruciform joints with load-carrying fillet welds.	7) Weld toe angle < 60° (see notes 1 and 2 below)

NOTE 1. Table G-II.6 does not cover effects of misalignment. They have to be considered explicitly in determination of stress.

NOTE 2. Table G-II.6 does not cover fatigue initiation from the root followed by propagation through the throat.

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Appendix G-III

Cycle counting Methods

A.1 The application of a loading event, in general, produces complex stress histories that rarely have constant amplitude at most of the structural details. In order to assess the fatigue damage at these details due to the complex stress history, the load history has to be reduced to a sequence of blocks of constant amplitude. The process of identification of the constant amplitude stress ranges and the associated number of cycles present in the stress history is known as 'cycle counting'. The damage accumulated due to these constant amplitude blocks can be calculated individually and summed using Palmgren-Miner's rule to calculate the total accumulated damage of the structure. The two most commonly employed methods for cycle counting are the 'Reservoir method' and the 'Rainflow method', both yielding identical results if the rainflow analysis is initiated from the highest peak in the stress history. The reservoir count is employed for short stress histories while the rainflow counting is employed for longer and more complex stress histories.

A.2 Cycle counting by the reservoir method

A.2.1. The graphical plot of the stress history, in this method, is imagined as a cross section of a reservoir filled with water. The water is drained from each of the lowest points successively till the entire reservoir is drained. Each drainage operation represents a cycle of stress range equal in magnitude to the height of the water drained in that particular operation.

A.2.2. The procedure for cycle count by the reservoir method is as follows :-

A.2.2.1 It is assumed that the stress history has been derived taking into consideration such provisions as are applicable with regard to loads, structural details, structural material, methods of analysis and any other modifications necessary.

A.2.2.2 The peaks and valleys are identified in the original stress history (figure A.1) and joined by straight line segments, if necessary. This modified stress history will be used for the reservoir count as shown in figure A.2**.**

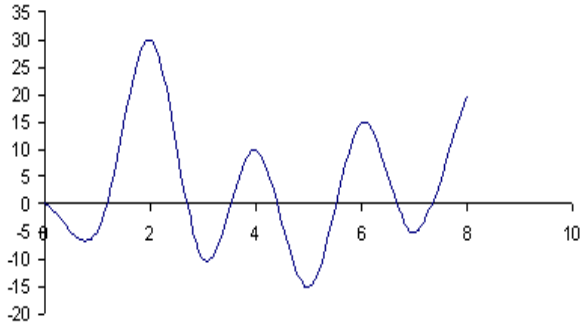


Figure A.1 Typical segment of a stress history

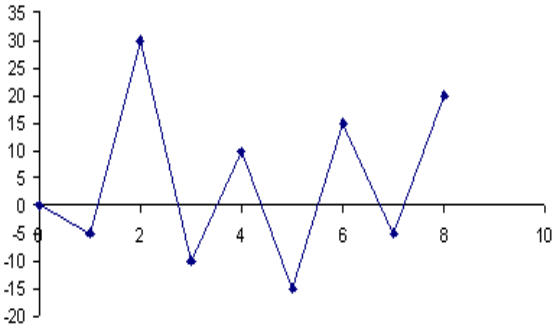


Figure A.2 Stress history segment showing peaks and valleys

A.2.2.3 A copy of the stress history is appended to the original (figure A.3) and the highest point (A) in the original segment and its counterpart (B) in the appended segment are marked and joined by a straight horizontal line. The portion of the stress history so enclosed will be used to represent the reservoir. In case there are two or more equal peaks in the original segment of the stress history then the first such peak will be considered along with its counterpart from the appended segment**.**

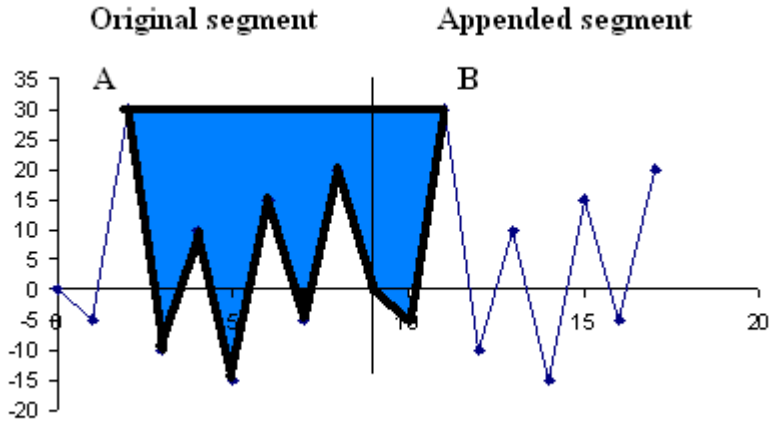


Figure A.3 Original and appended stress history

A.2.2.4 The reservoir is drained successively from the lowest points (E, F, D and C taken in order as shown in figure A.4) which retain water till the entire reservoir is emptied. Each drainage operation corresponds to a cycle of stress range equal in magnitude to the height of the water drained in that particular operation i.e. one cycle of stress range σ_A - σ_E when drainage is from trough E**.**

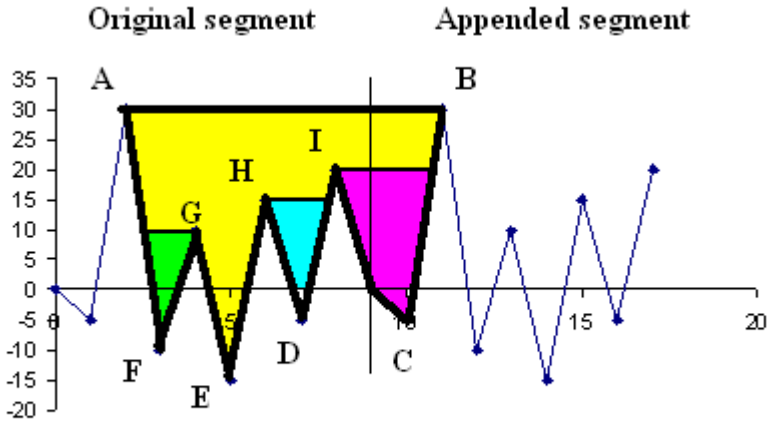


Figure A.4 Drainage sequence from troughs of the reservoir

A.2.2.5 The stress ranges and their associated number of cycles are sorted according to the magnitude of the stress ranges for further processing using the Palmgren-Miner criteria**.**

A.2.3. Consider the following example :-

A stress history consists of the following stress variation

Time	1	2	3	4	5	6	7	8	9	10	11	12	13	14
Stress	28	-18	8	2	22	-6	20	8	20	-18	22	-4	26	12
	A, O	B	C	D	E	F	G	H	I	J	K	L	M	N

In order to conduct a reservoir count appending the first point, as it is the highest, will suffice for the definition of the reservoir. A schematic diagram indicating the extent of drainage from each trough is as shown in figure A.5. The points in the stress history have been labeled from A to O for easy identification.

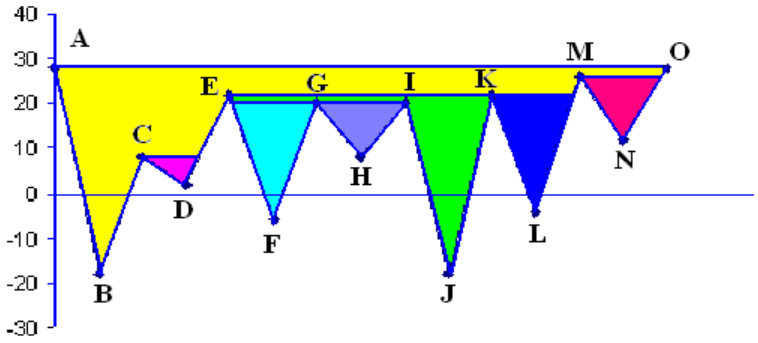


Figure A.5 Reservoir section showing drainage sequence

The results from the reservoir count can be tabulated as follows:-

Drainage from Trough	Highest water level at	Stress range
B	A	46
J	K	40
F	G	26
L	K	26
D	C	6
H	G	12
N	M	14

The above may be arranged in order for further processing.

A.3 Cycle counting by the rainflow method

A.3.1 The rainflow counting technique is based on the visualization of flow of rain over a sequence of pagoda roofs and essentially counts half cycles. In order to effect the visualization the stress history is rotated such that the time axis is vertical with the origin located towards the top. Rainflow is assumed to begin from a peak or a trough and the distance it travels determines the magnitude of the stress range, each flow contributing a half cycle.

A.3.2 The procedure for rainflow count is as follows :-

a) It is assumed that the stress history conforms to 2.2(a) and is modified in accordance with 2.2(b) so that the stress history is reduced to a sequence of peaks and troughs.

b) The stress history may be modified in accordance with 2.2(c) so that it begins and ends with the highest peak (or the deepest trough).

c) The stress history is rotated through 90^{o} such that the origin of the time axis is located towards the top (figure A.6).

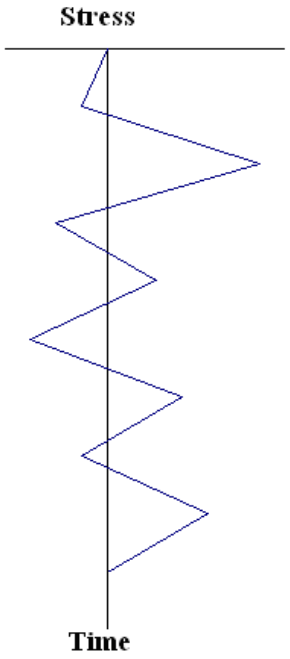


Figure A.6 Rotation of axes for rainflow counting

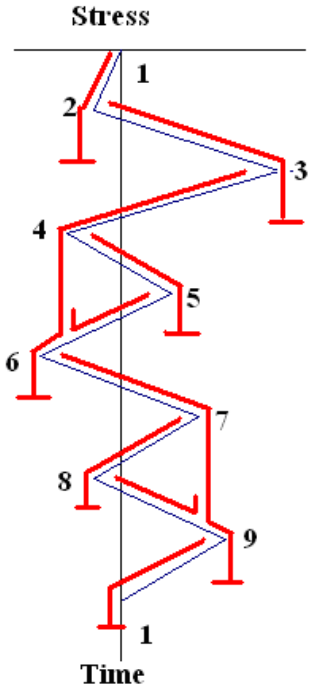


Figure A.7 Rainflow patterns

d) A drop begins to flow (figure A.7) left from a peak (1-2) or right from a trough(1-3) onto subsequent roofs (3-4-6) unless the surface receiving the drop is formed by a peak which is more positive than the origin of the drop (1-2) for a left flow, or, a trough that is more negative for a right flow(4-5).

e) The path of a drop cannot cross the path of a drop which has fallen from a higher roof (5-6).

f) A drop ceases to flow when it reaches the end of the stress history record (1-3).

g) The horizontal displacement of the drop from its origin to its final position measured in appropriate stress units represents a half cycle of the associated stress range.

A.3.3 Considering the same example as in 2.3 the rainflow patterns are as shown in figure A.8.

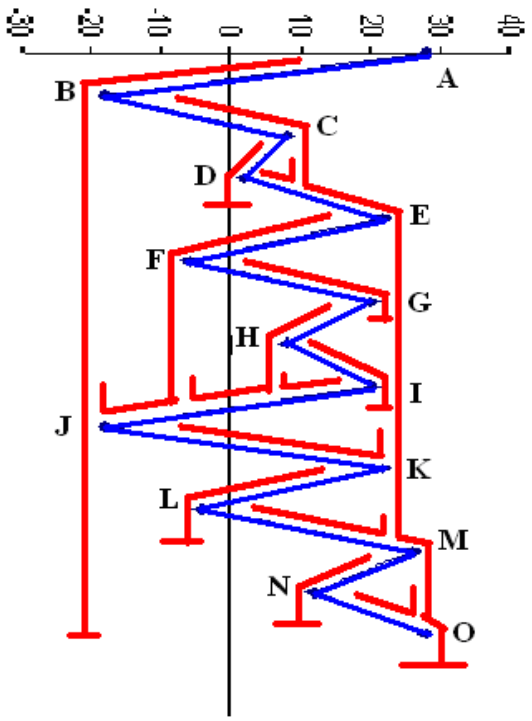


Figure A.8 Rainflow patterns

The results from the rainflow count can be tabulated as follows:-

Origin of flow	Termination of flow	Half cycle stress range
A	B	46
B	O	46
C	D	6
D	C	6
E	J	40
F	G	26
G	F	26
H	I	12
I	H	12
J	K	40
K	L	26
L	K	26
M	N	14
N	M	14

The half cycles in the above may be combined and subsequently arranged in order for further processing. It may be noted that the results of the rainflow and the reservoir counting are identical in this case.

A.3.4 Stress Histogram: If we divide stress range in units of 10, Stress histogram for the above cycles can be made as follows:-

Stress Range slab	**Mean Value of Stress Range slab**	**No of Cycles**
0-9.9	5	1
10-19.9	15	2
20-29.9	25	2
30-39.9	35	0
40-49.9	45	2

APPENDIX H

Distribution of Wheel Loads on Steel Troughing or Beams Spanning Transversely to the Track

When the running rails are supported directly on steel troughing or beams spanning transversely between the main girders, the pitch of the troughing or beams being, as it normally will be, less than half the axle spacing, the deflection of the rails, and the resulting stresses in the troughing or beams, may be calculated on the assumption that the rails have a uniform elastic support. The method of calculation is as follows:
The deflection δ of the rail seat due to a uniform load of 1 tonne per linear metre on each rail is first calculated, from the bending moment diagram and the moment of inertia of the troughing or beams supporting the rail. Then the "elastic modulus" of the track i.e. weight in kilograms per linear cm on each rail required to depress it one cm, represented by the symbol "U" is given by the formula:

$$U = \frac{1000}{100 \times \delta} = \frac{10}{\delta}$$

Now let I = the moment of inertia of each rail, about a horizontal axis, in cm^4 .

Y_0 = the depression of the rail in cm immediately below a load P tonnes.

And X_1 = the distance from the load to the point of contraflexure of the rail in cm.
Then

$$X_1 = 42.3 \sqrt[4]{I/U}$$

$$Y_o = \frac{9.3P}{\sqrt[4]{IU^3}} = \frac{393P}{UX_1}$$

The second of these formulae gives the deflection of the rail, and so the bending stress in the transverse troughing or beam, immediately under an axle. The effect of adjacent axles is calculated by means of the Master Diagram on page.129, which gives the relative depression due to an axle load at a distance X_1 cm.
In the above calculations, the load P is the wheel load of the locomotive with dynamic effect as per clause 2.4.1 (E) of IRS Bridge Rules.

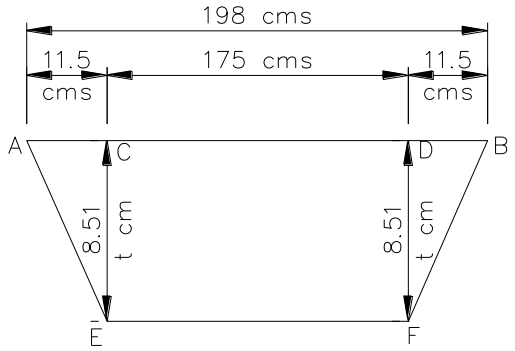
The following example will illustrate the method

Steel transverse sleepers or steel channel sleepers with a moment of inertia of 1558.8 $cm^4$and a section modulus of 207.8 $cm^{3.}$ at 74 cm centres, supported on main girders at 1.98m centres carry new 90R rails, having moment of inertia of 1600 cm^4. What is the stress in the sleepers under modified broad gauge loading 1987.

Assume E = 21100 kg/mm^2

Solution: The load applied to a sleeper by each rail due to a load of 1 tonne per metre on the rail is 0.74tonne.

Assuming that the loads are applied 175cm centres, the BM diagram for the sleeper will be as shown



Deflection at C below A

$$= \frac{\text{Moment of the area of the BM diagram about C}}{EI}$$

Now, reaction at A due to BM diagram loading

= ½ x 11.5 x 8.51 + 175/2 x 8.51

= 48.93 + 744.63

= 793.56 t cm^2

Moment at C

= 793.56x11.5 – 48.93 x 11.5/3

= 9125.94 – 187.57

= 8938.37 t cm^3

$$\text{Therefore, deflection at C} = \frac{8938.37}{2110 \times 1558.8}$$

= 0.002718 cm

Hence, $U = \frac{10}{0.002718} = 3679$

and X_1 = 42.3 $\sqrt[4]{1600/3679}$ = 34.35 cm

The dynamic effect = $\frac{7.32}{B + 5.49} = \frac{7.32}{1.98 + 5.49} = 0.98$

And the wheel load of MBG standard loco including Dynamic Effect

$$P = \frac{25}{2} \times 1.98$$

$= 24.75$ tonnes.

Therefore, Depression under one wheel,

$$Y_0 = \frac{393 \times P}{U \times X_1} = \frac{393 \times 24.75}{3679 \times 34.35}$$

= 0. 07697 cm

The relative value of the depression for a wheel load 205 cm away from the master diagram is -0.04 and that for a wheel 195 cm away is -0.04.

Ignoring the other wheels, which are too far away to have an appreciable effect, the total depression under one bogie of loco

$=0.07697(1-0.04-0.04)$

$=0.0708$

The corresponding bending moment in sleeper

$$= 8.51 \times \frac{.0708}{.002718}$$

= 221.67 t-cm

Therefore, bending stress in sleeper

$$= \frac{221.67}{207.8} = 1.067 t/cm^2 = 10.67 kg/mm^2$$

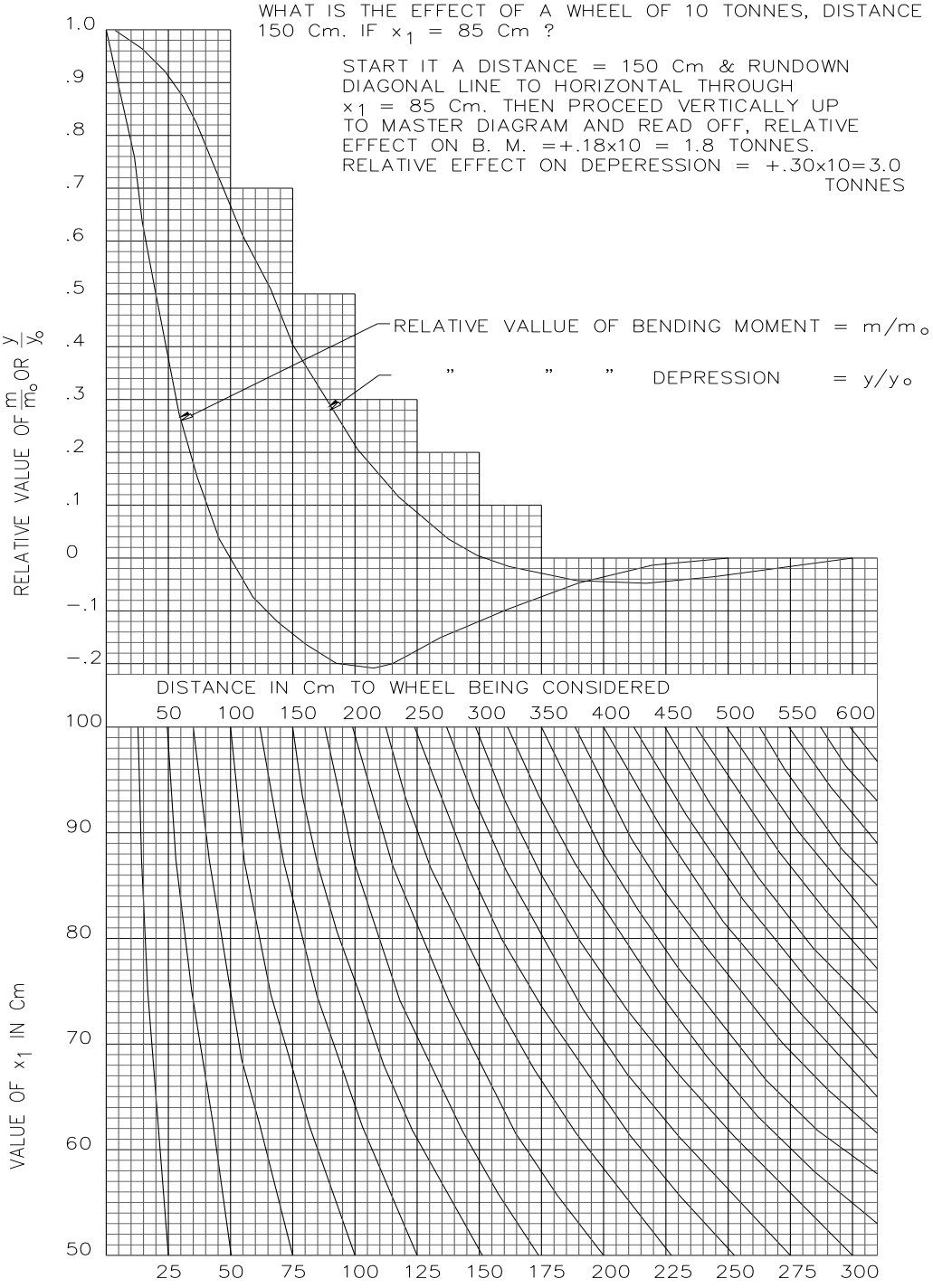
TRACK CALCULATIONS

MASTER DIAGRAM FOR BENDING MOMENT & DEPRESSION

EXAMPLE OF USE:–

WHAT IS THE EFFECT OF A WHEEL OF 10 TONNES, DISTANCE 150 Cm. IF x_1 = 85 Cm ?

START IT A DISTANCE = 150 Cm & RUNDOWN DIAGONAL LINE TO HORIZONTAL THROUGH x_1 = 85 Cm. THEN PROCEED VERTICALLY UP TO MASTER DIAGRAM AND READ OFF, RELATIVE EFFECT ON B. M. =+.18x10 = 1.8 TONNES. RELATIVE EFFECT ON DEPERESSION = +.30x10=3.0 TONNES



APPENDIX J
Recommendation for the design of combined Road-Rail Bridges

1. These recommendations relate to the design of bridge girders carrying both road and rail traffic. Bridges in which separate road and railway spans are carried on common piers are not dealt with. Relevant Road way Standard should be consulted wherever necessary.

2. **Type of combined bridges**: - Combined bridges may be classified, according to the relative positions of the road and railway as follow

	TYPE	USE
A.	Road and railway on same deck	Only for bridges carrying unimportant branch lines and roads with comparatively light traffic.
B.	Road and railway side by side, above or between main girders.	Not recommended on account of eccentric loading, but may be considered for short bridges, on which only a single traffic-lane road is required, and where road and railway are at nearly the same level.
C.	Cantilevered roads.	For short bridges in flat country, where either road or railway would have to be ramped up to take it over the other. For bridges of medium lengths, relative economy of type C and E should be examined.
D(1)	Railway above road and over top chords.	For bridges with spans up to about 76.2 m(250 ft) clear in hilly country, where track level would not have to be raised appreciably to give required clearance from high flood level.
D(2)	Railway above road and between girders.	As for D(1) but for longer spans. Type D(1) would be used for longer spans probably up to 91.40 m (300 ft) clear, in double track bridges to avoid increasing the spacing of main girders, provided rail level is suitable.
E(1)	Road above railway and over top chords.	For long bridges in flat country, with spans up to about 61.0 m to 91.40 m (200 ft to 300 ft) depending on (a) the total length of the bridge,(b) the number and gauge of the tracks ,and (c) the width of the roadway.
E(2)	Road above railway and between girder.	As for E(1), but for cases where it is more economical to accommodate the roadway between the main girders than to raise it sufficiently to enable it to project above the top chords.

Note: -

(1) The various limiting span lengths given above are based on judgment only and not on actual designs. Comparative designs for two or more types should be made in all border line cases to determine the most economical type.

(2) Type A bridges are very undesirable from the point of view of track maintenance, Railway operation and road traffic. New bridges of this type should, therefore, be constructed only in very exceptional circumstances.

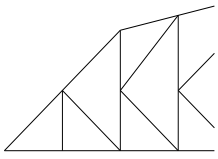
(3)`Type C combined bridges, having two single traffic-lane roads carried on cantilevers outside the main girders, have two great disadvantages,(a) fast traffic cannot pass slow traffic on the bridge, and (b) the eccentricity of loading, with only one traffic-lane loaded, requires a lot of extra metal in the main girders, especially when IRC class "AA" load is specified. Bridges of this type should, therefore, be used only for very short crossings.

3. Depth of Main girders and system of Triangulation

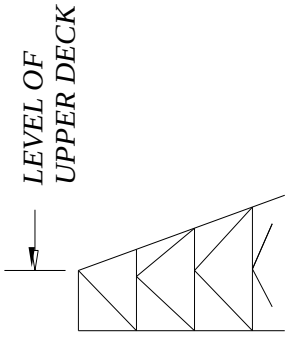
3.1 For spans of types A, B, C, D (2) and E(2) the economic depth of the main girders will probably be about one seventh of the span. A 'K' system of triangulation with polygonal top chords will generally be most suitable for bridges of these types with clear spans or 91.4m(300 ') or over, but for 106.7m (350') clear spans of types D(2) and E(2) fitted with lateral bracings between the top chords the possible variation in depth of the girders would not be large, and parallel chords will probably prove more economical.

3.2 For spans of types D (1) and E (1) parallel chords will be necessary, and where the road or railway has to climb to the level of the top chords, a reduction in the ratio of depth to length of span, or in the length of span itself, below the economic figures for bridges carrying railway loads only, may be found to give the minimum total cost of the bridge.

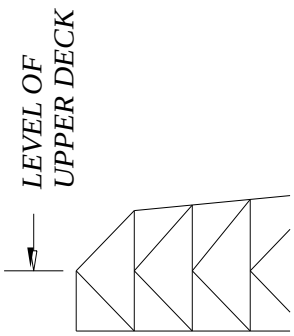
3.3 'K' trusses will generally be more economical than 'N' or warren truss for spans of 91.4m (300') or over, though this limit may be somewhat increased where trough-decking is used and the panel length increased. The arrangement of the members of 'K' trusses at the ends of the spans shown below will generally be the most suitable under the conditions shown against each.



For types A, B & C



For types D(2) & E(2) without lateral bracings between top chords.



For types D(2) & E(2) with lateral bracings between top chords.

4 Layout of approach roads

4.1 Types A-D. No special arrangements will have to be made in the case of types A, B and C except that level crossing gates will have to be provided at each end of a bridge of type A, while for a bridge of Type C one-half of the road will have to cross the railway at each end of the bridge by means of a level crossing and under-bridge or an over-bridge. In the case of types D and E the road will have to diverge from the railway at each end of the bridge, approach spans will be required for this purpose and the design of these must be such as to give reasonably good conditions for road traffic. The approach spans for bridges of types D will carry the railway and will, of course, be on the same alignment as the main spans. Their length will be governed by the condition that it must be possible to construct a roadway approach curve, clear of the approach span abutment, which can be traversed by road vehicles at the design speed.

4.2 Type E. in the case of Bridges of Type E the approach spans will carry the road way and their length will be controlled by the angle of divergence between the road and the railway and by the condition that the pier or tower supporting the end farther from the main span must clear the railway fixed structure diagram.

5 The design of the road approaches between the end of the main bridge and a point at which the distance between the road and railway is sufficient to accommodate and embankment shall be made by the railway, in consultation with the road authority.
The construction and maintenance of railway approach spans (type D bridges) and of piers or abutments carrying such spans will be the responsibility of the railway authority.

The design, construction and maintenance of roadway approach spans (type E bridges), which are over or partly over the railway, will be the responsibility of the railway authority. The design, construction and maintenance of other roadway approach spans, including pier and abutment, or of steel trestles or arch viaducts carrying in approach road may be undertaken by either the railway or the road authority, as mutually agreed in each case.

6. Handrailings or parapets

6.1 The clear distance from the lower rail to the top of the kerb shall not exceed 150mm (6 inches) unless the space if filled by vertical or inclined members, the clear distance between which is not more than 150mm (6 inches). The strength of the lower rail shall be at least as great as that of the top rail. The space between the lower rail and the top rail shall be filled by means of vertical, horizontal or inclined members, the clear distance between which shall be fixed with due regard to the safety of persons and animals using the structure.

6.2 In cases where a road vehicle mounting the kerb and breaking through the hand railing or parapet may endanger the stability of the bridge, the railway authority may require the height of the kerb, or the strength of the hand railing (or parapet) or both, to be suitably increased.