STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION: II

LOADS AND LOAD COMBINATIONS

(SEVENTH REVISION)

(Incorporating all amendments and errata published upto December, 2016)

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PERSONNEL OF THE BRIDGES SPECIFICATIONS AND STANDARDS COMMITTEE
(As on 16th December, 2016)

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2	(Das. S.N.)	Director General (Road Development) & Special Secretary, Ministry of Road Transport and Highways & Honorary Treasurer, Indian Roads Congress, New Delhi
3	Nahar, Sajjan Singh	Secretary General, Indian Roads Congress, New Delhi

STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

INTRODUCTION

The brief history of the Bridge Code given in the Introduction to Section I "General Features of Design" generally applies to Section II also. The draft of Section II for "Loads and Stresses", as discussed at Jaipur Session of the Indian Roads Congress in 1946, was considered further in a number of meetings of the Bridges Committee for finalisation. In the years 1957 and 1958, the work of finalising the draft was pushed on vigorously by the Bridges Committee.

In the Bridges Committee meeting held at Bombay in August 1958, all the comments received till then on the different clauses of this Section were disposed off finally and a drafting Committee consisting of S/Shri S.B. Joshi, K.K. Nambiar, K.F. Antia and S.K. Ghosh was appointed to work in conjunction with the officers of the Roads Wing of the Ministry for finalising this Section.

This Committee at its meeting held at New Delhi in September 1958 and later through correspondences finalized Section II of the Bridge Code, which was printed in 1958 and reprinted in 1962 and 1963.

The Second Revision of Section II of the IRC:6 Code (1964 edition) included all the amendments, additions and alterations made by the Bridges Specifications and Standards (BSS) Committee in their meetings held from time to time.

The Executive Committee of the Indian Roads Congress approved the publication of the Third Revision in metric units in 1966.

The Fourth Revision of Section II of the Code (2000 Edition) included all the amendments, additions and alterations made by the BSS Committee in their meetings held from time to time and was reprinted in 2002 with Amendment No.1, reprinted in 2004 with Amendment No. 2 and again reprinted in 2006 with Amendment Nos. 3, 4 and 5.

The Bridges Specifications and Standards Committee and the IRC Council at various meetings approved certain amendments viz. Amendment No. 6 of November 2006 relating to Sub- Clauses 218.2, 222.5, 207.4 and Appendix-2, Amendment No. 7 of February 2007 relating to Sub-Clauses of 213.7, Note 4 of Appendix-I and 218.3, Amendment No. 8 of January 2008 relating to Sub-Clauses 214.2(a), 214.5.1.1 and 214.5.2 and new Clause 212 on Wind load.

As approved by the BSS Committee and IRC Council in 2008, the Amendment No. 9 of May 2009 incorporating changes to Clauses 202.3, 208, 209.7 and 218.5 and Combination of Loads for limit state design of bridges has been introduced in Appendix-3, apart from the new Clause 222 on Seismic Force for design of bridges.

The Bridges Specifications and Standards Committee in its meeting held on 26th October, 2009 further approved certain modifications to Clause 210.1, 202.3, 205, Note below Clause 208, 209.1, 209.4, 209.7, 222.5.5, Table 8, Note below Table 8, 222.8, 222.9, Table 1 and deletion of Clause 213.8, 214.5.1.2 and Note below para 8 of Appendix-3. The Convenor of B-2 Committee was authorized to incorporate these modifications in the draft for Fifth Revision of IRC:6, in the light of the comments of some members. The Executive Committee, in its meeting held on 31st October, 2009, and the IRC Council in its 189th meeting held on 14th November, 2009 at Patna approved publishing of the Fifth Revision of IRC: 6.

The 6th Revision of IRC: 6 includes all the amendments and errata published from time to time upto December, 2013. The revised edition of IRC was approved by the Bridges Specifications and Standards Committee in its meeting held on 06.01.2014 and Executive Committee meeting held on 09.01.2014 for publishing.

The 7th revision of IRC: 6-2016, includes all amendments and errata published in Indian Highways up to November 2016. All these amendments are approved by Bridges Specifications and Standard Committee meetings. The Bridges Specification and Standard Committee approved the proposed amendments in changing the title as “Loads & Loads Combination” instead of “Load & Stresses” in order to bring the functional harmony of code. This was discussed in 209th mid-term Council meet held on 26 September 2016 and council approved the proposed amendments and change in the title of code for publications.

The personnel of the Loads and Stresses Committee (B-2) is given below:

Banerjee, A.K.	......	Convenor
Parameswaran, (Mrs.) Dr. Lakshmy	......	Co-Convenor
Sharma, Aditya	......	Member Secretary

Members

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Corresponding Members

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Jain, Dr Sudhir K	Subbarao, Dr H

Ex-officio Members

(Porwal, S.S.)	President, Indian Roads Congress
	ADG, BRO
(Das. S.N.)	Director General (Road Development) & Special Secretary, Ministry of Road Transport and Highways & Honorary Treasurer, Indian Roads Congress, New Delhi
Nahar, Sajjan Singh	Secretary General, Indian Roads Congress, New Delhi

SCOPE

The object of the Standard Specifications and Code of Practice is to establish a common procedure for the design and construction of road bridges in India. This publication is meant to serve as a guide to both the design engineer and the construction engineer but compliance with the rules therein does not relieve them in any way of their responsibility for the stability and soundness of the structure designed and erected by them. The design and construction of road bridges require an extensive and through knowledge of the science and technique involved and should be entrusted only to specially qualified engineers with adequate practical experience in bridge engineering and capable of ensuring careful execution of work.

201 CLASSIFICATION

201.1 Road bridges and culverts shall be divided into classes according to the loadings they are designed to carry.

IRC CLASS 70R LOADING: This loading is to be normally adopted on all roads on which permanent bridges and culverts are constructed. Bridges designed for Class 70R Loading should be checked for Class A Loading also as under certain conditions, heavier stresses may occur under Class A Loading.

IRC CLASS AA LOADING: This loading is to be adopted within certain municipal limits, in certain existing or contemplated industrial areas, in other specified areas, and along certain specified highways. Bridges designed for Class AA Loading should be checked for Class A Loading also, as under certain conditions, heavier stresses may occur under Class A Loading.

IRC CLASS A LOADING: This loading is to be normally adopted on all roads on which permanent bridges and culverts are constructed.

IRC CLASS B LOADING: This loading is to be normally adopted for timber bridges.

IRC CLASS SPECIAL VEHICLE (SV) LOADING: This loading is to be adopted for design of new bridges in select corridors as may be decided by concerned authorities where passage of trailer vehicles carrying stator units, turbines, heavy equipment and machinery may occur occasionally. This loading represents a spectrum of special vehicles in the country and should be considered for inclusion in the design wherever applicable.

For particulars of the above five types of loading, see Clause **204**.

201.2 Existing bridges which were not originally constructed or later strengthened to take one of the above specified I.R.C. Loadings will be classified by giving each a number equal to that of the highest standard load class whose effects it can safely withstand.

Annex A gives the essential data regarding the limiting loads in each bridge's class, and forms the basis for the classification of bridges.

201.3 Individual bridges and culverts designed to take electric tramways or other special loadings and not constructed to take any of the loadings described in Clause **201.1** shall be classified in the appropriate load class indicated in Clause **201.2**.

202 LOADS, FORCES AND LOAD EFFECTS

202.1 The loads, forces and load effects to be considered in designing road bridges and culverts are :

1)	Dead Load	G
2)	Live Load	Q
3)	Snow Load (See note i)	G_s
4)	Impact factor on vehicular live load	Q_{im}
5)	Impact due to floating bodies or Vessels as the cases may be	F_{im}
6)	Vehicle collision load	V_c
7)	Wind load	W
8)	Water current	F_{wc}
9)	Longitudinal forces caused by tractive effort of vehicles or by braking of vehicles and/or those caused by restraint of movement of free bearings by friction or deformation	$F_a/F_b/F_f$
10)	Centrifugal force	F_{cf}
11)	Buoyancy	G_b
12)	Earth Pressure including live load surcharge, if any	F_{ep}
13)	Temperature effects (see note ii)	F_{te}
14)	Deformation effects	F_d

15)	Secondary effects	F_s
16)	Erection effects	F_{er}
17)	Seismic force	F_{eq}
18)	Wave pressure (see note iii)	F_{wp}
19)	Grade effect (see note iv)	G_e

Notes :

1. *The snow loads may be based be based on actual observation or past records in the particular area or local practices, if existing.*
2. *Temperature effects (F_{te}) in this context is not the frictional force due to the movement of bearing but forces that are caused by the restraint effects.*
3. *The wave forces shall be determined by suitable analysis considering drawing and inertia forces etc. on single structural members based on rational methods or model studies. In case of group of piles, piers etc., proximity effects shall also be considered.*
4. *For bridges built in grade or cross-fall, the bearings shall normally be set level by varying the thickness of the plate situated between the upper face of the bearing and lower face of the beam or by any other suitable arrangement. However, where the bearings are required to be set parallel to the inclined grade or cross-fall of the superstructure, an allowance shall be made for the longitudinal and transverse components of the vertical loads on the bearings.*

202.2 All members shall be designed to sustain safely most critical combination of various loads, forces and stresses that can co-exist and all calculations shall tabulate distinctly the various combinations of the above loads and stresses covered by the design. Besides temperature, effect of environment on durability shall be considered as per relevant codes.

202.3 Combination of Loads and Forces and Permissible Increase in Stresses

The load combination shown in **Table 1** shall be adopted for working out stresses in the members. The permissible increase of stresses in various members due to these combinations is also indicated therein. These combinations of forces are not applicable for working out base pressure on foundations for which provision made in relevant IRC Bridge Code shall be adopted. For calculating stresses in members using working stress method of design the load combination shown in **Table 1** shall be adopted.

The load combination as shown in **Annex B** shall be adopted for limit state design approach.

Table 1: Load Combinations and Permissible Stresses (Clause 202.3)

1	2	3	4	5	6	7	8	9	10			11	12	13	14	15	16	17	18	19	20	21	22
	G	Q	G_s	Q_{im}	F_{im}	V_c	W	F_{wc}	(F_a or F_b) &/ F_f			F_{cf}	G_b	F_{ep}	F_{te}	F_d	Fs	F_{er}	F_{eq}	F_{wp}	G_e	%	
	Dead Load (G)	Live Load (Q)	Snow Load (G_s)	Vehicle Impact (Q_{im})	Impact Floating Bodies (F_{im})	Vehicle Collision Load (V_c)	Wind (W)	Water Current (F_{wc})	Tractive (F_a)	Braking (F_b)	Bearing Friction (F_f)	Centrifugal Force (F_{cf})	Buoyancy (G_b)	Earth Pressure (F_{ep})	Temperature (F_{te})	Deformation effects (F_d)	Secondary effects (F_s)	Erection Effects (F_{er})	Seismic (F_{eq})	Wave Pressure (F_{wp})	Grade Effect (G_e)	Permissible Stresses	Remarks
I	1	1	*	1				1	1	1	1	1	1	1							1	100	Service Condition
IIA	1	1	*	1				1	1	1	1	1	1	1	1	1	1				1	115	
IIB	1	0.5		1				1	0.5	0.5	1	0.5	1	1	1	1	1				1	115	
IIIA	1	1	*	1			1	1	1	1	1	1	1	1	1	1	1			1	1	133	
IIIB	1	0.5		1			1	1	0.5	0.5	1	0.5	1	1	1	1	1			1	1	133	
IV	1	1	*	1			1	1	1	1	1	1	1	1		1	1			1	1	133	
V	1					1																150	
VI	1	0.2		1				1	0.2	0.2	1	0.2	1	1	1	1	1		1	1	1	150	
VII	1	1	*	1	1		1	1	1	1	1	1	1	1	1	1	1				1	133	
VIII	1						1	1			1		1	1				1			1	133	Construction condition
IX	1							1			1		1	1				1	0.5		1	150	

Notes:

1) **Where Snow Load is applicable, Clause **221** shall be referred for combination of snow load and live load*
2) *Any load combination involving temperature, wind and/or earthquake acting independently or in combination, maximum permissible tensile stress in Prestressed Concrete Members shall be limited to the value as per relevant Code (IRC:112).*
3) *Use of fractional live load shown in **Table 1** is applicable only when the design live load given in **Table 6** is considered. The structure must also be checked with no live load.*
4) *The gradient effect due to temperature is considered in the load combinations IIB and IIIB. The reduced live load (Q) is indicated as 0.5. Its effects (F_a, F_b and F_{cf}) are also shown as 0.5, as 0.5 stands for the reduced live load to be considered in this case. However for F_f it is shown as 1, since it has effects of dead load besides reduced live load. Q_{im} being a factor of live load as shown as 1. Whenever a fraction of live load 0.5 shown in the above Table under column Q is specified, the associated effects due to live load (Q_{im}, F_a, F_b, F_f and F_{cf}) shall be considered corresponding to the associated fraction of live load. When the gradient effect is considered, the effects, if any due to overall rise of fall of temperature of the structure shall also be considered.*
5) *Seismic effect during erection stage is reduced to half in load combination IX when construction phase does not exceed 5 years.*
6) *The load combinations (VIII and IX) relate to the construction stage of a new bridge. For repair, rehabilitation and retrofitting, the load combination shall be project-specific.*
7) *Clause **219.5.2** may be referred to, for reduction of live load in Load Combination VI.*

203 DEAD LOAD

The dead load carried by a girder or member shall consist of the portion of the weight of the superstructure (and the fixed loads carried thereon) which is supported wholly or in part by the girder or member including its own weight. The following unit weights of materials shall be used in determining loads, unless the unit weights have been determined by actual weighing of representative samples of the materials in question, in which case the actual weights as thus determined shall be used.

	Materials	Weight (t/m^3)
1)	Ashlar (granite)	2.7
2)	Ashlar (sandstone)	2.4
3)	Stone setts :	
	a) Granite	2.6
	b) Basalt	2.7

4)	Ballast (stone screened, broken, 2.5 cm to 7.5 cm guage, loose):	
	a) Granite	1.4
	b) Basalt	1.6
5)	Brickwork (pressed) in cement mortar	2.2
6)	Brickwork (common) in cement mortar	1.9
7)	Brickwork (common) in lime mortar	1.8
8)	Concrete (asphalt)	2.2
9)	Concrete (breeze)	1.4
10)	Concrete (cement-plain)	2.5
11)	Concrete (cement – plain with plums)	2.5
12)	Concrete (cement-reinforced)	2.5
13)	Concrete (cement-prestressed)	2.5
14)	Concrete (lime-brick aggregate)	1.9
15)	Concrete (lime-stone aggregate)	2.1
16)	Earth (compacted)	2.0
17)	Gravel	1.8
18)	Macadam (binder premix)	2.2
19)	Macadam (rolled)	2.6
20)	Sand (loose)	1.4
21)	Sand (wet compressed)	1.9
22)	Coursed rubble stone masonry (cement mortar)	2.6
23)	Stone masonry (lime mortar)	2.4
24)	Water	1.0
25)	Wood	0.8
26)	Cast iron	7.2
27)	Wrought iron	7.7
28)	Steel (rolled or cast)	7.8

204 LIVE LOADS

204.1 Details of I.R.C. Loadings

204.1.1 For bridges classified under Clause **201.1**, the design live load shall consist of standard wheeled or tracked vehicles or trains of vehicles as illustrated in **Figs. 1, 2 & 4** and **Annex A** or Special Vehicle (SV) as per Clause **204.5**, if applicable. The trailers attached to the driving unit are not to be considered as detachable.

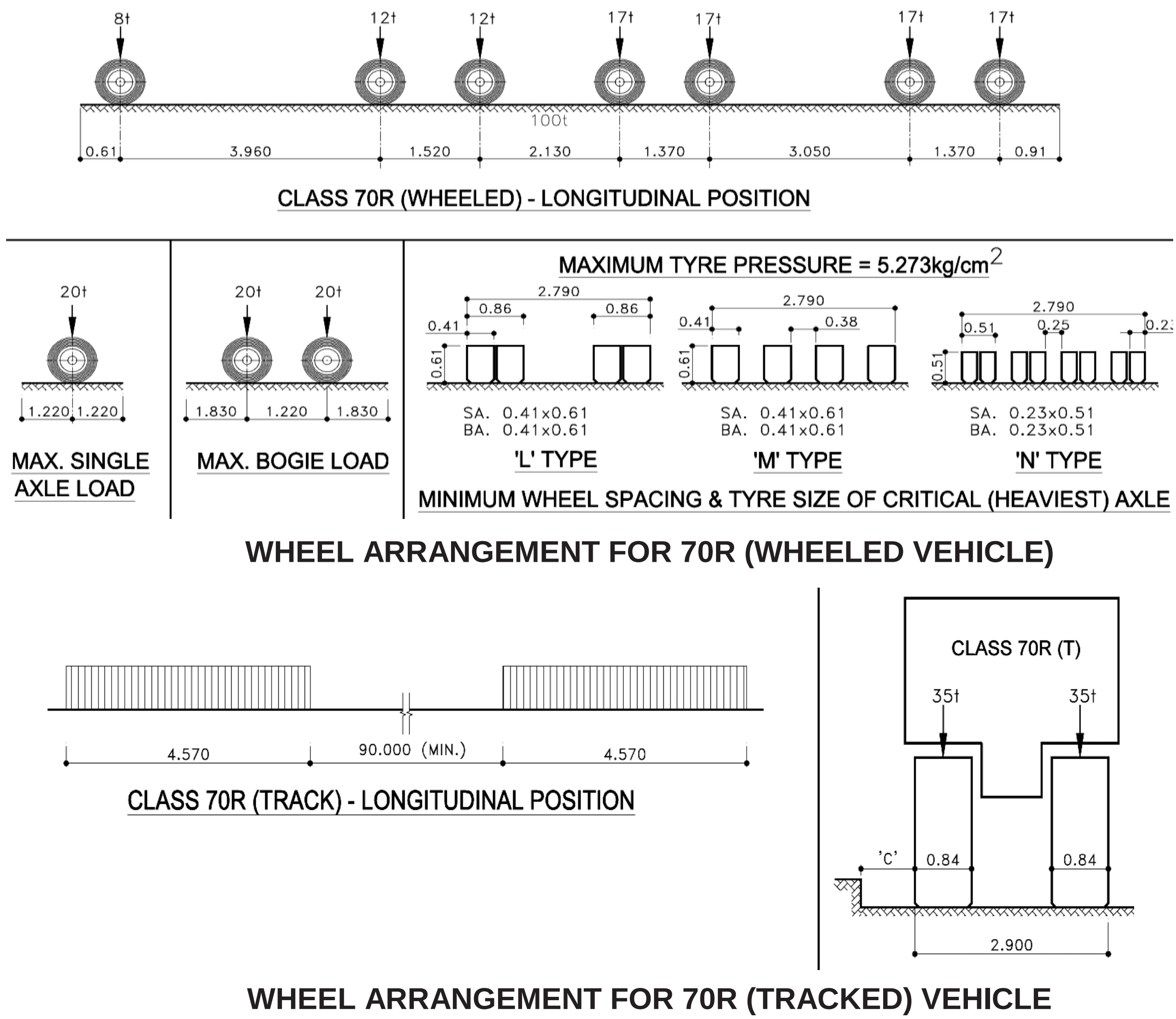


Fig. 1: Class 70 R Wheeled and Tracked Vehicles (Clause 204.1)

Notes:

1) *The nose to tail spacing between two successive vehicles shall not be less than 90 m for tracked vehicle. For wheeled vehicle, spacing between successive vehicles shall not be less than 30 m. It will be measured from the centre of the rear-most axle of the leading vehicle to the centre of the first axle of the following vehicle.*

2) *For multi-lane bridges and culverts, each Class 70R loading shall be considered to occupy two lanes and no other vehicle shall be allowed in these two lanes. The passing/crossing vehicle can only be allowed on lanes other than these two lanes. Load combination is as shown in **Table 6 & 6A**.*
3) *The maximum loads for the wheeled vehicle shall be 20 tonne for a single axle or 40 tonne for a bogie of two axles spaced not more than 1.22 m centres.*
4) *Class 70R loading is applicable only for bridges having carriageway width of 5.3 m and above (i.e. 1.2 x 2 + 2.9 = 5.3). The minimum clearance between the road face of the kerb and the outer edge of the wheel or track, 'C', shall be 1.2 m.*
5) *The minimum clearance between the outer edge of wheel or track of passing or crossing vehicles for multilane bridge shall be 1.2 m. Vehicles passing or crossing can be either same class or different class, Tracked or Wheeled.*
6) *Axle load in tonnes, linear dimension in meters.*
7) *For tyre tread width deductions and other important notes, refer NOTES given in **Annex A**.*

204.1.2 Within the kerb to kerb width of the roadway, the standard vehicle or train shall be assumed to travel parallel to the length of the bridge and to occupy any position which will produce maximum stresses provided that the minimum clearances between a vehicle and the roadway face of kerb and between two passing or crossing vehicles, shown in **Figs. 1, 2 & 4**, are not encroached upon

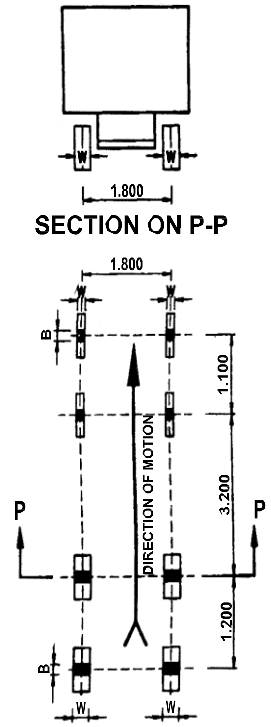
204.1.3 For each standard vehicle or train, all the axles of a unit of vehicles shall be considered as acting simultaneously in a position causing maximum stresses.

204.1.4 Vehicles in adjacent lanes shall be taken as headed in the direction producing maximum stresses.

204.1.5 The spaces on the carriageway left uncovered by the standard train of vehicles shall not be assumed as subject to any additional live load unless otherwise shown in **Table 6**.

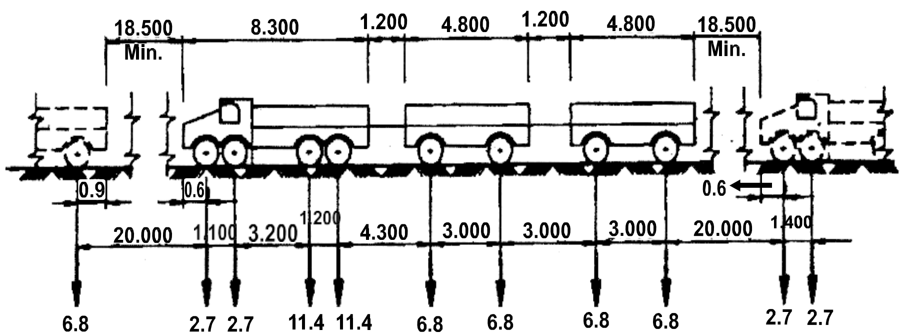
204.2 Dispersion of Load through Fills of Arch Bridges

The dispersion of loads through the fills above the arch shall be assumed at 45 degrees both along and perpendicular to the span in the case of arch bridges.



PLAN

DRIVING VEHICLE



Class A Train of Vehicles

Fig. 2: Class 'A' Train of Vehicles (Clause 204.1)

Notes:

1) *The nose to tail distance between successive trains shall not be less than 18.5 m.*
2) *For single lane bridges having carriageway width less than 5.3 m, one lane of Class A shall be considered to occupy 2.3 m. Remaining width of carriageway shall be loaded with 500 Kg/m^2, as shown in **Table 6**.*
3) *For multi-lane bridges each Class A loading shall be considered to occupy single lane for design purpose. Live load combinations as shown in **Table 6** shall be followed.*
4) *The ground contact area of the wheels shall be as given in **Table 2**.*

Table 2: Ground Contact Dimensions for Class A Loading

Axle load (tonne)	Ground contact area	
	B (mm)	W (mm)
11.4	250	500
6.8	200	380
2.7	150	200

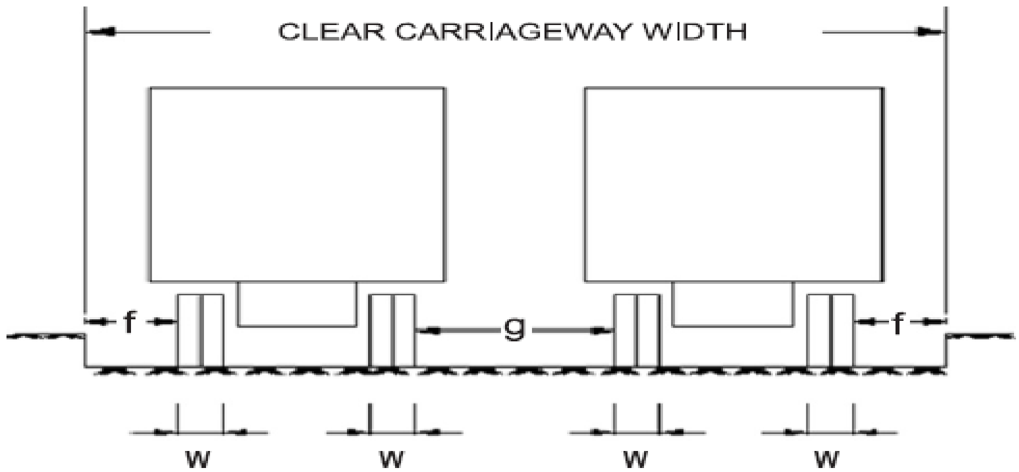


Fig.3: Minimum Clearance for 2 Class A Train Vehicles

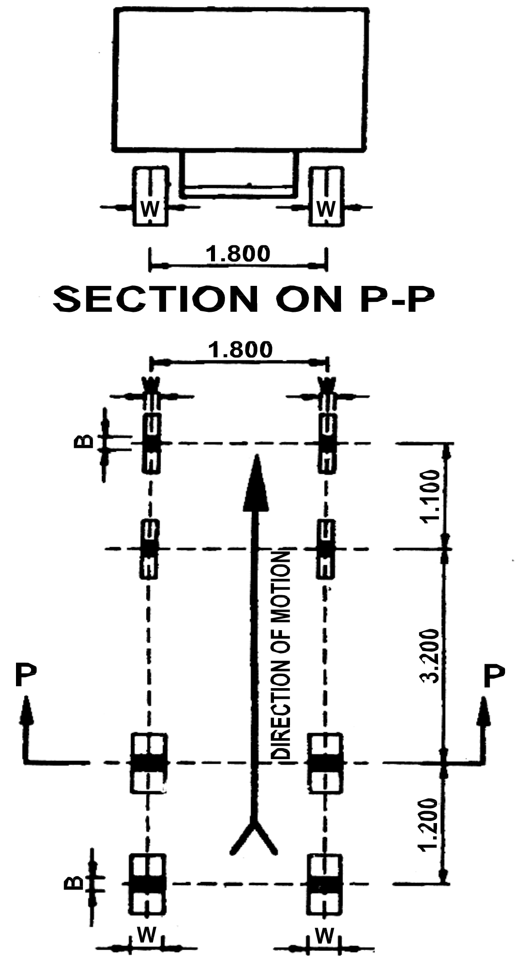
5) *The minimum clearance, f, between outer edge of the wheel and the roadway face of the kerb and the minimum clearance, g, between the outer edges of passing or crossing vehicles on multi-lane bridges shall be as given in* ***Table 3****.*

Table 3: Minimum Clearance for Class A Train Vehicle

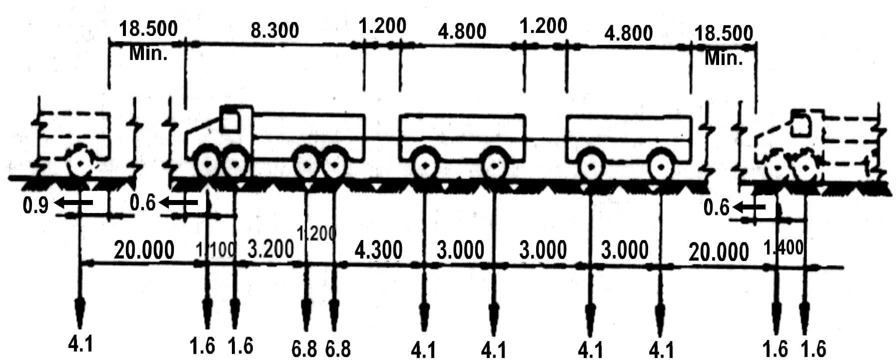
Clear carriageway width	g	*f*
5.3 m(*) to 6.1 m(**)	Varying between 0.4 m to 1.2 m	150 mm for all carriageway width
Above 6.1 m	1.2 m	

(*) = [2x(1.8+0.5)+0.4+2x0.15]
(**)= [2x(1.8+0.5)+1.2+2x0.15]

6) *Axle loads in tonne. Linear dimensions in metre*.



PLAN
DRIVING VEHICLE



Class B Train of Vehicles

Fig. 4: Class 'B' Train of Vehicles (Clause 204.1)

Notes:

1) *The nose to tail distance between successive trains shall not be less than 18.5 m.*
2) *No other live load shall cover any part of the carriageway when a train of vehicles (or trains of vehicles in multi-lane bridge) is crossing bridge.*
3) *The ground contact area of the wheels shall be as given in **Table 4**.*

Table 4: Ground Contact Dimensions for Class B Loading

Axle load (tonne)	Ground contact area	
	B (mm)	W (mm)
6.8	200	380
4.1	150	300
1.6	125	175

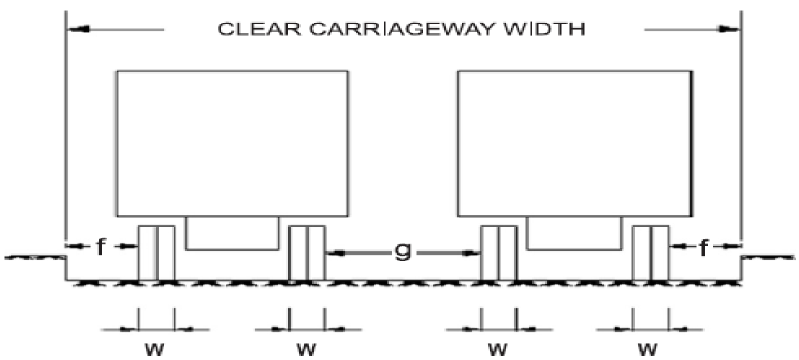


Fig. 5: Minimum Clearance for 2 Class B Train

4) *For bridges having carriageway width less than 5.06 m, only single lane of Class B loading shall be considered.*
5) *The minimum clearances, f, between outer edge of the wheel and the roadway face of the kerb and the minimum clearance, g, between the outer edges of passing or crossing vehicles on multi-lane bridges shall be as given in **Table 5***
6) *Axle loads in tonne. Linear dimensions in metre*

Table 5: Minimum Clearance for Class B Train

Clear carriageway width	g	*f*
5.06 m(*) to 5.86 m(**)	Varying between 0.4 m to 1.2 m	150 mm for all carriageway width
Above 5.86 m	1.2 m	

(*) = [2x(1.8+0.38)+0.4+2x0.15]
(**)= [2x(1.8+0.38)+1.2+2x0.15]

204.3 Combination of Live Load

This clause shall be read in conjunction with Clause **104.3** of **IRC:5**. The carriageway live load combination shall be considered for the design as shown in **Table 6** .

Table 6: Live Load Combination

S.No	Carriageway Width (CW)	Number of Lanes for Design Purposes	Load Combination (Refer Table 6A for diagrammatic representation)
1)	Less than 5.3 m	1	One lane of Class A considered to occupy 2.3 m. The remaining width of carriageway shall be loaded with 500 kg/m^2
2)	5.3 m and above but less than 9.6 m	2	One lane of Class 70R OR two lanes for Class A
3)	9.6 m and above but less than 13.1 m	3	One lane of Class 70R for every two lanes with one lanes of Class A on the remaining lane OR 3 lanes of Class A
4)	13.1 m and above but less than 16.6 m	4	One lane of Class 70R for every two lanes with one lane of Class A for the remaining lanes, if any, OR one lane of Class A for each lane.
5)	16.6 m and above but less than 20.1 m	5	
6)	20.1 m and above but less than 23.6 m	6	

Notes :

1) *The minimum width of the two-lane carriageway shall be 7.5 m as per Clause* ***104.3*** *of* ***IRC:5.***
2) *See Note No. 2 below* ***Fig. A-1*** *of* ***Annex A*** *regarding use of 70R loading in place of Class AA Loading and vice-versa.*

Table 6A: Live Load Combinations

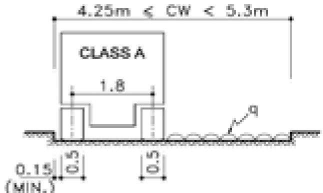
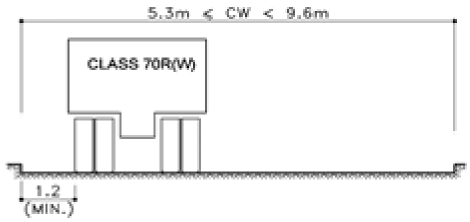
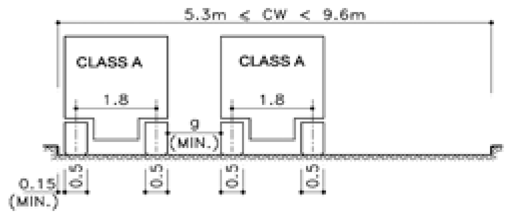
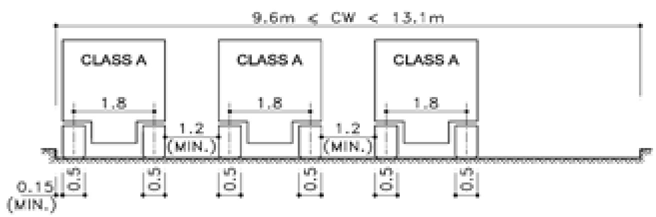
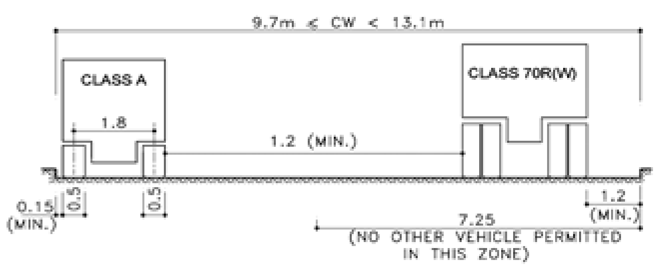
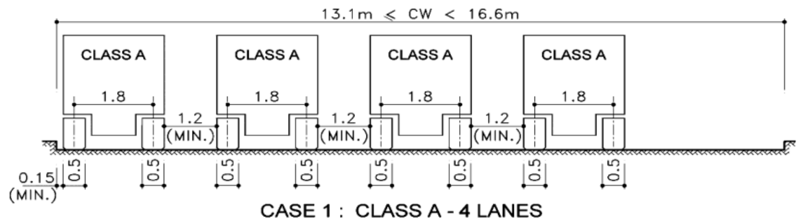
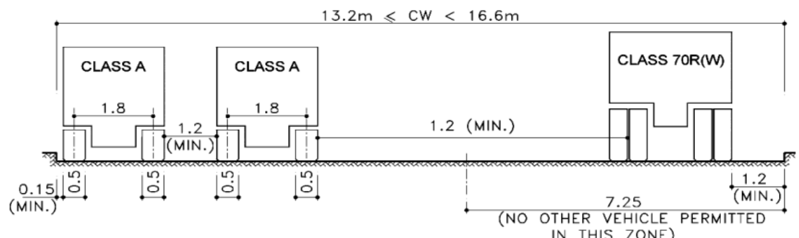
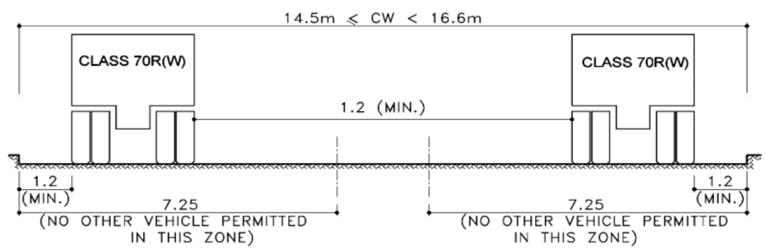
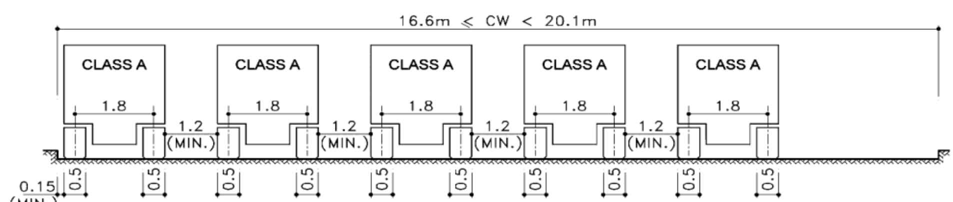
S.NO.	NO. OF LANES FOR DESIGN PURPOSE	CARRIAGEWAY WIDTH (CW) & LOADING ARRANGEMENT
1.	1 LANE	 CASE 1 : CLASS A - 1 LANE
2.	2 LANES	 CASE 1 : CLASS 70R (W)
		 CASE 2 : CLASS A - 2 LANES
3.	3 LANES	 CASE 1 : CLASS A - 3 LANES
		 CASE 2 : CLASS A - 1 LANE + CLASS 70R (W)

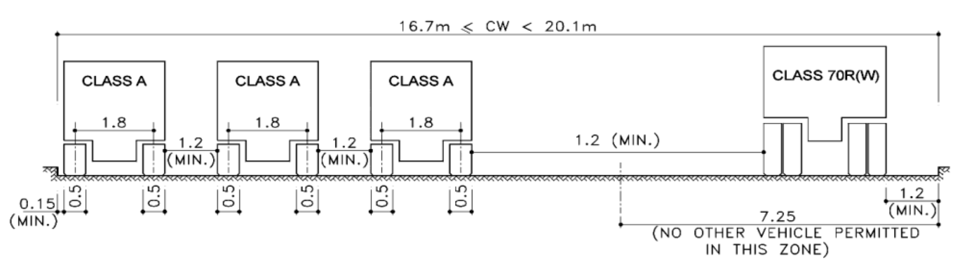
Table 6A: Live Load Combinations contd..

<table>
<tr><th>S.NO.</th><th>NO. OF LANES FOR DESIGN PURPOSE</th><th>CARRIAGEWAY WIDTH (CW) & LOADING ARRANGEMENT</th></tr>
<tr><td rowspan="3">4.</td><td rowspan="3">4 LANES</td><td>

CASE 1 : CLASS A - 4 LANES</td></tr>
<tr><td>

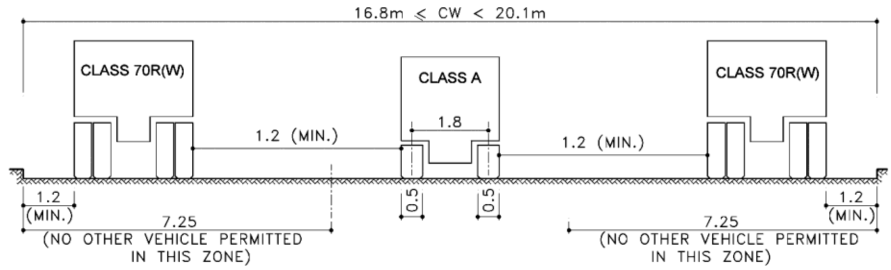
CASE 2 : CLASS A - 2 LANE +CLASS 70R (W)</td></tr>
<tr><td>

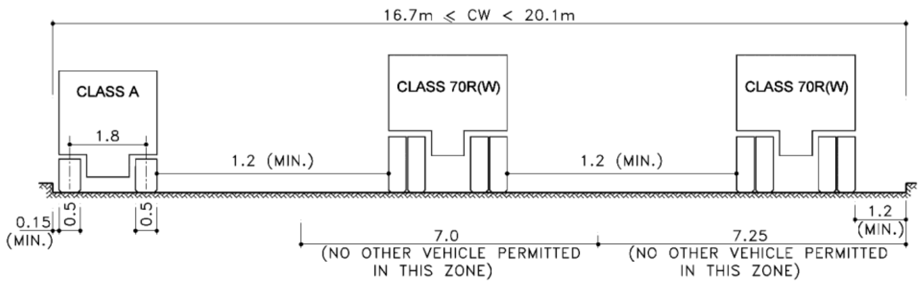
CASE 3 : CLASS 70R (W) - 2 LANES</td></tr>
<tr><td rowspan="2">5.</td><td rowspan="2">5 LANES</td><td>

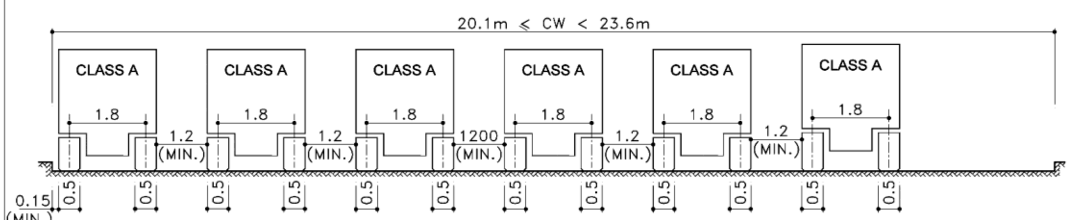
CASE 1 : CLASS A - 5 LANES</td></tr>
<tr><td>

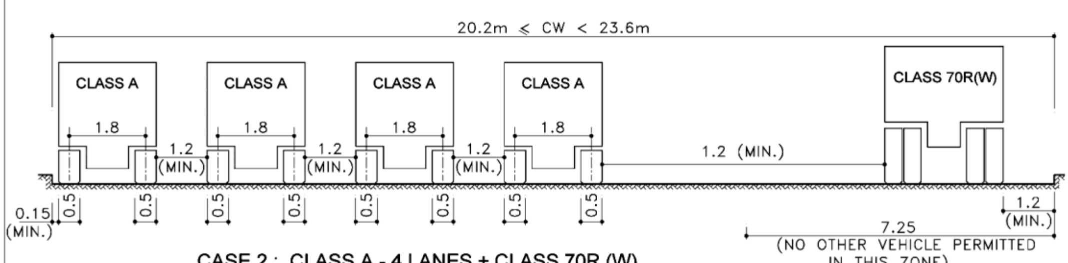
CASE 2 : CLASS A - 3 LANES +CLASS 70R (W)</td></tr>
</table>

Table 6A: Live Load Combinations contd..

<table>
<tr><th>S.NO.</th><th>NO. OF LANES FOR DESIGN PURPOSE</th><th>CARRIAGEWAY WIDTH (CW) & LOADING ARRANGEMENT</th></tr>
<tr><td rowspan="2">5.</td><td rowspan="2">5 LANES CONTD.....</td><td>


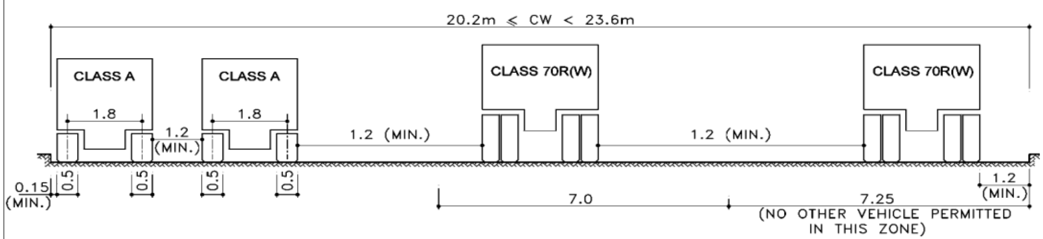
CASE 3 : CLASS 70R (W) - 2 LANES + CLASS A -1 LANE
</td></tr>
<tr><td>


CASE 4 : CLASS A -1 LANE + CLASS 70R (W) - 2 LANES
</td></tr>
<tr><td rowspan="2">6.</td><td rowspan="2">6 LANES</td><td>


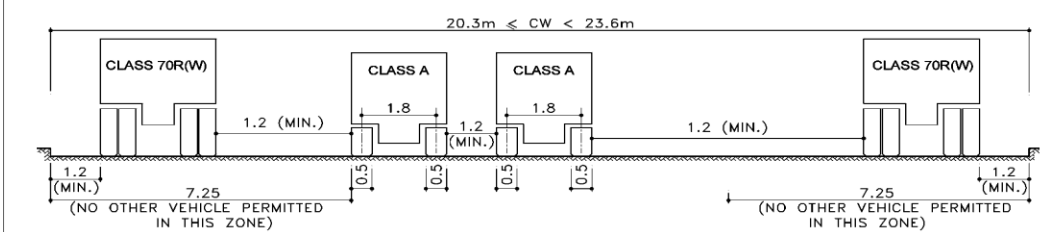
CASE 1 : CLASS A - 6 LANES
</td></tr>
<tr><td>


CASE 2 : CLASS A - 4 LANES + CLASS 70R (W)
</td></tr>
</table>

Table 6A: Live Load Combinations contd..

<table>
<tr><th>S.NO.</th><th>NO. OF LANES FOR DESIGN PURPOSE</th><th>CARRIAGEWAY WIDTH (CW) & LOADING ARRANGEMENT</th></tr>
<tr><td>6.</td><td>6 LANES CONTD.....</td><td>


CASE 3 : CLASS A - 2- LANES + CLASS 70R (W) - 2 LANES



CASE 4 : CLASS 70R (W) + CLASS A - 2 LANES + CLASS 70R (W)
</td></tr>
</table>

Notes:

a) *Class 70R Wheeled loading in the* ***Table 6 & 6A*** *can be replaced by Class 70R tracked, Class AA tracked or Class AA wheeled vehicle.*

b) *Maximum number of vehicles which can be considered, are only shown in the* ***Table 6A****. In case minimum number of vehicles govern the design (e.g. torsion) the same shall also be considered.*

c) *All dimensions in* ***Table 6A*** *are in metre.*

204.4 Congestion Factor

For bridges, Flyovers/grade separators close to areas such as ports, heavy industries and mines and any other areas where frequent congestion of heavy vehicles may occur, as may be decided by the concerned authorities, additional check for congestion of vehicular live load on the carriageway shall be considered. In the absence of any stipulated value, the congestion factor, as mentioned in **Table 7** shall be considered as multiplying factor on the global effect of vehicular live load (including impact). Under this condition, horizontal force due to braking/acceleration, centrifugal action, temperature effect and effect of transverse eccentricity of live load impact shall not be included.

Table 7: Congestion Factor

S. No.	Span Range	Congestion factor
1)	Above 10 m and upto 30 m	1.15
2)	30.0 m to 40.0 m	1.15 to 1.30
3)	40.0 m to 50.0 m	1.30 to 1.45
4)	50.0 m to 60.0 m	1.45 to 1.60
5)	60.0 m to 70.0 m	1.60 to 1.70
6)	Beyond 70.0 m	1.70

Note: *For Intermediate bridges spans, the value of multiplying factor may be interpolated.*

204.5 Special Vehicle (SV)

IRC Class SV Loading: Special Multi Axle Hydraulic Trailer Vehicle

(Prime Mover with 20 Axle Trailer - GVW = 385 Tonnes)

204.5.1 The longitudinal axle arrangement of SV loading shall be as given in **Fig 6.**

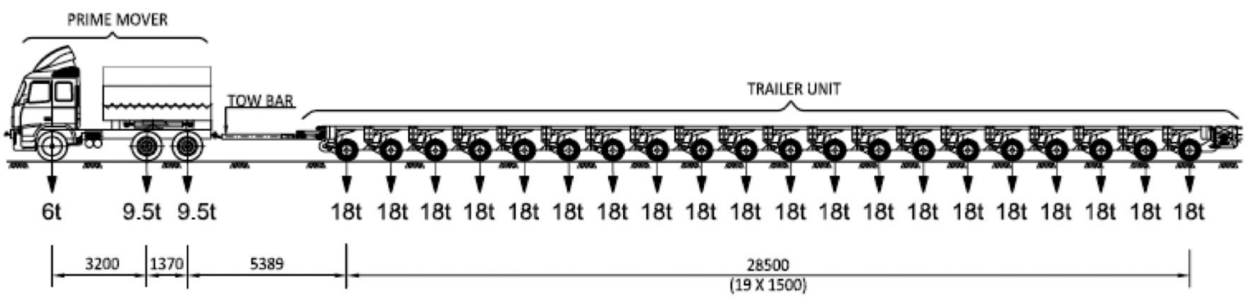


Fig 6: Typical Axle Arrangement for Special Vehicle

204.5.2 The transverse wheel spacing and the axle arrangement of SV loading shall be as given in **Fig. 6A**

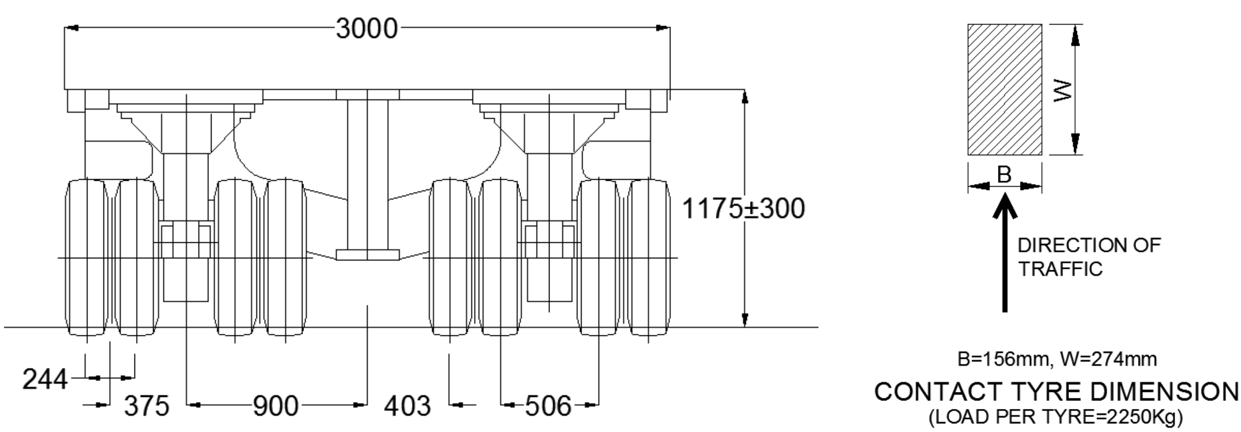


Fig 6A: Transverse Wheel Spacing of Special Vehicle

204.5.3 The SV loading shall be considered to ply close to center of carriageway with a maximum eccentricity of 300 mm for single carriageway bridges or for dual carriageway bridges, as shown **Fig. 6B**

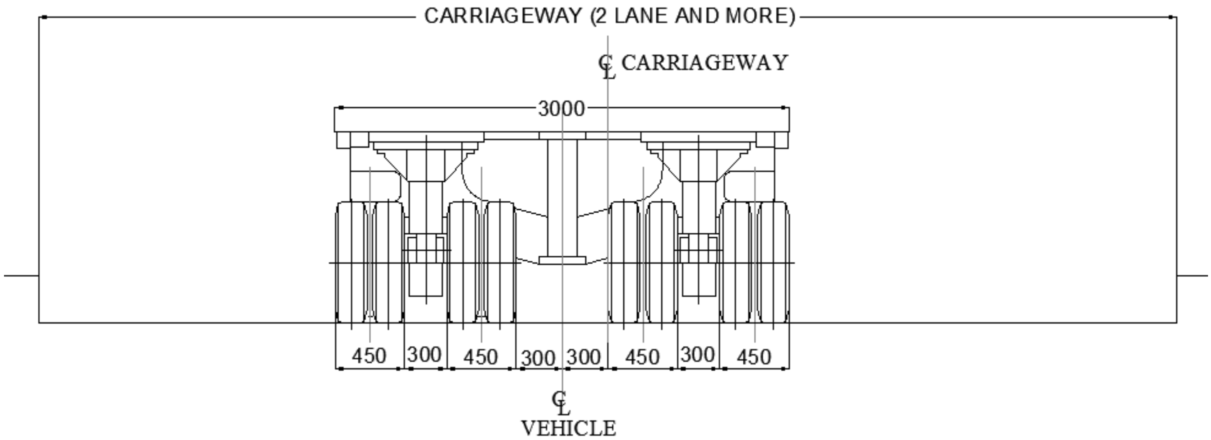


Fig. 6B: Transverse placement for Special Vehicle

Note*: Dimensions in all the above sketches are in millimetres*

204.5.4 During the passage of SV loading, no other vehicle shall be considered to ply on the bridge. No wind, seismic, braking force and dynamic impact on the live load need to be considered as the SV shall move at a speed not exceeding 5kmph over the bridge. For the load combination with special vehicle, the partial safety factor on live load for verification of equilibrium and structural strength under Ultimate Limit State and for verification of Serviceability Limit State shall be taken as 1.0.

Note*: The movement of Special Vehicle shall be regulated / monitored to ensure that it moves at a speed less than 5 kmph and also does not ply on the bridge on a high wind condition.*

204.6 Fatigue Load

Movement of traffic on bridges causes fluctuating stresses, resulting into possible fatigue damage. The stress spectrum due to vehicular traffic depends on the composition of traffic, vehicle attributes i.e., gross vehicle weight, axle spacing and axle load, vehicle spacing, structural configuration of the bridge and dynamic effects.

The truck defined in **Fig. 7A** shall be used for the fatigue life assessment of steel, concrete and composite bridges. The transverse wheel spacing and tyre arrangement of this truck shall be as per **Fig. 7B.** 50% of the impact factors mentioned in Clause **208** shall be applied to this fatigue load.

The stress range resulting from the single passage of the fatigue load along the longitudinal direction of the bridge, shall be used for fatigue assessment with the fatigue load so positioned as to have worst effect on the detail or element of the bridge under consideration. The minimum clearance between outer edge of the wheel of the fatigue vehicle and roadway face of the kerb shall be 150 mm.

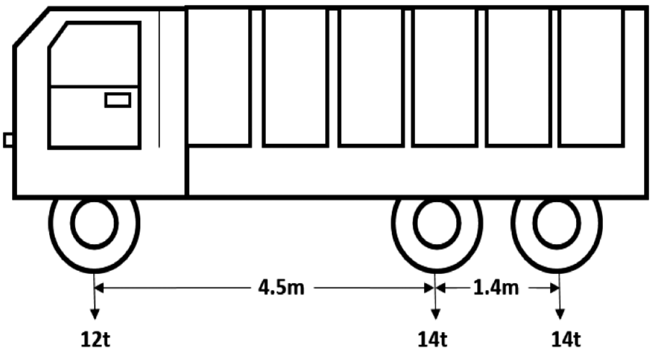


Fig. 7A: Fatigue Truck

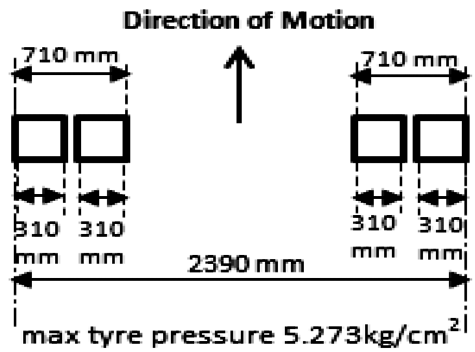


Fig. 7B: Transverse Wheel Spacing and Tyre Arrangement

Fig. 7: Fatigue Load (40T)

For all types of bridges (i.e. Concrete, Steel or Composite) the fatigue check shall be carried out under frequent combination of Serviceability Limit State (SLS), with load factors for fatigue load, taken as equal to 1.0. For design for fatigue limit state, reference shall be made to. **IRC:112** for Concrete bridges, **IRC:24** for Steel bridges and **IRC:22** for Steel Concrete Composite bridges.

In absence of any specific provision in these codes, following number of cycles may be considered for fatigue assessment, depending upon the location of the bridge and the category of roads:

1) The bridges close to areas such as ports, heavy industries and mines and other areas, where generally heavy vehicles ply shall be designed for the stress induced due to 10 x 10^6 cycles.
2) Other bridges shall be designed for the stress induced due to 2 x 10^6 cycles.

Bridges on rural roads need not be designed for fatigue.

205 REDUCTION IN THE LONGITUDINAL EFFECT ON BRIDGES ACCOMMODATING MORE THAN TWO TRAFFIC LANES

Reduction in the longitudinal effect on bridges having more than two traffic lanes due to the low probability that all lanes will be subjected to the characteristic loads simultaneously shall be in accordance with the **Table 8.**

Table 8: Reduction in Longitudinal Effects

Number of lanes	Reduction in longitudinal effect
For two lanes	No reduction
For three lanes	10% reduction
For four lanes	20% reduction
For five or more lanes	20% reduction

Notes:

1) *However, it should be ensured that the reduced longitudinal effects are not less severe than the longitudinal effect, resulting from simultaneous loads on two adjacent lanes. Longitudinal effects mentioned above are bending moment, shear force and torsion in longitudinal direction.*
2) ***Table 8** is applicable for individually supported superstructure of multi-laned carriageway. In the case of separate sub-structure and foundations, the number of lanes supported by each of them is to be considered while working out the reduction percentage. In the case of combined sub-structure and foundations, the total number of lanes for both the carriageway is to be considered while working out the reduction percentage.*

206 FOOT OVER BRIDGE, FOOTWAY, KERB, RAILINGS, PARAPET AND CRASH BARRIERS

The horizontal force specified for footway, kerb, railings, parapet and crash barriers in this section need not be considered for the design of main structural members of the bridge. However, the connection between kerb/railings/parapet, crash barrier and the deck should be adequately designed and detailed.

206.1 For all parts of bridge floors accessible only to pedestrians and animals and for all footways the loading shall be 400 kg/m^2. For the design of foot over bridges the loading shall be taken as 500 kg/m^2. Where crowd loads are likely to occur, such as, on bridges located near towns, which are either centres of pilgrimage or where large congregational fairs are held seasonally, the intensity of footway loading shall be increased from 400 kg/m^2 to 500 kg/m^2. When crowd load is considered, the bridge should also be designed for the case of entire carriageway being occupied by crowd load.

206.2 Kerbs, 0.6 m or more in width, shall be designed for the above loads and for a local lateral force of 750 kg per metre, applied horizontally at top of the kerb. If kerb width is less than 0.6 m, no live load shall be applied in addition to the lateral load specified above.

206.3 In bridges designed for any of the loadings described in Clause **204.1**, the main girders, trusses, arches, or other members supporting the footways shall be designed for the following live loads per square metre for footway area, the loaded length of footway taken in each case being, such as, to produce the worst effects on the member under consideration:

a) For effective span of 7.5 m or less, 400 kg/m^2 or 500 kg/m^2 as the case may be, based on Sub-Clause **206.1**.

b) For effective spans of over 7.5 m but not exceeding 30 m, the intensity of load shall be determined according to the equation:

$$P = P' - \left(\frac{40L - 300}{9}\right)$$

c) For effective spans of over 30 m, the intensity of load shall be determined according to the equation :

$$P = \left(P' - 260 + \frac{4800}{L}\right)\left(\frac{16.5 - W}{15}\right)$$

where,

P'	=	400 kg/m^2 or 500 kg/m^2 as the case may be, based on Sub-Clause **206.1.** When crowd load is considered for design of the bridge, the reduction mentioned in this clause will not be applicable.
P	=	the live load in kg/m^2
L	=	the effective span of the main girder, truss or arch in m, and
W	=	width of the footway in m

206.4 Each part of the footway shall be capable of resisting an accidental load of 4 tonne, which shall be deemed to include impact, distributed over a contact area of 300 mm in diameter. For working stress approach, the permissible stress shall be increased by 25% to meet this provision. For limit state design, the load combination as per **Table B-2** shall be followed. This provision need not be made where vehicles cannot mount the footway as in the case of a footway separated from the roadway by means of an insurmountable obstacle, such as, crash barrier, truss or a main girder.

Note : *A footway kerb shall be considered mountable by vehicles.*

206.5 The Pedestrian/Bicycle Railings/Parapets

The pedestrian/bicycle railings/parapets can be of a large variety of construction. The design loads for two basic types are given below:-

i) **Type** : Solid/partially filled in parapet continuously cantilevering along full length from deck level

Loading : Horizontal and vertical load of 150 kg/m acting simultaneously on the top level of the parapet.

ii) **Type** : Frame type with discrete vertical posts cantilevering from the curb/deck with minimum two rows of horizontal rails (third row bring the curb itself, or curb replaced by a low level 3rd rail). The rails may be simply supported or continuous over the posts

Loading : Each horizontal railing designed for horizontal and vertical load of 150 kg/m, acting simultaneously over the rail. The filler portion, supported between any two horizontal rails and vertical rails should be designed to resist horizontal load of 150 kg/m^2. The posts to resist horizontal load of 150 kg/m X spacing between posts in metres acting on top of the post.

206.6 Crash Barriers

Crash barriers are designed to withstand the impact of vehicles of certain weights at certain angle while travelling at the specified speed as given in **Table 9**. They are expected to guide the vehicle back on the road while keeping the level of damage to vehicle as well as to the barriers within acceptable limits.

Table 9: Application for design of Crash Barrier

Category	Application	Containment for
P-1: Normal Containment	Bridges carrying expressway, or equivalent	15 kN vehicle at 110 km/h, and 20^o angle of impact
P-2: Low Containment	All other bridges except bridge over railways	15 kN vehicle at 80 km/h and 20^o angle of impact
P-3: High Containment	At hazardous and high risk locations, over busy railway lines, complex interchanges, etc.	300 kN vehicle at 60 km/h and 20^o angle of impact

The barriers can be of rigid type, using cast-in-situ/precast reinforced concrete panels, or of flexible type, constructed using metallic cold-rolled and/or hot-rolled sections. The metallic type, called semi-rigid type, suffers large dynamic deflection of the order of 0.9 to 1.2 m due to impact, whereas the 'rigid' concrete type suffers comparatively negligible deflection. The efficacy of the two types of barriers is established on the basis of full size tests carried out by the laboratories specializing in such testing. Due to the complexities of the structural action, the value of impact force cannot be quantified.

Table 10: Minimum Design Resistance

S.No	Requirement	Types of Crash Barrier		
		P-1 In-situ/ Precast	**P-2 In-situ/ Precast**	**P-3 In-situ**
1)	Shape	Shape on traffic side to be as per IRC:5, or New Jersey (NJ) Type of 'F' Shape designated thus by AASHTO		
2)	Minimum grade of concrete	M40	M40	M40
3)	Minimum thickness of R C wall (at top)	175 mm	175 mm	250 mm
4)	Minimum moment of resistance at base of the wall [see note (i)] for bending in vertical plane with reinforcement adjacent to the traffic face [see note (ii)]	15 kNm/m	7.5 kNm/m	100 kNm/m for end section and 75 kNm/m for intermediate section [see note (iii)]
5)	Minimum moment of resistance for bending in horizontal plane with reinforcement adjacent to outer face [see note (ii)]	7.5 kNm/m	3.75 kNm/m	40 kNm/m
6)	Minimum moment of resistance of anchorage at the base of a precast reinforced concrete panel	22.5 kNm/m	11.25 kNm/m	Not applicable
7)	Minimum transverse shear resistance at vertical joints between precast panels, or at vertical joints made between lengths of in-situ crash barrier.	44 kN/m of joint	22.5 kN/m of joint	Not applicable
8)	Minimum height	900 mm	900 mm	1550 mm

Notes :

- i) *The base of wall refers to horizontal sections of the parapet within 300 mm above the adjoining paved surface level. The minimum moments of resistance shall reduce linearly from the base of wall value to zero at top of the parapet.*
- ii) *In addition to the main reinforcement, in items 4 & 5 above, distribution steel equal to 50 percent of the main reinforcement shall be provided in the respective faces.*
- iii) *For design purpose the crash barrier Type P-3 shall be divided into end sections extending a distance not greater than 3.0 m from ends of the crash barrier and intermediate sections extending along remainder of the crash barrier.*
- iv) *If concrete barrier is used as a median divider, the steel is required to be placed on both sides.*
- v) *In case of P-3 In-situ type, a minimum horizontal transverse shear resistance of 135 kN/m shall be provided.*

A certificate from such laboratory can be the only basis of acceptance of the semi-rigid type, in which case all the design details and construction details tested by the laboratory are to be followed in to without modifications and without changing relative strengths and positions of any of the connections and elements.

For the rigid type of barrier, the same method is acceptable. However, in absence of testing/test certificate, the minimum design resistance shown in **Table 10** should be built into the section

206.7 Vehicle barriers/pedestrian railing between footpath and carriageway

Where considerable pedestrian traffic is expected, such as, in/near townships, rigid type of reinforced concrete crash barrier should be provided separating the vehicular traffic from the same. The design and construction details should be as per Clause **206.6**. For any other type of rigid barrier, the strength should be equivalent to that of rigid RCC type.

For areas of low intensity of pedestrian traffic, semi-rigid type of barrier, which suffers large deflections, can be adopted.

207 Tramway Loading

207.1 When a road bridge carries tram lines, the live load due to the type of tram cars sketched in **Fig. 8** shall be computed and shall be considered to occupy a 3 m width of roadway

207.2 A nose to tail sequence of the tram cars or any other sequence which produces the heaviest stresses shall be considered in the design.

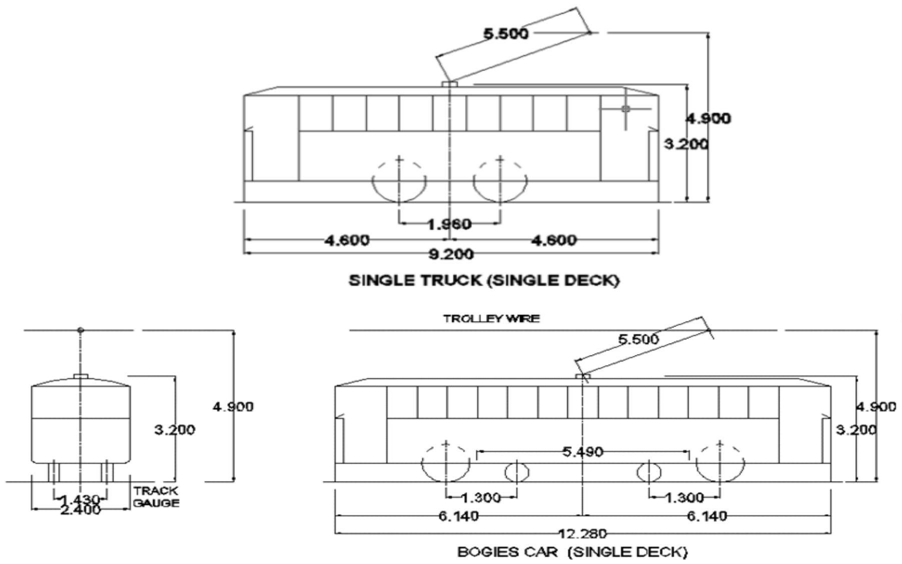


Fig. 8 Average Dimension of Tramway Rolling Stock (Clause 207.1)

Notes:

1) *Clearance between passing single deck bogie cars on straight tracks laid at standard 2.75 m track centres shall be 300 mm.*
2) *Clearance between passing double bogie cars on straight tracks laid at standard 2.75 m track centres shall be 450 mm.*
3) *Linear dimensions in meter.*

Table 11: ROLLING STOCK WEIGHT

Description	Loaded Weight (tonne)	Unloaded Weight (tonne)
Single truck (Single deck)	9.6	7.9
Bogie car (Single deck)	15.3	12.2
Bogie car (Double deck)	21.5	16.0

207.3 Stresses shall be calculated for the following two conditions and the maximum thereof considered in the design:-

a) Tram loading, followed and preceded by the appropriate standard loading specified in Clause **204.1** together with that standard loading on the traffic lanes not occupied by the tram car lines.

b) The appropriate standard loading specified in Clause **204.1** without any tram cars

208 IMPACT

208.1 Provision for impact or dynamic action shall be made by an increment of the live load by an impact allowance expressed as a fraction or a percentage of the applied live load.

208.2 For Class A or Class B Loading

In the members of any bridge designed either for Class A or Class B loading (vide Clause **204.1**), this impact percentage shall be determined from the curves indicated in **Fig.9**. The impact fraction shall be determined from the following equations which are applicable for spans between 3 m and 45 m

i. Impact factor fraction for reinforced concrete bridges $= \frac{4.5}{6+L}$

ii. Impact factor fraction for steel bridges $= \frac{9}{13.5+L}$

Where *L* is length in meters of the span as specified in Clause **208.5**

208.3 For Class AA Loading and Class 70R Loading

The value of the impact percentage shall be taken as follows:-

a) For spans less than 9 m :

For tracked vehicles	:	25 percent for spans upto 5 m linearly reducing to 10 percent for spans upto 9 m
For wheeled vehicles	:	25 Percent

b) For spans of 9 m or more :

i) **Reinforced Concrete Bridges**

1) Tracked Vehicles	:	10 percent upto a span of 40 m and in accordance with the curve in **Fig. 9** for spans in excess of 40 m
2) Wheeled Vehicles	:	25 percent for spans upto 12 m and in accordance with the curve in **Fig. 9** for spans in excess of 12 m.

ii) **Steel Bridges**

3) Tracked Vehicles	:	10 percent for all spans
4) Wheeled vehicles	:	25 percent for spans upto 23 m and in accordance with the curve indicated in **Fig. 9** for spans in excess of 23 m

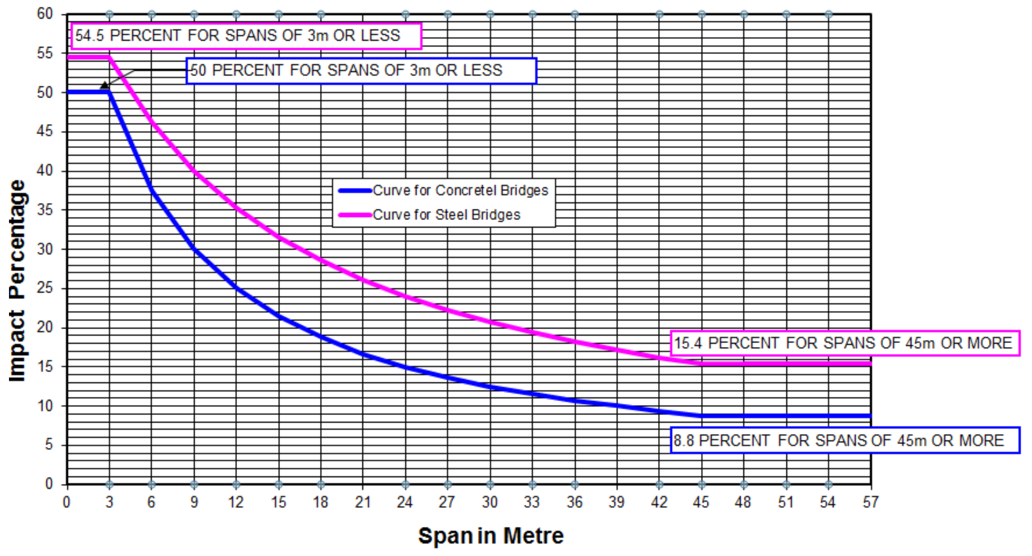


Fig. 9: Impact Percentage for Highway Bridges for Class A and Class B Loading (Clause 208.2)

208.4 No impact allowance shall be added to the footway loading specified in Clause **206.**

208.5 The span length to be considered for arriving at the impact percentages specified in Clause **208.2** and **208.3** shall be as follows:

a) For spans simply supported or continuous or for arches, the effective span on which the load is placed.

b) For bridges having cantilever arms without suspended spans the effective overhang of the cantilever arms reduced by 25 percent for loads on the cantilever arms and the effective span between supports for loads on the main span.

c) For bridges having cantilever arms with suspended span the effective overhang of the cantilever arm plus half the length of the suspended span for loads on the cantilever arm, the effective length of the suspended span for loads on the suspended span and the effective span between supports for load on the main span.

Note: *For individual members of a bridge, such as, a cross girder or deck slab, etc. the value of L mentioned in Clause **208.2** or the spans mentioned in clause **208.3** shall be the effective span of the member under consideration.*

208.6 In any bridge structure where there is a filling of not less than 0.6 m including the road crust, the impact percentage to be allowed in the design shall be assumed to be one-half of what is specified in Clauses **208.2** and **208.3**.

208.7 For calculating the pressure on the bearings and on the top surface of the bed blocks, full value of the appropriate impact percentage shall be allowed. But, for the design of piers abutments and structures, generally below the level of the top of the bed block, the appropriate impact percentage shall be multiplied by the factor given below:

a)	For calculating the pressure at the bottom surface of the bed block	:	0.5
b)	For calculating the pressure on the top 3 m of the structure below the bed block	:	0.5 decreasing uniformly to zero
c)	For calculating the pressure on the portion of structure more than 3 m below the bed block	:	zero

208.8 In the design of members subjected to among other stresses, direct tension, such as, hangers in a bowstring girder bridge and in the design of member subjected to direct compression, such as, spandrel columns or walls in an open spandrel arch, the impact percentage shall be taken the same as that applicable to the design of the corresponding member or members of the floor system which transfer loads to the tensile or compressive members in question.

208.9 These clauses on impact do not apply to the design of suspension bridges and foot over bridges. In cable suspended bridges and in other bridges where live load to dead load ratio is high, the dynamic effects such as vibration and fatigue shall be considered. For long span foot over bridges (with frequency less than 5 Hz and 1.5 Hz in vertical and horizontal direction) the dynamic effects shall be considered, if necessary, for which specialist literature may be referred.

209 WIND LOAD

209.1 This clause is applicable to normal span bridges with individual span length up to 150 m or for bridges with height of pier up to 100 m. For all other bridges including cable stayed bridges, suspension bridges and ribbon bridges specialist literature shall be used for computation of design wind load.

209.1.1 The wind pressure acting on a bridge depends on the geographical locations, the terrain of surrounding area, the fetch of terrain upwind of the site location, the local topography, the height of bridge above the ground, horizontal dimensions and cross-section of bridge or its element under consideration. The maximum pressure is due to gusts that cause local and transient fluctuations about the mean wind pressure.

All structures shall be designed for the wind forces as specified in Clause **209.3** and **209.4**. These forces shall be considered to act in such a direction that the resultant stresses in the member under consideration are maximum.

In addition to applying the prescribed loads in the design of bridge elements, stability against overturning, uplift and sliding due to wind shall be considered.

209.2 The wind speed at the location of bridge shall be based on basic wind speed map as shown in **Fig. 10**. The intensity of wind force shall be based on hourly mean wind speed and pressure as shown in **Table 12**. The hourly mean wind speed and pressure values given in **Table 12** corresponds to a basic wind speed of 33 m/s, return period of 100 years, for bridges situated in plain terrain and terrain with obstructions, with a flat topography. The hourly mean wind pressure shall be appropriately modified depending on the location of bridge for other basic wind speed as shown in **Fig. 10** and used for design (see notes below **Table 12**).

Table 12: Hourly Mean Wind Speed and Wind pressure
(For a Basic wind speed of 33 m/s as shown in Fig. 10)

H (m)	Bridge Situated in			
	Plain Terrain		Terrain with Obstructions	
	V_z (m/s)	P_z (N/m^2)	V_z (m/s)	P_z (N/m^2)
Up to 10 m	27.80	463.70	17.80	190.50
15	29.20	512.50	19.60	230.50
20	30.30	550.60	21.00	265.30
30	31.40	590.20	22.80	312.20
50	33.10	659.20	24.90	373.40
60	33.60	676.30	25.60	392.90
70	34.00	693.60	26.20	412.80
80	34.40	711.20	26.90	433.30
90	34.90	729.00	27.50	454.20
100	35.30	747.00	28.20	475.60

Where

H = the average height in metres of exposed surface above the mean retarding surface (ground or bed or water level)

V_z = Hourly mean speed of wind in m/s at height H

P_z = Horizontal wind pressure in N/m^2 at height H

Notes :

1) *Intermediate values may be obtained by linear interpolation.*

2) *Plain terrain refers to open terrain with no obstruction or with very well scattered obstructions having height up to 10 m. Terrain with obstructions refers to a terrain with numerous closely spaced structures, forests or trees upto 10 m in height with few isolated tall structures or terrain with large number of high closed spaced obstruction like structures, trees forests etc.*
3) *For other values of basic wind speed as indicated in **Fig. 10**, the hourly mean wind speed shall be obtained by multiplying the corresponding wind speed value by the ratio of basic wind speed at the location of bridge to the value corresponding to **Table 12**, (i.e., 33 m/sec.)*
4) *The hourly mean wind pressure at an appropriate height and terrain shall be obtained by multiplying the corresponding pressure value for base wind speed as indicated in **Table 12** by the ratio of square of basic wind speed at the location of wind to square of base wind speed corresponding to **Table 12** (i.e., 33 m/sec).*
5) *If the topography (hill, ridge escarpment or cliff) at the structure site can cause acceleration or funneling of wind, the wind pressure shall be further increased by 20 percent as stated in Note 4.*
6) *For construction stages, the hourly mean wind pressure shall be taken as 70 percent of the value calculated as stated in Note 4 and 5.*
7) *For the design of foot over bridges in the urban situations and in plain terrain, a minimum horizontal wind load of 1.5 kN/m^2 (150 kg/m^2) and 2 kN/m^2 (200 kg/m^2) respectively shall be considered to be acting on the frontal area of the bridge.*

209.3 Design Wind Force on Superstructure

209.3.1 The superstructure shall be designed for wind induced horizontal forces (acting in the transverse and longitudinal direction) and vertical loads acting simultaneously. The assumed wind direction shall be perpendicular to longitudinal axis for a straight structure or to an axis chosen to maximize the wind induced effects for a structure curved in plan.

209.3.2 The transverse wind force on a bridge superstructure shall be estimated as specified in Clause **209.3.3** and acting on the area calculated as follows:

a) **For a deck structure:**

The area of the structure as seen in elevation including the floor system and railing, less area of perforations in hand railing or parapet walls shall be considered. For open and solid parapets, crash barriers and railings, the solid area in normal projected elevation of the element shall be considered.

b) **For truss structures:**

Appropriate area as specified in **Annex C** shall be taken.

c) **For construction stages:**

The area at all stages of construction shall be the appropriate unshielded solid area of structure.

209.3.3 The transverse wind force F_T (in N) shall be taken as acting at the centroids of the appropriate areas and horizontally and shall be estimated from:

$$F_T = P_Z \times A_1 \times G \times C_D$$

where, P_Z is the hourly mean wind pressure in N/m^2 (see **Table 12**), A_1 is the solid area in m^2 (see Clause **209.3.2**), G is the gust factor and C_D is the drag coefficient depending on the geometric shape of bridge deck.

For highway bridges up to a span of 150 m, which are generally not sensitive to dynamic action of wind, gust factor shall be taken as 2.0.

The drag coefficient for slab bridges with width to depth ratio of cross-section, i.e $b/d \geq 10$ shall be taken as 1.1.

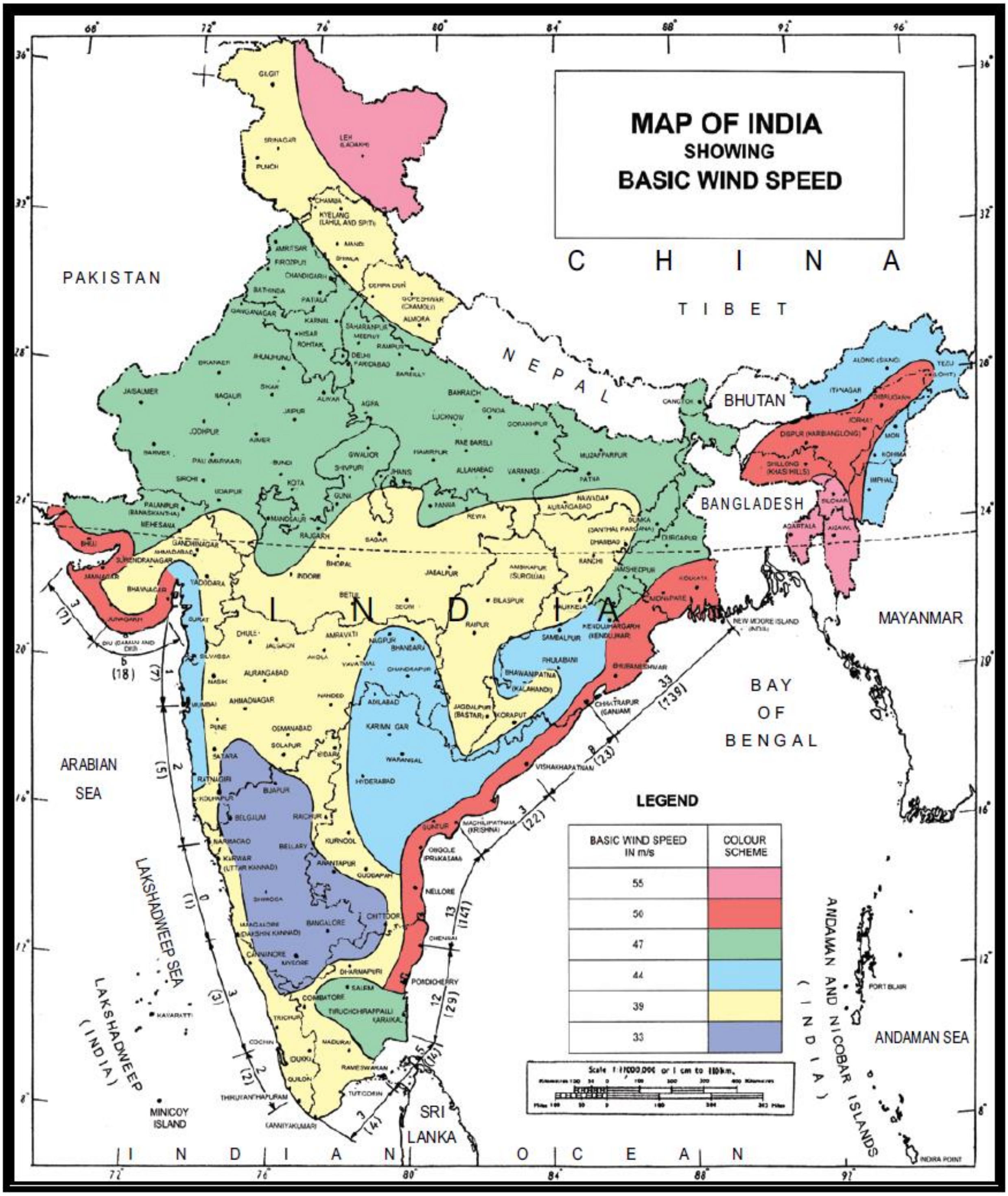
For bridge decks supported by single beam or box girder, C_D shall be taken as 1.5 for b/d ratio of 2 and as 1.3 if $b/d \geq 6$. For intermediate b/d ratios C_D shall be interpolated. For deck supported by two or more beams or box girders, where the ratio of clear distance between the beams of boxes to the depth does not exceed 7, C_D for the combined structure shall be taken as 1.5 times C_D for the single beam or box.

For deck supported by single plate girder it shall be taken as 2.2. When the deck is supported by two or more plate girders, for the combined structure C_D shall be taken as $2(1+c/20d)$, but not more than 4, where c is the centre to centre distance of adjacent girders, and d is the depth of windward girder.

For truss girder superstructure the drag coefficients shall be derived as given in **Annex C**.

For other type of deck cross-sections C_D shall be ascertained either from wind tunnel tests or, if available, for similar type of structure, specialist literature shall be referred to.

209.3.4 The longitudinal force on bridge superstructure F_L (in N) shall be taken as 25 percent and 50 percent of the transverse wind load as calculated as per Clause **209.3.3** for beam/ box/plate girder bridges and truss girder bridges respectively.



Based upon Survey of India Outline Map printed in 1993.

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.
The boundary of Meghalaya shown on this map is as interpreted from the North-Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.
Responsibility for correctness of internal details shown on the map rests with the publisher.
The state boundaries between Uttaranchal & Uttar Pradesh, Bihar & Jharkhand and Chhatisgarh & Madhya Pradesh have not been verified by Governments concerned.

Fig. 10: Basic Wind Speed in m/s (BASED ON 50-YEARS RETURN PERIOD)

The Fig. 10 have been reproduced in confirmation of Bureau of Indian Standards

209.3.5 An upward or downward vertical wind load F_V (in N) acting at the centroid of the appropriate area, for all superstructures shall be derived from:

$$F_V = P_Z \times A_3 \times G \times C_L$$

Where,

P_z	=	Hourly mean wind speed in N/m^2 at height H
A_3	=	Area in plain in m^2
C_L	=	Lift coefficient which shall be taken as 0.75 for normal type of slab, box, I-girder and plate girder bridges. For other type of deck cross-sections C_L shall be ascertained either from wind tunnel tests or, if available, for similar type of structure. Specialist literature shall be referred to.
G	=	Gust factor as defined in **209.3.3**

209.3.6 The transverse wind load per unit exposed frontal area of the live load shall be computed using the expression F_T given in Clause **209.3.3** except that C_D against shall be taken as 1.2. The exposed frontal area of live load shall be the entire length of the superstructure seen in elevation in the direction of wind as defined in clause or any part of that length producing critical response, multiplied by a height of 3.0 m above the road way surface. Areas below the top of a solid barrier shall be neglected.

The longitudinal wind load on live load shall be taken as 25 percent of transverse wind load as calculated above. Both loads shall be applied simultaneously acting at 1.5 m above the roadway.

209.3.7 The bridges shall not be considered to be carrying any live load when the wind speed at deck level exceeds 36 m/s.

209.3.8 In case of cantilever construction an upward wind pressure of $P_Z \times C_L \times G$ N/m^2 (see Clause **209.3.5** for notations) on bottom soffit area shall be assumed on stabilizing cantilever arm in addition to the transverse wind effect calculated as per Clause **209.3.3.** In addition to the above, other loads defined in Clause **218.3** shall also be taken into consideration.

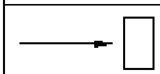
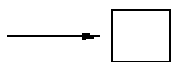
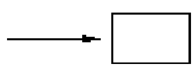
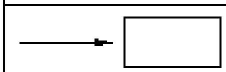
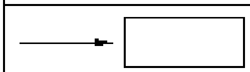
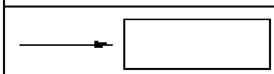
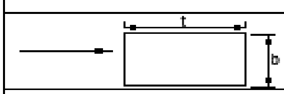
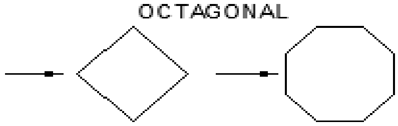
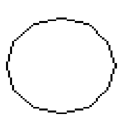
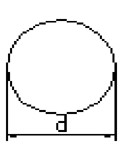
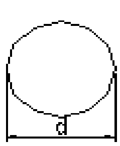
209.4 Design Wind Forces on Substructure

The substructure shall be designed for wind induced loads transmitted to it from the superstructure and wind loads acting directly on the substructure. Loads for wind directions both normal and skewed to the longitudinal centerline of the superstructure shall be considered.

F_T shall be computed using expression in Clause **209.3.3** with A_1 taken as the solid area in normal projected elevation of each pier. No allowance shall be made for shielding.

For piers, C_D shall be taken from **Table 13**. For piers with cross-section dissimilar to those given in **Table 13**, C_D shall be ascertained either from wind tunnel tests or, if available, for similar type of structure, specialist literature shall be referred to C_D shall be derived for each pier, without shielding.

Table 13 Drag Coefficients C_D for piers

PLAN SHAPE	$\frac{t}{b}$	C_D FOR PIER $\frac{\text{HEIGHT}}{\text{BREADTH}}$ RATIOS OF						
		1	2	4	6	10	20	40
	$\leq \frac{1}{4}$	1.3	1.4	1.5	1.6	1.7	1.9	2.1
	$\frac{1}{3}$ $\frac{1}{2}$	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	$\frac{2}{3}$	1.3	1.4	1.5	1.6	1.8	2.0	2.2
	1	1.2	1.3	1.4	1.5	1.6	1.8	2.0
	$1\frac{1}{2}$	1.0	1.1	1.2	1.3	1.4	1.5	1.7
	2	0.8	0.9	1.0	1.1	1.2	1.3	1.4
	3	0.8	0.8	0.8	0.9	0.9	1.0	1.2
	≥ 4	0.8	0.8	0.8	0.9	0.9	0.9	1.1
SQUARE OR OCTAGONAL 		1.0	1.1	1.2	1.3	1.4	1.4	1.4
 12 SIDE POLYGON		0.7	0.8	0.9	0.9	1.0	1.1	1.3
 CIRCLE WITH SMOOTH SURFACE WHERE $tV_z \geq 6\ m^2/s$		0.5	0.5	0.5	0.5	0.5	0.6	0.6
 CIRCLE WITH SURFACE WHERE $tV_z < 6\ m^2/s$ CIRCLE WITH ROUGH SURFACE OR WITH PROJECTION		0.7	0.7	0.8	0.8	0.9	1.0	1.2

Notes:

1) *For rectangular piers with rounded corners with radius r, the value of C_D derived from **Table 13** shall be multiplied by (1-1.5 r/b) or 0.5, whichever is greater.*
2) *For a pier with triangular nosing, C_D shall be derived as for the rectangle encompassing the outer edges of pier.*
3) *For pier tapering with height, C_D shall be derived for each of the unit heights into which the support has been subdivided. Mean values of t and b for each unit height shall be used to evaluate t/b. The overall pier height and mean breadth of each unit height shall be used to evaluate height/breadth.*
4) *After construction of the superstructure C_D shall be derived for height to breadth ratio of 40.*

209.5 Wind Tunnel Testing

Wind tunnel testing by established procedures shall be conducted for dynamically sensitive structures such as cable stayed, suspension bridges etc., including modeling of appurtenances.

210 HORIZONTAL FORCES DUE TO WATER CURRENTS

210.1 Any part of a road bridge which may be submerged in running water shall be designed to sustain safely the horizontal pressure due to the force of the current.

210.2 On piers parallel to the direction of the water current, the intensity of pressure shall be calculated from the following equation:

$$P = 52KV^2$$

where,

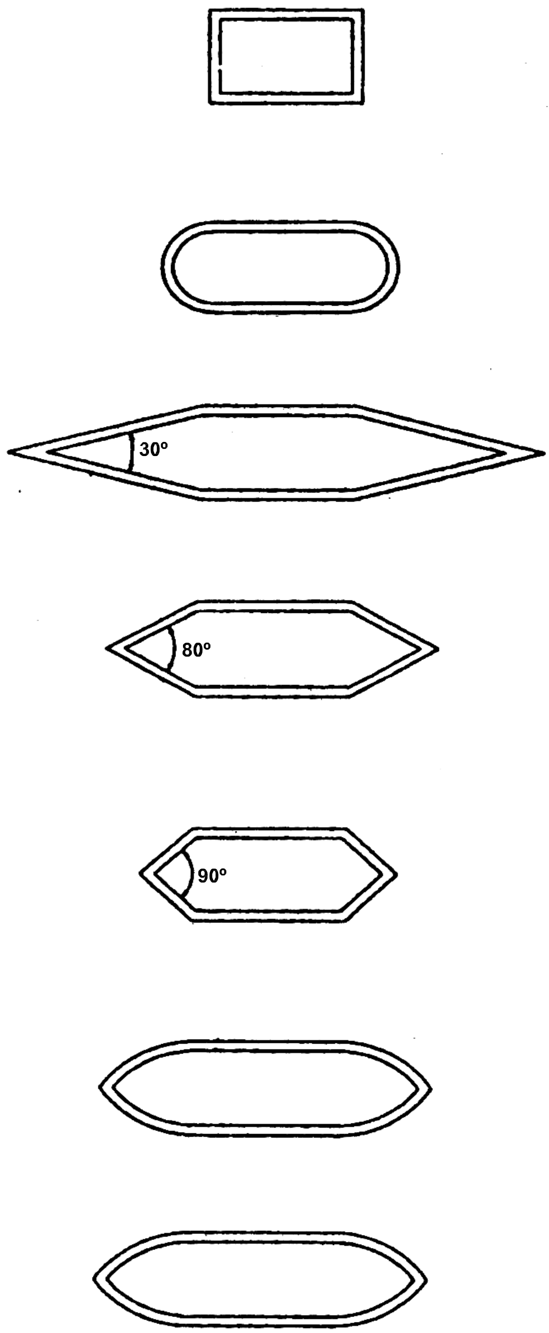
P = intensity of pressure due to water current, in kg/m^2

V = the velocity of the current at the point where the pressure intensity is being calculated, in metre per second, and

K = a constant having the following values for different shapes of piers illustrated in **Fig.11**

i)	Square ended piers (and for the superstructure)	1.50
ii)	Circular piers or piers with semi-circular ends	0.66
iii)	Piers with triangular cut and ease waters, the angle included between the faces being 30º or less	0.50
iv)	Piers with triangular cut and ease waters, the angle included between the faces being more than 30º but less than 60º	0.50 to 0.70

v)	Piers with triangular cut and ease waters, the angle included between the faces being more than 60º but less than 90º	0.70 to 0.90
vi)	Piers with cut and ease waters of equilateral arcs of circles	0.45
vii)	Piers with arcs of the cut and ease waters intersecting at 90º	0.50



Piers with square ends

Circular piers or piers with semi-circular ends

Piers with triangular cut and ease waters, the angle included between the faces being 30 degrees or less

Piers with triangular cut and ease waters, the angle included between the faces being more than 30 degrees but less than 60 degrees

Piers with triangular cut and ease waters, the angle included between the faces being 60 to 90 degrees

Piers with cut and ease waters of equilateral arcs of circles

Piers with arcs of the cut and ease waters intersecting at 90 degrees

Fig.11: Shapes of Bridge Piers (Clause 210.2)

210.3 The value of V^2 in the equation given in Clause **210.2** shall be assumed to vary linearly from zero at the point of deepest scour to the square of the maximum velocity at the free surface of water. The maximum velocity for the purpose of this sub-clause shall be assumed to be √2 times the maximum mean velocity of the current.

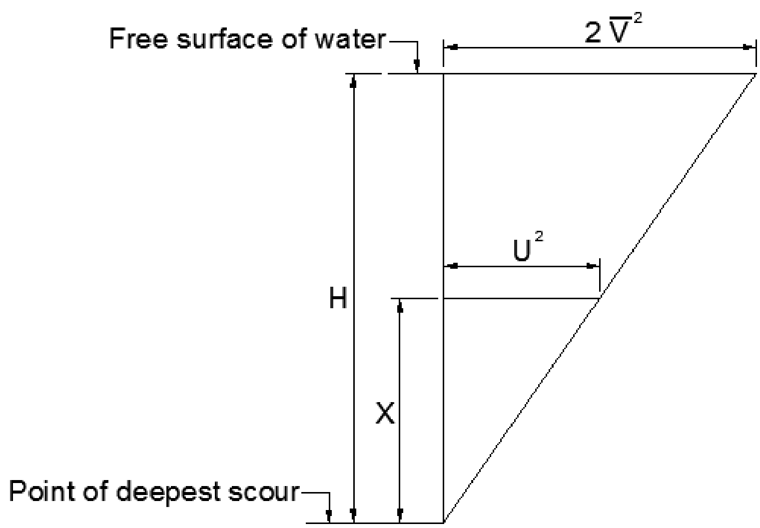


Fig. 12: Velocity Distribution

Square of velocity at a height 'X' from the point of deepest Scour $= U^2 = \frac{2\overline{V}^2 X}{H}$

Where, $\overline{V}$ is the maximum mean velocity.

210.4 When the current strikes the pier at an angle, the velocity of the current shall be resolved into two components – one parallel and the other normal to the pier.

a) The pressure parallel to the pier shall be determined as indicated in Clause **210.2** taking the velocity as the component of the velocity of the current in a direction parallel to the pier.

b) The pressure of the current, normal to the pier and acting on the area of the side elevation of the pier, shall be calculated similarly taking the velocity as the component of the velocity of the current in a direction normal to the pier, and the constant K as 1.5, except in the case of circular piers where the constant shall be taken as 0.66.

210.5 To provide against possible variation of the direction of the current from the direction assumed in the design, allowance shall be made in the design of piers for an extra variation in the current direction of 20 degrees that is to say, piers intended to be parallel to the direction of current shall be designed for a variation of 20 degrees from the normal direction of current and piers originally intended to be inclined at θ degree to the direction of the current shall be designed for a current direction inclined at (20±θ) degrees to the length of the pier.

210.6 In case of a bridge having a pucca floor or having an inerodible bed, the effect of cross-currents shall in no case be taken as less than that of a static force due to a difference of head of 250 mm between the opposite faces of a pier.

210.7 When supports are made with two or more piles or trestle columns, spaced closer than three times the width of piles/columns across the direction of flow, the group shall be treated as a solid rectangle of the same overall length and width and the value of K taken as 1.25 for calculating pressures due to water currents, both parallel and normal to the pier. If such piles/columns are braced, then the group should be considered as a solid pier, irrespective of the spacing of the columns.

211 LONGITUDINAL FORCES

211.1 In all road bridges, provision shall be made for longitudinal forces arising from any one or more of the following causes:

a) Tractive effort caused through acceleration of the driving wheels;

b) Braking effect resulting from the application of the brakes to braked wheels; and

c) Frictional resistance offered to the movement of free bearings due to change of temperature or any other cause.

Note : *Braking effect is invariably greater than the tractive effort*.

211.2 The braking effect on a simply supported span or a continuous unit of spans or on any other type of bridge unit shall be assumed to have the following value:

a) In the case of a single lane or a two lane bridge : twenty percent of the first train load plus ten percent of the load of the succeeding trains or part thereof, the train loads in one lane only being considered for the purpose of this sub- clause. Where the entire first train is not on the full span, the braking force shall be taken as equal to twenty percent of the loads actually on the span or continuous unit of spans.

b) In the case of bridges having more than two-lanes: as in (a) above for the first two lanes plus five per cent of the loads on the lanes in excess of two.

Note : *The loads in this Clause shall not be increased on account of impact.*

211.3 The force due to braking effect shall be assumed to act along a line parallel to the roadway and 1.2 m above it. While transferring the force to the bearings, the change in the vertical reaction at the bearings should be taken into account.

211.4 The distribution of longitudinal horizontal forces among bridge supports is effected by the horizontal deformation of bridges, flexing of the supports and rotation of the foundations. For spans resting on stiff supports, the distribution may be assumed as given below in Clause **211.5.** For spans resting on flexible supports, distribution of horizontal forces may be carried out according to procedure given below in Clause **211.6**.

211.5 Simply supported and continuous spans on unyielding supports

211.5.1 *Simply supported spans on unyielding supports*

211.5.1.1 For a simply supported span with fixed and free bearings (other than elastomeric type) on stiff supports, horizontal forces at the bearing level in the longitudinal direction shall be greater of the two values given below:

	Fixed bearing	**Free bearing**
i)	$F_h - \mu (R_g+R_q)$	$\mu (R_q+R_g)$
or		
ii)	$\frac{F_h}{2} + \mu (R_g+R_q)$	$\mu (Rg+R_q)$

Where

F_h = Applied Horizontal force

R_g = Reaction at the free end due to dead load

R_q = Reaction at free end due to live load

μ = Coefficient of friction at the movable bearing which shall be assumed to have the following values:

i)	For steel roller bearings	0.03
ii)	For concrete roller bearings	0.05
iii)	For sliding bearings:	
a)	Steel on cast iron or steel on steel	0.4
b)	Gray cast iron	0.3
	Gray cast iron (Mechanite)	
c)	Concrete over concrete with bitumen layer in between	0.5

	d)	Teflon on stainless steel	0.03 and 0.05 Whichever is governing

Notes:

a) *For design of bearing, the corresponding forces may be taken as per relevant IRC Codes.*

b) *Unbalanced dead load shall be accounted for properly. The structure under the fixed bearing shall be designed to withstand the full seismic and design braking/tractive force.*

211.5.1.2 In case of simply supported small spans upto 10 m resting on unyielding supports and where no bearings are provided, horizontal force in the longitudinal direction at the bearing level shall be

$$= \frac{F_h}{2} \text{ or } \mu R_g \text{ whichever is greater}$$

211.5.1.3 For a simply supported span siting on identical elastomeric bearings at each end resting on unyielding supports. Force at each end

$$= \frac{F_h}{2} + V_r\, l_{tc}$$

Where

V_r = Shear rating of the elastomeric bearings

l_{tc} = Movement of deck above bearing, other than that due to applied force

211.5.1.4 The substructure and foundation shall also be designed for 10 percent variation in movement of the span of either side.

211.5.2 For continuous bridges with one fixed bearing or other free bearings on unyielding support refer **Table 14** below.

Table 14: Horizontal forces at Bearing Level for Continuous spans on unyielding supports

Fixed bearing	Free bearing
Case-I	
($\mu R - \mu L$) +ve F_h acting in +ve direction (a) If, $F_h > 2\mu R$ $F_h - (\mu R + \mu L)$ (b) If, $F_h < 2\mu R$ $\frac{F_h}{1+\sum n_R} + (\mu R - \mu L)$	μRx
Case-II	
($\mu R - \mu L$) +ve F_h acting in -ve direction (c) If, $F_h > 2\mu L$ $F_h - (\mu R + \mu L)$ (d) If, $F_h < 2\mu L$ $\frac{F_h}{1+\sum n_R} - (\mu R - \mu L)$	μRx

Where

n_L or n_R = number of free bearings to the left or right of fixed bearings, respectively

μL or μR = The total horizontal force developed at the free bearings to the left or the right of the fixed bearing respectively

μR_x = the net horizontal force developed at any one of the free bearings considered to the left or right of the fixed bearings

Note : *In seismic areas, the fixed bearing shall also be checked for full seismic force and braking/ tractive force. The structure under the fixed bearing shall be designed to withstand the full seismic and design braking/tractive force.*

211.6 Simply Supported and Continuous Spans on Flexible Supports

211.6.1 Shear rating of a support is the horizontal force required to move the top of the support through a unit distance taking into account horizontal deformation of the bridges, flexibility of the support and rotation of the foundation. The distribution of 'applied' longitudinal horizontal forces (e.g., braking, seismic, wind etc.) depends solely on shear ratings of the supports and may be estimated in proportion to the ratio of individual shear ratings of a support to the sum of the shear ratings of all the supports.

211.6.2 The distribution of self-induced horizontal force caused by deck movement (owing to temperature, shrinkage, creep, elastic shortening, etc.) depends not only on shear ratings of the supports but also on the location of the 'zero' movement point in the deck. The shear rating of the supports, the distribution of applied and self-induced horizontal force and the determination of the point of zero movement may be made as per recognized theory for which reference may be made to publications on the subjects.

211.7 The effects of braking force on bridge structures without bearings, such as, arches, rigid frames, etc., shall be calculated in accordance with approved methods of analysis of indeterminate structures.

211.8 The effects of the longitudinal forces and all other horizontal forces should be calculated upto a level where the resultant passive earth resistance of the soil below the deepest scour level (floor level in case of a bridge having pucca floor) balances these forces.

212 CENTRIFUGAL FORCES

212.1 Where a road bridge is situated on a curve, all portions of the structure affected by the centrifugal action of moving vehicles are to be proportioned to carry safely the stress induced by this action in addition to all other stress to which they may be subjected.

212.2 The centrifugal force shall be determined from the following equation:

$$C = \frac{WV^2}{127R}$$

Where

C = Centrifugal force acting normally to the traffic (1) at the point of action of the wheel loads or (2) uniformly distributed over every metre length on which a uniformly distributed load acts, in tonnes.

W = Live load (1) in case of wheel loads, each wheel load being considered as acting over the ground contact length specified in Clause 204, in tonnes, and (2) in case of a uniformly distributed live load, in tonnes per linear metre

V = The design speed of the vehicles using the bridge in km per hour, and

R = The radius of curvature in metres

212.3 The centrifugal force shall be considered to act at a height of 1.2 m above the level of the carriageway.

212.4 No increase for impact effect shall be made on the stress due to centrifugal action.

212.5 The overturning effect of the centrifugal force on the structure as a whole shall also be duly considered.

213 BUOYANCY

213.1 In the design of abutments, especially those of submersible bridges, the effects of buoyancy shall also be considered assuming that the fill behind the abutments has been removed by scour.

213.2 To allow for full buoyancy, a reduction shall be made in the gross weight of the member affected by reducing its density by the density of the displaced water.

Note:

1) *The density of water may be taken as 1.0 t/m^3*
2) *For artesian condition, HFL or actual water head, whichever is higher, shall be considered for calculating the uplift.*

213.3 In the design of submerged masonry or concrete structures, the buoyancy effect through pore pressure may be limited to 15 percent of full buoyancy.

213.4 In case of submersible bridges, the full buoyancy effect on the superstructure shall be taken into consideration.

214 EARTH PRESSURE

214.1 Lateral Earth Pressure

Structure designed to retain earth fills shall be proportioned to withstand pressure calculated in accordance with any rational theory. Coulomb's theory shall be acceptable for non-cohesive soils. For cohesive soil Coulomb's theory is applicable with Bell's correction. For calculating the earth pressure at rest Rankine's theory shall be used.

Earth retaining structures shall, however, be designed to withstand a horizontal pressure not less than that exerted by a fluid weighing 480 kg/m^3 unless special methods are adopted to eliminate earth pressure.

The provisions made under this clause are not applicable for design of reinforced soil structures, diaphragm walls and sheet piles etc., for which specialist literature shall be referred.

214.1.1 Lateral Earth Pressure under Non-Seismic Condition for Non-Cohesive Soil

214.1.1.1 Active pressure

The coefficient of active earth pressure K_a estimated based on Coulomb earth pressure theory is as shown in **Fig. 13A**

$$K_a = \frac{\cos^2(\phi-\alpha)}{\cos^2\alpha\ \cos(\delta+\alpha)} \times \left[\frac{1}{1+\left\{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\cos(\alpha-\beta)\cos(\delta+\alpha)}\right\}^{1/2}}\right]^2$$

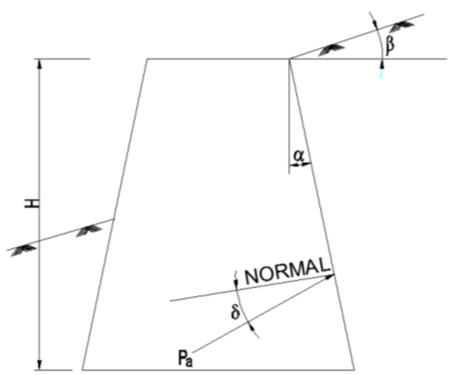


Fig.13A: Diagram for Active Earth Pressure

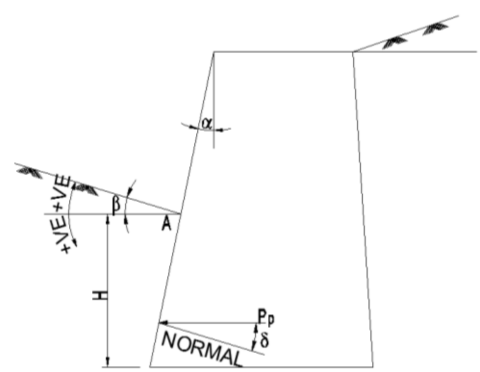


Fig.13B: Diagram for Passive Earth Pressure

Where,

ϕ	=	Angle of internal friction of soil
α	=	Angle which earth face of the wall makes with the vertical
β	=	Slope of earth fill
δ	=	Angle of friction between the earth and earth fill should be equal to 2/3 of ϕ subjected to a maximum of 22.5^0

Point of Application: The centre of pressure exerted by the backfill, when considered dry, is located at an elevation of 0.42 of the height of the wall above the base and 0.33 of height of wall when considered wet.

214.1.1.2 Passive pressure

The coefficient of active earth pressure K_p estimated based on Coulomb earth pressure theory is as shown in **Fig. 13B**

$$K_p = \frac{\cos^2(\phi + \alpha)}{\cos^2 \alpha \cos(\delta - \alpha)} \times \left[\frac{1}{1 - \left\{ \frac{\sin(\phi + \delta)\sin(\phi + \beta)}{\cos(\alpha - \beta)\cos(\delta - \alpha)} \right\}^{1/2}} \right]^2$$

Where

ϕ = Angle of internal friction of soil

α = Angle which earth face of the wall makes with the vertical

β = Slope of earth fill

δ = Angle of friction between the earth and earth fill should be equal to 2/3 of ϕ subjected to a maximum of 22.5^0

Point of Application: The centre of pressure exerted by the backfill is located at an elevation of 0.33 of the height of the wall above the base, both for wet and dry back fills.

214.1.1.3 Live Load Surcharge

A live load surcharge shall be applied on abutments and retaining walls. The increase in horizontal pressure due to live load surcharge shall be estimated as

$$\Delta = \mathrm{k} \text{ x } \gamma \text{ x } \mathrm{h}_{eq}$$

Where

k = Coefficient of lateral earth pressure

γ = Density of soil

h_{eq} = Equivalent height of soil for vehicular loading which shall be 1.2 m

The live load surcharge need not be considered for any earth retaining structure beyond 3 m from edge of formation width.

214.1.2 Lateral earth pressure under Seismic conditions for non –cohesive soil

The pressure from earthfill behind abutments during an earthquake shall be as per the following expression.

214.1.2.1 Active Pressure due to Earthfill

The total dynamic force in kg/m length wall due to dynamic active earth pressure shall be:

$$(P_{aw})\mathrm{dyn} = \frac{1}{2}\mathrm{wh}^2 C_a$$

Where

C_a = Coefficient of dynamic active earth pressure

w = Unit weight of soil in kg/m^3

h = Height of wall in metre and

$$C_a = \frac{(1 \pm A_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda\ \cos^2\alpha\cos(\delta + \alpha + \lambda)} \times \left[\frac{1}{1 + \left\{\frac{\sin(\phi+\delta)\sin(\phi-\beta-\lambda)}{\cos(\alpha-\beta)\cos(\delta+\alpha+\lambda)}\right\}^{1/2}}\right]^2 \qquad 214.1.2.\ (a)$$

Where

A_v = Vertical Seismic coefficient

ϕ = Angle of internal friction of soil

λ = $tan^{-1}\dfrac{A_h}{1 \pm A_v}$

α = Angle which earth face of the wall makes with the vertical

β = Slope of earth fill

δ = Angle of friction between the wall and earth fill and

A_h = Horizontal seismic coefficient, shall be taken as (Z/2), for zone factor Z, refer **Table 16**

For design purpose, the greater value of Ca shall be taken, out of its two values corresponding to ± A_v.

Point of application - From the total pressure computed as above subtract the static active pressure obtained by putting $A_h = A_v = \lambda = 0$ in the expression given in equation **214.1.2 (a)**. The remainder is the dynamic increment. The static component of the total pressure shall be applied at an elevation h/3 above the base of the wall. The point of application of the dynamic increment shall be assumed to be at mid-height of the wall.

214.1.2.2 Passive Pressure due to Earthfill

The total dynamic force in kg/m length wall due to dynamic Passive earth pressure shall be:

$$(P_{Pw})dyn = \frac{1}{2}wh^2C_p$$

Where

C_p = Coefficient of dynamic Passive Earth Pressure

$$\frac{(1 \pm A_v)\cos^2(\phi + \alpha - \lambda)}{\cos\lambda\ \cos^2\alpha\cos(\delta - \alpha + \lambda)} \times \left[\frac{1}{1 - \left\{\frac{\sin(\phi+\delta)\sin(\phi+\beta-\lambda)}{\cos(\alpha-\beta)\cos(\delta-\alpha+\lambda)}\right\}^{1/2}}\right]^2 \qquad 214.1.2.\ (b)$$

w, h, ϕ, α, β, and δ are as defined in above and

$$\lambda = \tan^{-1}\frac{A_h}{1 \pm A_v}$$

Point of application – From the static passive pressure obtained by putting A_h=A_v=λ=0 in the expression given in equation **214.1.2(b)**, subtract the total pressure computed as above. The remainder is the dynamic decrement. The static component of the total pressure shall be applied at an elevation h/3 above the base of the wall. The point of application of the dynamic decrement shall be assumed to be at an elevation 0.5h above the base of the wall.

214.1.2.3 Active Pressure due to Uniform Surcharge

The active pressure against the wall due to a uniform surcharge of intensity q per unit area of the inclined earthfill surface shall be:

$$\left(P_{aq}\right)_{dyn} = \frac{\mathrm{qh}\cos\alpha}{\cos(\alpha-\beta)}\mathrm{C_a} \qquad 214.1.2(c)$$

Point of application - The dynamic increment in active pressures due to uniform surcharge shall be applied at an elevation of 0.66h above the base of the wall, while the static component shall be applied at mid-height of the wall.

214.1.2.4 Passive Pressure due to Uniform Surcharge

The passive pressure against the wall due to a uniform surcharge of intensity q per unit area of the inclined earthfill shall be:

$$\left(P_{pq}\right)_{dyn} = \frac{qh\ \cos\alpha}{\cos(\alpha-\beta)}\mathrm{C_p} \qquad 214.1.2(d)$$

Point of application - The dynamic decrement in passive pressures due to uniform surcharge shall be applied at an elevation of 0.66 h above the base of the-walls while the static component shall be applied at mid-height of the wall.

214.1.2.5 Effect of Saturation on Lateral Earth Pressure

For submerged earth fill, the dynamic increment (or decrement) in active and passive earth pressure during earthquakes shall be found from expressions given in **214.1.2 (a)** and **214.1.2(b)** above with the following modifications:

a) The value of δ shall be taken as 1/2 the value of δ for dry backfill.

b) The value of λ_s shall be taken as follows:

$$\lambda s = tan^{-1}\frac{W_s}{W_s - 1} \times \frac{A_h}{1 \pm A_v} \qquad 214.1.2\ (e)$$

Where

W_s	=	Saturated unit weight of soil in gm/cc
A_h	=	Horizontal seismic coefficient
A_v	=	Vertical Seismic coefficient

c) Buoyant unit weight shall be adopted.

d) From the value of earth pressure found out as above, subtract the value of earth pressure determined by putting $A_h = A_V = \lambda_s = 0$ but using buoyant unit weight. The remainder shall be dynamic increment.

214.1.3 At-Rest Lateral Earth Pressure Coefficient

The coefficient of at-rest earth pressure shall be taken as

$$K_0 = 1 - \sin\phi$$

Where

ϕ	=	Coefficient of internal friction of soil
K_0	=	Coefficient of earth pressure at rest

Walls that have of no movement should be designed for "at-rest" earth pressure. Typical examples of such structures are closed box cell structures.

Point of application: The centre of pressure exerted by the backfill is located at an elevation of 0.33 of the height of the wall.

214.1.4 Active and Passive Lateral Earth Pressure Coefficients for cohesive (C–ϕ) soil – non Seismic condition

The active and passive pressure coefficients (K_a and K_p) for lateral active and passive earth pressure shall be calculated based on Coulomb's formula taking into consideration of wall friction. For cohesive soils, the effect of 'C' shall be added as per procedure given by Bell.

For cohesive soils, active pressure shall be estimated by

$$P_a = K_a \gamma z - 2C\sqrt{K_a}$$

For cohesive soils, passive pressure shall be estimated by

$$P_p = K_p \gamma z + 2C\sqrt{K_p}$$

The value of angle of wall friction may be taken as 2/3rd of ϕ, the angle of repose, subject to limit of 22½ degree.

Where

P_a	=	Active lateral earth pressure
P_p	=	Passive lateral earth pressure
K_a	=	Active coefficient of lateral earth pressure
K_p	=	Passive coefficient of lateral earth pressure
γ	=	Density of soil (For saturated earth fill, saturated unit weight of soil shall be adopted)
z	=	Depth below surface of soil
C	=	Soil cohesion

Point of Application – The centre of earth pressure exerted shall be located at 0.33 of height for triangular variation of pressure and 0.5 of height for rectangular variation of pressure.

214.1.5 Earth Pressure for Partially Submerged Backfills

The ratio of lateral dynamic increment in active pressure due to backfill to the vertical pressures at various depths along the height of wall may be taken as shown in **Fig. 14 a.**

The pressure distribution of dynamic increment in active pressures due to backfill may be obtained by multiplying the vertical effective pressures by the coefficients in **Fig. 14b** at corresponding depths.

Lateral dynamic increment due to surcharge multiplying with q is shown in **Fig. 14b.**

A similar procedure as in **214.1.5** may be utilized for determining the distribution of dynamic decrement in passive pressures. Concrete or masonry inertia forces due to horizontal and vertical earthquake accelerations are the products of the weight of wall and the horizontal and vertical seismic coefficients respectively.

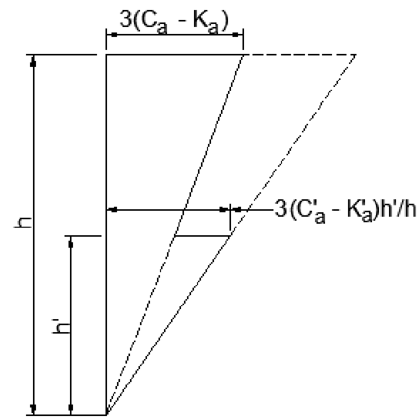


Fig.14a

Distribution of the ratio = $\dfrac{\text{Lateral Dynamic Increment due to back fill with height of wall}}{\text{Vertical Effective Pressure}}$

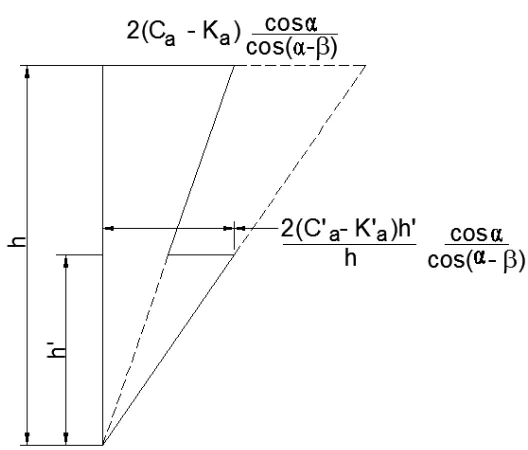


Fig.14b

Distribution of the ratio = $\dfrac{\text{Lateral Dynamic Increment due to surcharge with height of wall}}{\text{Surcharge intensity}}$

Note:

*C_a is computed as in **214.1.2 (a)** for dry (moist) saturated backfills*

*C_a^1 is computed as in **214.1.2 (a)** and **214.1.2 (e)** for submerged backfills*

K_a^1 is the value of C_a when $A_h = A_v = \lambda = 0$

K_a^1 is the value of C_a^1 when $A_h = A_v = \lambda = 0$

h^1 is the height of submergence above the base of the wall

214.1.6 Earth Pressure for Integral Bridges

For calculation of earth pressure on bridge abutments in integral bridges, the specialist literature shall be referred.

214.2 Reinforced concrete approach slab with 12 mm dia 150 mm c/c in each direction both at top and bottom as reinforcement in M30 grade concrete covering the entire width of the roadway, with one end resting on the structure designed to retain earth and extending for a length of not less than 3.5 m into the approach shall be provided.

214.3 Design shall be provided for the thorough drainage of backfilling materials by means of weep holes and crushed rock or gravel drains; or pipe drains, or perforated drains. Where such provisions are not provided, the hydrostatic pressures shall also be considered for the design.

214.4 The pressure of submerged soils (not provided with drainage arrangements) shall be considered as made up of two components:

a) Pressure due to the earth calculated in accordance with the method laid down in Clause **214.1.1**, unit weight of earth being reduced for buoyancy, and

b) Full hydrostatic pressure of water

215 TEMPERATURE

215.1 General

Daily and seasonal fluctuations in shade air temperature, solar radiation, etc. cause the following:

a) Changes in the overall temperature of the bridge, referred to as the effective bridge temperature. Over a prescribed period there will be a minimum and a maximum, together with a range of effective bridge temperature, resulting in loads and/or load effects within the bridge due to:

   i) Restraint offered to the associated expansion/contraction by the form of construction (e.g., portal frame, arch, flexible pier, elastomeric bearings) referred to as temperature restraint; and

   ii) Friction at roller or sliding bearings referred to as frictional bearing restraint;

b) Differences in temperature between the top surface and other levels through the depth of the superstructure, referred to as temperature difference and resulting in associated loads and/or load effects within the structure.

Provisions shall be made for stresses or movements resulting from variations in the temperature.

215.2 Range of effective bridge temperature

Effective bridge temperature for the location of the bridge shall be estimated from the isotherms of shade air temperature given on **Figs. 15 and 16.** Minimum and maximum effective bridge temperatures would be lesser or more respectively than the corresponding minimum and maximum shade air temperatures in concrete bridges. In determining load effects due to temperature restraint in concrete bridges the effective bridge temperature when the structure is effectively restrained shall be taken as datum in calculating the expansion up to the maximum effective bridge temperature and contraction down to the minimum effective bridge temperature.

The bridge temperature when the structure is effectively restrained shall be estimated as given in **Table 15** below.

Table 15: Range of Bridge Temperature

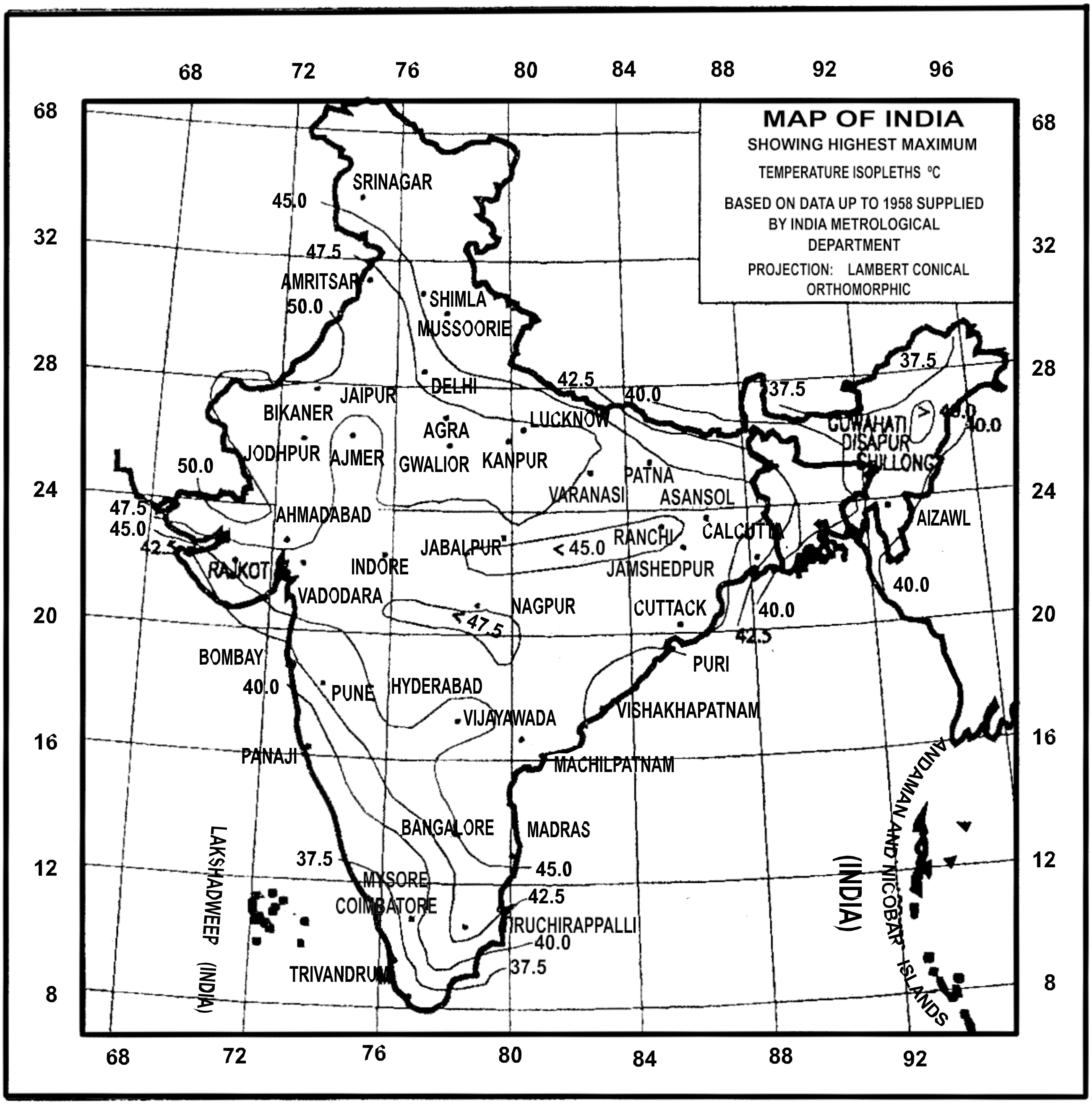
Bridge location having difference between maximum and minimum air shade temperature	Bridge temperature to be assumed when the structure is effectively restrained
> 20ºC	Mean of maximum and minimum air shade temperature ± 10ºC whichever is critical
< 20ºC	Mean of maximum and minimum air shade temperature ± 5ºC whichever is critical

For metallic structures the extreme range of effective bridge temperature to be considered in the design shall be as follows:

1) Snowbound areas from – 35ºC to + 50ºC
2) For other areas (Maximum air shade temperature + 15ºC) to (minimum air shade temperature – 10ºC). Air shade temperatures are to be obtained from **Figs. 15 and 16.**

215.3 Temperature Differences

Effect of temperature difference within the superstructure shall be derived from positive temperature differences which occur when conditions are such that solar radiation and other effects cause a gain in heat through the top surface of the superstructure. Conversely, reverse temperature differences are such that heat is lost from the top surface of the bridge deck as a result of re-radiation and other effects. Positive and reverse temperature differences for the purpose of design of concrete bridge decks shall be assumed as shown in **Fig. 17a**. These design provisions are applicable to concrete bridge decks with about 50 mm wearing surface. So far as steel and composite decks are concerned, **Fig. 17b** may be referred for assessing the effect of temperature gradient.



The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.

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Fig. 15 Chart Showing Highest Maximum Temperature

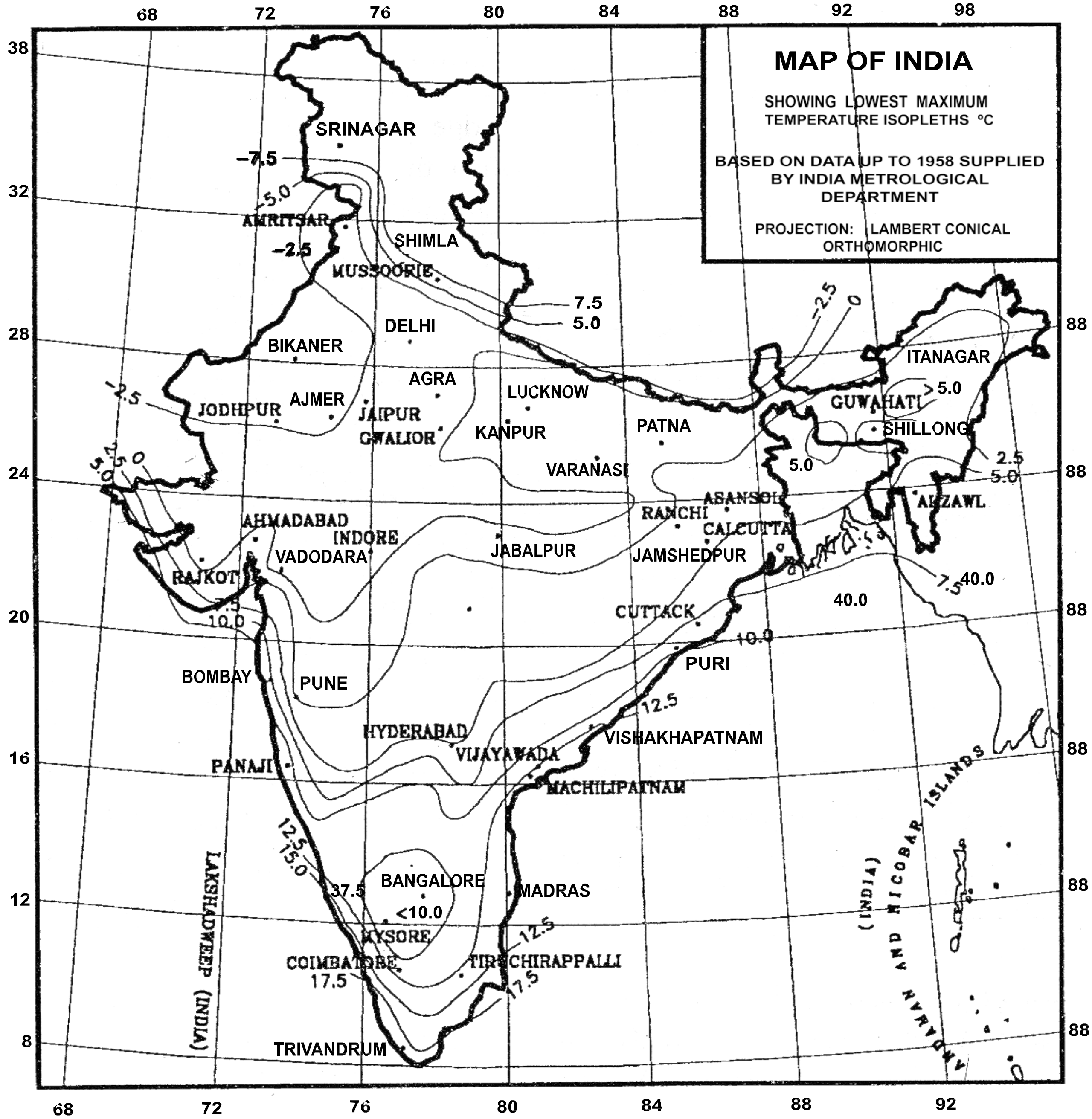

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Fig. 16 **Chart Showing Lowest Minimum Temperature**

215.4 Material Properties

For the purposes of calculating temperature effects, the coefficient of thermal expansion for RCC, PSC and steel structure may be taken as $12.0 \times 10^{-6}/^{0}C$.

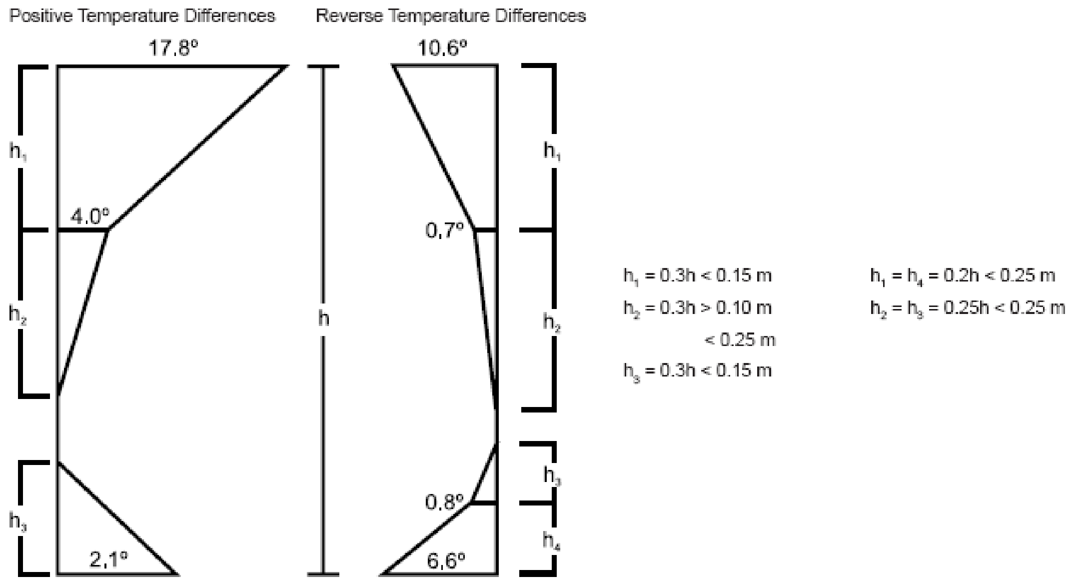
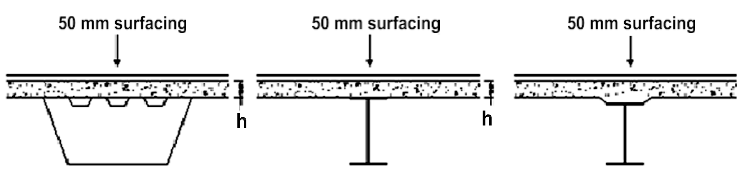
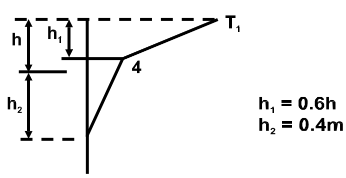
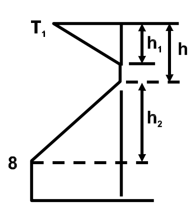


Fig. 17a: Design Temperature Differences for Concrete Bridge Decks





H (m)	T_1 (°C)
0.2	18
0.3	20.5



(m)	T_1 (°C)
0.2	4.4

Fig. 17b: Temperature Differences Across Steel and Composite Section

Note : *For intermediate slab thickness, T_1 may be interpolated.*

216 DEFORMATION EFFECTS (for Steel Bridges only)

216.1 A deformation effects is defined as the bending stress in any member of an open web-girder caused by the vertical deflection of the girder combined with the rigidity of the joints.

216.2 All steel bridges shall be designed, manufactured and erected in a manner such that the deformation effects are reduced to a minimum. In the absence of calculation, deformation stresses shall be assumed to be not less than 16 percent of the dead and live loads stresses.

216.3 In prestressed girders of steel, deformation effects may be ignored.

217 SECONDARY EFFECTS

217.1 a) **Steel Structures:** Secondary effects are additional effects brought into play due to the eccentricity of connections, floor beam loads applied at intermediate points in a panel, cross girders being connected away from panel points, lateral wind loads on the end-posts of through girders etc., and effects due to the movement of supports

b) R**einforced Concrete Structures:** Secondary effects are additional effects brought into play due either to the movement of supports or to the deformations in the geometrical shape of the structure or its member, resulting from causes, such as, rigidity of end connection or loads applied at intermediate points of trusses or restrictive shrinkage of concrete floor beams.

217.2 All bridges shall be designated and constructed in a manner such that the secondary effects are reduced to a minimum and they shall be allowed for in the design.

217.3 For reinforced concrete members, the shrinkage coefficient for purposes of design may be taken as 2×10^{-4}

218 ERECTION EFFECTS AND CONSTRUCTION LOADS

218.1 The effects of erection as per actual loads based on the construction programme shall be accounted for in the design. This shall also include the condition of one span being completed in all respects and the adjacent span not in position. However, one span dislodged condition need not be considered in the case of slab bridge not provided with bearings.

218.2 Construction loads are those which are incident upon a structure or any of its constituent components during the construction of the structures.

A detailed construction procedure associated with a method statement shall be drawn up during design and considered in the design to ensure that all aspects of stability and strength of the structure are satisfied.

218.3 Examples of Typical Construction Loadings are given below. However, each individual case shall be investigated in complete detail.

Examples:

a) Loads of plant and equipment including the weight handled that might be incident on the structure during construction.

b) Temporary super-imposed loading caused by storage of construction material on a partially completed a bridge deck.

c) Unbalanced effect of a temporary structure, if any, and unbalanced effect of modules that may be required for cantilever segmental construction of a bridge.

d) Loading on individual beams and/or completed deck system due to travelling of a launching truss over such beams/deck system.

e) Thermal effects during construction due to temporary restraints.

f) Secondary effects, if any, emanating from the system and procedure of construction.

g) Loading due to any anticipated soil settlement.

h) Wind load during construction as per Clause **209**. For special effects, such as, unequal gust load and for special type of construction, such as, long span bridges specialist literature may be referred to.

i) Seismic effects on partially constructed structure as per Clause **219**.

219 SEISMIC FORCE

219.1 Applicability

219.1.1 All bridges supported on piers, pier bents and arches, directly or through bearings, and not exempted below in the category (a) and (b), are to be designed for horizontal and vertical forces as given in the following clauses.

The following types of bridges need not be checked for seismic effects:

a) Culverts and minor bridges up to 10 m span in all seismic zones

b) Bridges in seismic zones II and III satisfying both limits of total length not exceeding 60 m and spans not exceeding 15 m

219.1.2 Special investigations should be carried out for the bridges of following description:

a) Bridges more than 150 m span

b) Bridges with piers taller than 30 m in Zones IV and V

c) Cable supported bridges, such as extradosed, cable stayed and suspension bridges

d) Arch bridges having more than 50 m span

e) Bridges having any of the special seismic resistant features such as seismic isolators, dampers etc.

f) Bridges using innovative structural arrangements and materials.

g) Bridge in near field regions

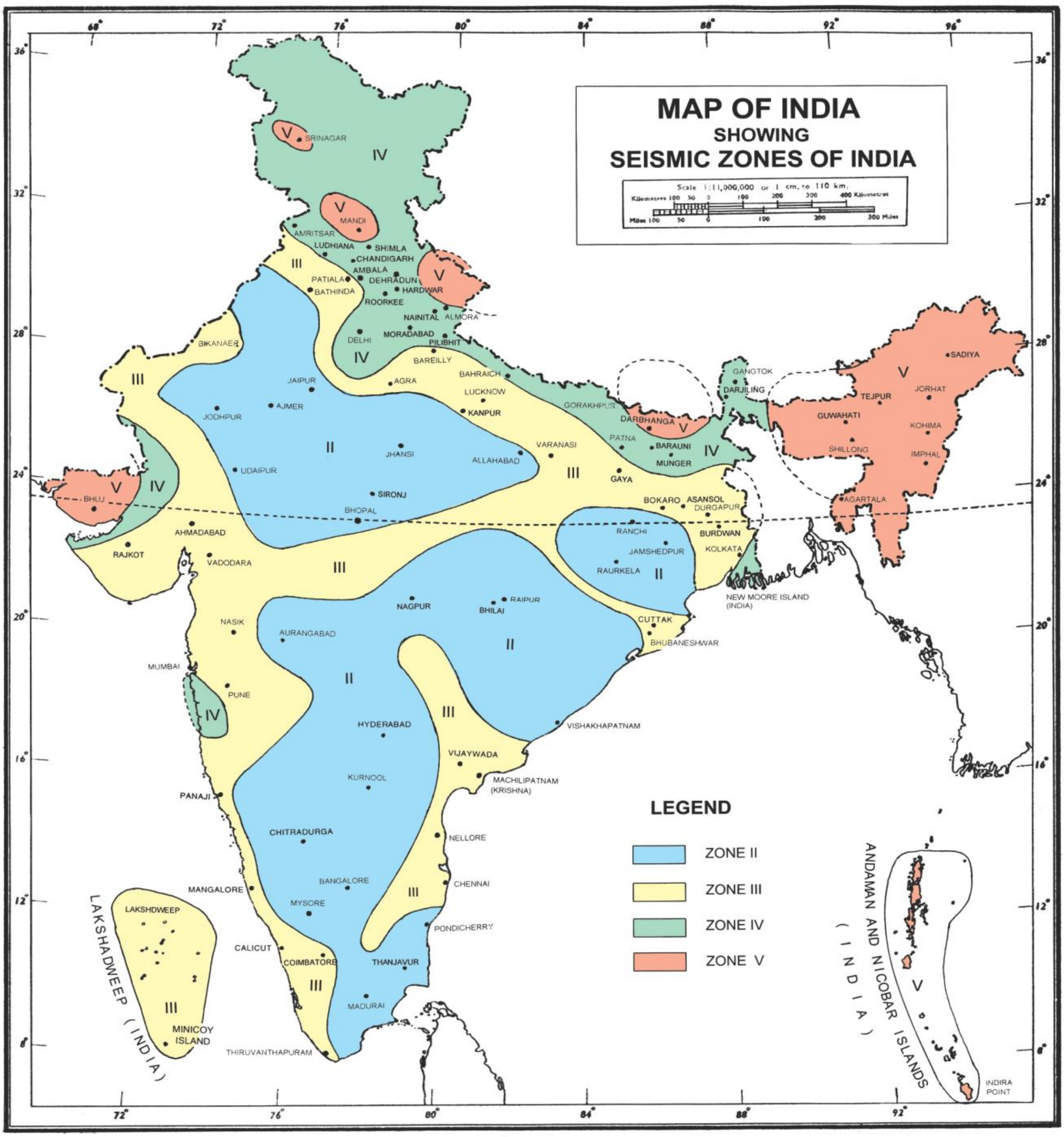
In all seismic zones, areas covered within 10 km from the known active faults are classified as ‘Near Field Regions’. The information about the active faults should be sought by bridge authorities for projects situated within 100 km of known epicenters as a part of preliminary investigations at the project preparation stage.

For all bridges located within 'Near Field Regions', except those exempted in Clause **219.1.1**, special investigations should be carried out.

Notes for special investigations:

1) Special investigations should include aspects such as need for site specific spectra, independency of component motions, spatial variation of excitation, need to include soil-structure interaction, suitable methods of structural analysis in view of geometrical and structural non-linear effects, characteristics and reliability of seismic isolation and other special seismic resistant devices, etc.

2) Site specific spectrum, wherever its need is established in the special investigation, shall be used, subject to the minimum values specified for relevant seismic zones, given in **Fig. 18.**



Based upon Survey of India Outline Map printed in 1993.

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate base line.
The boundary of Meghalaya shown on this map is as interpreted from the North-Eastern Areas (Reorganisation) Act, 1971, but has yet to be verified.
Responsibility for correctness of internal details shown on the map rests with the publisher.
The state boundaries between Uttaranchal & Uttar Pradesh, Bihar & Jharkhand and Chhatisgarh & Madhya Pradesh have not been verified by Governments concerned.

NOTE — Towns falling at the boundary of zones demarcation line between two zones shall be considered in higher zone.

Fig 18 Seismic Zones

The Fig. 18 have been reproduced in confirmation of Bureau of Indian Standards

219.1.3 Masonry and plain concrete arch bridges with span more than 10 m shall be avoided in Zones IV and V and in 'Near Field Region'.

219.2 Seismic Zones

For the purpose of determining the seismic forces, the Country is classified into four zones as shown in **Fig. 18**. For each Zone a factor 'Z' is associated, the value of which is given in **Table 16**.

Table 16: Zone factor (Z)

Zone No.	Zone Factor (Z)
V	0.36
IV	0.24
III	0.16
II	0.10

219.3 Components of Seismic Motion

The characteristics of seismic ground motion expected at any location depend upon the magnitude of earthquake, depth of focus, distance of epicenter and characteristics of the path through which the seismic wave travels. The random ground motion can be resolved in three mutually perpendicular directions. The components are considered to act simultaneously, but independently and their method of combination is described in Clause **219.4**. Two horizontal components are taken as of equal magnitude, and vertical component is taken as two third of horizontal component.

In zones IV and V the effects of vertical components shall be considered for all elements of the bridge.

The effect of vertical component may be omitted for all elements in zones II and III, except for the following cases:

a) prestressed concrete decks
b) bearings and linkages
c) horizontal cantilever structural elements
d) for stability checks and
e) bridges located in the 'Near Field Regions'

219.4 Combination of component Motions

1. The seismic forces shall be assumed to come from any horizontal direction. For this purpose two separate analyses shall be performed for design seismic forces acting along two orthogonal horizontal directions. The design seismic force resultants (i.e. axial force, bending moments, shear forces, and torsion) at any cross-section of a bridge component resulting from the analyses in the two orthogonal horizontal directions shall be combined as given in **Fig.19**.

   a) $\pm r_1 \pm 0.3r_2$

   b) $\pm 0.3r_1 \pm r_2$

   Where

   r_1 = Force resultant due to full design seismic force along *x* direction

   r_2 = Force resultant due to full design seismic force along *z* direction

2. When vertical seismic forces are also considered, the design seismic force resultants at any cross section of a bridge component shall be combined as below:

   a) $\pm r_1 \pm 0.3\ r_2 \pm 0.3\ r_3$

   c) $\pm 0.3\ r_1 \pm r_2 \pm 0.3\ r_3$

   d) $\pm 0.3\ r_1 \pm 0.3\ r_2 \pm r_3$

   Where r_1 and r_2 are as defined above and r_3 is the force resultant due to full design seismic force along the vertical direction.

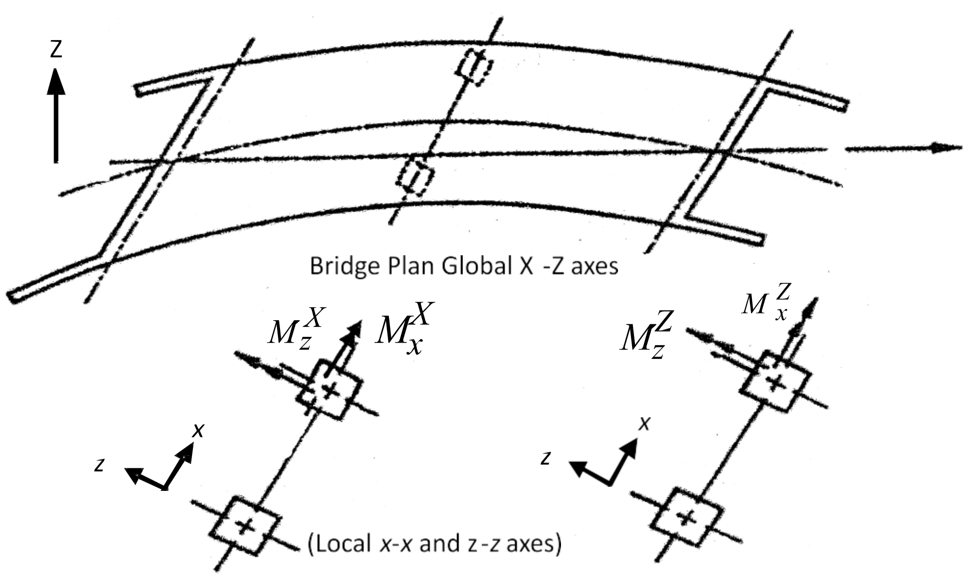


Fig. 19: Combination of Orthogonal Seismic Forces

Table 17: Design Moment for Ground Motion

	Moments for Ground Motion along x-axis	Moments for Ground Motion along Z-axis
Design Moments	$M_x = M_x^X + 0.3\, M_x^Z$	$M_Z = M_z^X + 0.3\, M_z^Z$
	$M_x = 0.3 M_x^X + M_x^Z$	$M_Z = 0.3 M_z^X + M_z^Z$
	Where, M_x and M_z are absolute moments about local	

Note: *Analysis of bridge as a whole is carried out for global axes X and Z effects obtained are combined for design about local axes as shown*

219.5 Computation of Seismic Response

Following methods are used for computation of seismic response depending upon the complexity of the structure and the input ground motion.

1) For most of the bridges, elastic seismic acceleration method is adequate. In this method, the first fundamental mode of vibration is calculated and the corresponding acceleration is read from **Fig. 20.** This acceleration is applied to all parts of the bridge for calculation of forces as per Clause **219.5.1**

2) Elastic Response Spectrum Method: This is a general method, suitable for more complex structural systems (e. g. continuous bridges, bridges with large difference in pier heights, bridges which are curved in plan, etc), in which dynamic analysis of the structure is performed to obtain the first as well as higher modes of vibration and the forces obtained for each mode by use of response spectrum from **Fig. 20** and Clause **219.5.1**. These modal forces are combined by following appropriate combinational rules to arrive at the design forces. Reference is made to specialist literature for the same.

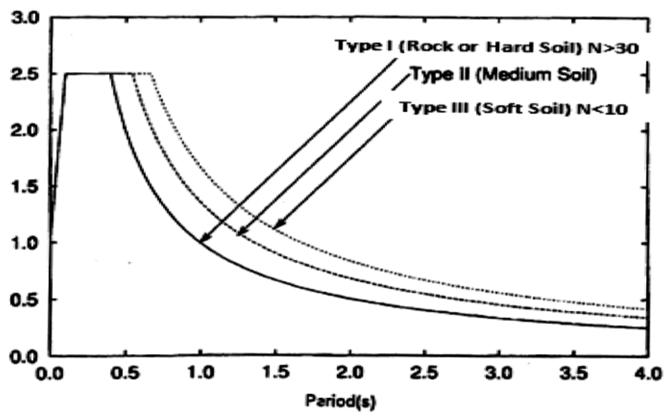


Fig. 20: Response Spectra

Note : *For structural components like short and rigid abutments, the value of Sa /g shall be taken as 1. Also, the response reduction factor R shall be taken as 1.0 for seismic design of such structures.*

219.5.1 Horizontal Seismic Force

The horizontal seismic forces acting at the centers of mass, which are to be resisted by the structure as a whole, shall be computed as follows:

$$F_{eq}=A_h \text{ (Dead Load + Appropriate Live Load)}$$

Where

F_{eq} = Seismic force to be resisted

A_h = Horizontal seismic coefficient = (Z/2) x (I) x (S_a/g)

Appropriate live load shall be taken as per Clause **219.5.2**

Z = Zone factor as given in **Table 16**

I = Importance factor (see Clause **219.5.1.1**)

T = Fundamental period of the bridge (in sec.) for horizontal vibrations

Fundamental time period of the bridge member is to be calculated by any rational method of analysis adopting the Modulus of Elasticity of Concrete (E_{cm}) as per IRC:112, and considering moment of inertia of cracked section which can be taken as 0.75 times the moment of inertia of gross uncracked section, in the absence of rigorous calculation. The fundamental period of vibration can also be calculated by method given in **Annex D**.

S_a/g = Average responses acceleration coefficient for 5 percent damping of load resisting elements depending upon the fundamental period of vibration T as given in **Fig. 20** which is based on the following equations:

For rocky or hard soil sites,
Type I soil with N > 30

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T, & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.40 \\ 1.00/T & 0.40 \leq T \leq 4.00 \end{cases}$$

For medium soil sites,
Type II soil with 10 < N ≤30

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T, & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.55 \\ 1.36/T & 0.55 \leq T \leq 4.00 \end{cases}$$

For soft soil sites,
Type III soil with N < 10

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T, & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.67 \\ 1.67/T & 0.67 \leq T \leq 4.00 \end{cases}$$

Notes:-

1. *Type I - Rock of Hard Soil: Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC) having N above 30, where N is the standard penetration value.*
2. *Type II Medium Soils : All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines SP with N>15*
3. *Type III Soft Soils: All soils other than SP with N<10*
4. *The value N(Corrected Value) are at founding level and allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1883.*

Note: *In absence of calculation of fundamental period for small bridges, (S_a / g) may be taken as 2.5*

For damping other than 5 percent offered by load resisting elements, the multiplying factors as given in **Table 18**.

Table 18: Multiplying Factor for Damping

Damping (%)	2	5	10
Factor	1.4	1.0	0.8
Application	Prestressed concrete, Steel and composite steel elements	Reinforced Concrete elements	Retrofitting of old bridges with RC piers

219.5.1.1 Seismic importance factor (I)

Bridges are designed to resist design basis earthquake (DBE) level, or other higher or lower magnitude of forces, depending on the consequences of their partial or complete non-availability, due to damage or failure from seismic events. The level of design force is obtained by multiplying (Z/2) by factor 'I', which represents seismic importance of the structure. Combination of factors considered in assessing the consequences of failure and hence choice of factor 'I'- include inter alia,

a) Extent of disturbance to traffic and possibility of providing temporary diversion,

b) Availability of alternative routes,

c) Cost of repairs and time involved, which depend on the extent of damages, - minor or major,

d) Cost of replacement, and time involved in reconstruction in case of failure,

e) Indirect economic loss due to its partial or full non-availability, Importance factors are given in **Table 19** for different types of bridges.

Table 19 Importance Factor

Seismic class	**Illustrative examples**	**Importance factor 'I'**
Normal bridges	All bridges except those mentioned in other classes	1
Important bridges	a) River bridges and flyovers inside cities b) Bridges on National and State Highways c) Bridges serving traffic near ports and other centers of economic activities d) Bridges crossing railway lines	1.2
Large critical bridges in all Seismic Zones	a)Long bridges more than 1km length across perennial rivers and creeks b) Bridges for which alternative routes are not available	1.5

Note: *While checking for seismic effects during construction, the importance factor of 1 should be considered for all bridges in all zones*.

219.5.2 Live load components

i) The seismic force due to live load shall not be considered when acting in the direction of traffic, but shall be considered in the direction perpendicular to the traffic.

ii) The horizontal seismic force in the direction perpendicular to the traffic shall be calculated using 20 percent of live load (excluding impact factor).

iii) The vertical seismic force shall be calculated using 20 percent of live load (excluding impact factor).

Note *: The reduced percentages of live loads are applicable only for calculating the magnitude of seismic design force and are based on the assumption that only 20 percent of the live load is present over the bridge at the time of earthquake.*

219.5.3 Water current and depth of scour

The depth of scour under seismic condition to be considered for design shall be 0.9 times the maximum scour depth. The flood level for calculating hydrodynamic force and water current force is to be taken as average of yearly maximum design floods. For river bridges, average may preferably be based on consecutive 7 years' data, or on local enquiry in the absence of such data.

219.5.4 Hydrodynamic and earth pressure forces under seismic condition

In addition to inertial forces arising from the dead load and live load, hydrodynamic forces act on the submerged part of the structure and are transmitted to the foundations. Also, additional earth pressures due to earthquake act on the retaining portions of abutments. For values of these loads reference is made to IS 1893. These forces shall be considered in the design of bridges in zones IV and V.

The modified earth pressure forces described in the preceding paragraph need not be considered on the portion of the structure below scour level and on other components, such as wing walls and return walls.

219.5.5 Design forces for elements of structures and use of response reduction factor

The forces on various members obtained from the elastic analysis of bridge structure are to be divided by Response Reduction Factor given in **Table 20** before combining with other forces as per load combinations given in **Table 1**. The allowable increase in permissible stresses should be as per **Table 1**.

Table 20 Response Reduction Factors

Bridge Component		'R' With Ductile Detailing	'R' without Ductile Detailing (for Bridges in zone II only)
a) Superstructure of integral / Semi integral bridge /Framed bridges		2.0	1.0
b) Other types of Superstructure, including precast segmental construction		1.0	1.0
Substructure			
(i) Masonry/PCC Piers, Abutments		1.0	1.0
(ii) RCC wall piers and abutments transverse direction (where plastic hinge can not develop)		1.0	1.0
(iii) RCC wall piers and abutments in longitudinal direction (where hinges can develop)		3.0	2.5
(iv) RCC Single Column		3.0	2.5
(v) RCC/PSC Frames	a) Column	4.0	3.0
	b) RCC beam	3.0	2.0
	b) PSC beam	1.0	1.0
(vi) Steel Framed Construction		3.0	2.5
(vii) Steel Cantilever Pier		1.5	1.0
Bearings and Connections (see note v also)		1.0	1.0
Stoppers (Reaction Blocks) Those restraining dislodgement or drifting away of bridge elements. (See Note (vi) also)		1.0	1.0

Notes :

i) Those parts of the structural elements of foundations which are not in contact with soil and transferring load to it, are treated as part of sub-structure element.

ii) Response reduction factor is not to be applied for calculation of displacements of elements of bridge and for bridge as a whole.

iii) When elastomeric bearings are used to transmit horizontal seismic forces, the response reduction factor (R) shall be taken as 1.0 for RCC, masonry and PCC substructure

iv) Ductile detailing is mandatory for piers of bridges located in seismic zones III, IV and V and when adopted for bridges in seismic zone II, for which "R value with ductile detailing" as given in ***Table 20*** *shall be used*

v) *Bearings and connections shall be designed to resist the lesser of the following forces, i.e., (a) design seismic forces obtained by using the response reduction factors given in **Table 20** and (b) forces developed due to over strength moment when hinge is formed in the substructure.*

vi) *When connectors and stoppers are designed as additional safety measures in the event of failure of bearings, R value specified in **Table 20** for appropriate substructure shall be adopted.*

219.6 Fully Embedded Portions

For embedded portion of foundation at depths exceeding 30 m below scour level, the seismic force due to foundation mass may be computed using design seismic coefficient equal to $0.5A_h$.

For portion of foundation between the scour level and up to 30 m depth, the portion of foundation mass may be computed using seismic coefficient obtained by linearly interpolating between A_h at scour level and $0.5A_h$ at a depth 30 m below scour level

219.7 Liquefaction

In loose sands and poorly graded sands with little or no fines, the vibrations due to earthquake may cause liquefaction, or excessive total and differential settlements. Founding bridges on such sands should be avoided unless appropriate methods of compaction or stabilization are adopted. Alternatively, the foundations should be taken deeper below liquefiable layers, to firm strata. Reference should be made to the specialist literature for analysis of liquefaction potential.

219.8 Foundation Design

For design of foundation, the seismic force after taking into account of appropriate R factor should be taken as 1.35 and 1.25 times the forces transmitted to it by concrete and steel substructure respectively, so as to provide sufficient margin to cover the possible higher forces transmitted by substructure arising out of its over strength. However, these over strength factors are not applicable when R=1. Also, the dynamic increment of earth pressure due to seismic need not be enhanced.

219.9 Ductile Detailing

Mandatory Provisions

i) In zones IV and V, to prevent dislodgement of superstructure, "reaction blocks" (additional safety measures in the event of failure of bearings) or other types of seismic arresters shall be provided and designed for the seismic force (F_{eq}/R). Pier and abutment caps shall be generously dimensioned, to prevent dislodgement of severe ground–shaking. The examples of seismic features shown in **Figs. 21 to 23** are only indicative and suitable arrangements will have to be worked out in specific cases.

ii) To improve the performance of bridges during earthquakes, the bridges in Seismic Zones III, IV and V may be specifically detailed for ductility for which IRC:112 shall be referred.

Recommended Provisions

i) In order to mitigate the effects of earthquake forces described above, special seismic devices such as Shock Transmission Units, Base Isolation, Seismic Fuse, Lead Plug, etc, may be provided based on specialized literature, international practices, satisfactory testing etc.

ii) Continuous superstructure (with fewer number of bearings and expansion joints) or integral bridges (in which the substructure or superstructure are made joint less, i.e. monolithic), if not unsuitable otherwise, can possibly provide high ductility leading to correct behaviour during earthquake.

iii) Where elastomeric bearings are used, a separate system of arrester control in both directions may be introduced to cater to seismic forces on the bearing.

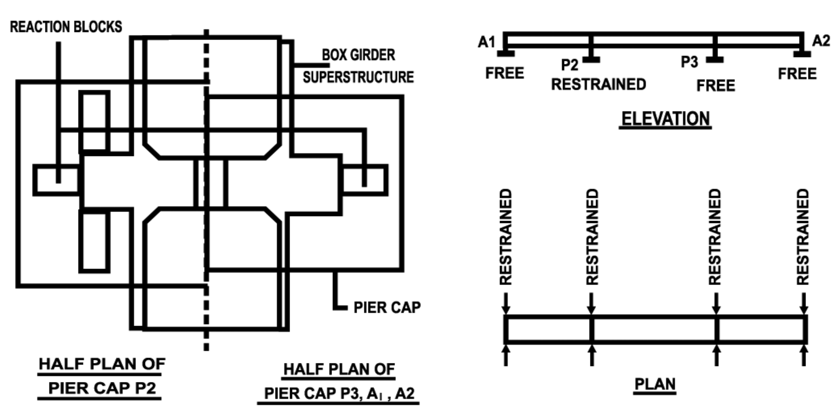


Fig. 21: Example of Seismic Reaction Blocks for Continuous Superstructure

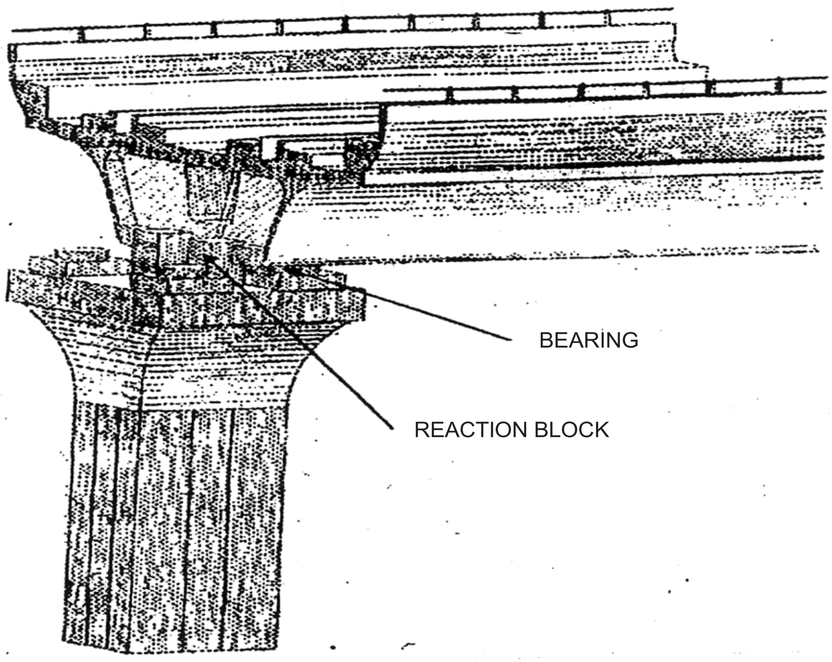


Fig. 22: Example of Seismic Reaction Blocks for Simply Supported Bridges

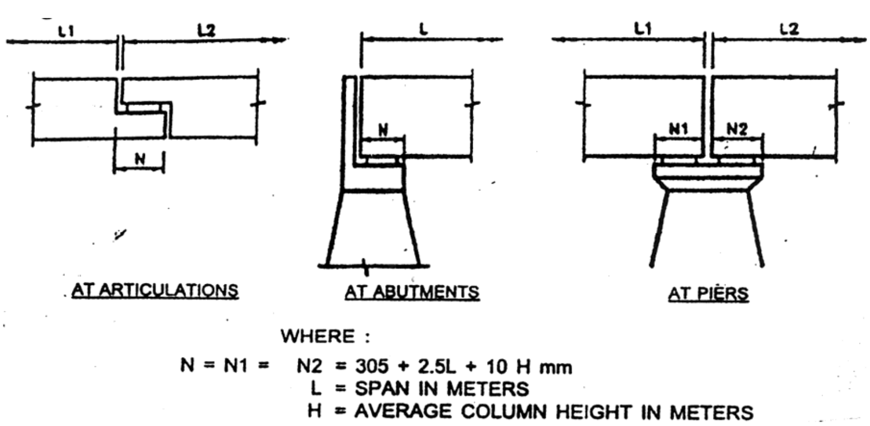


Fig. 23: Minimum Dimension for Support

220 BARGE IMPACT ON BRIDGES

220.1 General

1) Bridges crossing navigable channels of rivers, creeks and canals as well as the shipping channels in port areas and open seas shall be provided with "navigation spans" which shall be specially identified and marked to direct the waterway traffic below them. The span arrangement, horizontal clearances between the inner faces of piers within the width of the navigational channel, vertical clearances above the air-draft of the ships/barges upto soffit of deck and minimum depth of water in the channel below the maximum laden draft of the barges shall be decided based on the classification of waterways as per Inland Waterways Authority of India (IWAI) or the concerned Ports and Shipping Authorities.

2) Bridge components located in a navigable channel of rivers and canals shall be designed for barge impact force due to the possibility of barge accidentally colliding with the structure.

3) For bridges located in sea, and in waterways under control of ports, the bridge components may have to be designed for vessel collision force, for which the details of the ships/barges shall be obtained from the concerned authority. Specialist literature may be referred for the magnitudes of design forces and appropriate design solutions.

4) The design objective for bridges is to minimize the risk of the structural failure of a bridge component due to collision with a plying barge in a cost-effective manner and at the same time reduce the risk of damage to the barge and resulting environmental pollution, if any. Localized repairable damage of substructure and superstructure components is permitted provided that:

    a) Damaged structural components can be inspected and repaired in a relatively cost effective manner not involving detailed investigation, and

    b) Sufficient ductility and redundancy exist in the remaining structure to prevent consequential progressive collapse, in the event of impact.

5) The Indian waterways have been classified in 7 categories by IWAI. The vessel displacement tonnage for each of the class of waterway is shown in **Table 21**. Barges and their configurations which are likely to ply, their dimensions, the Dead Weight Tonnage (DWT), the minimum dimensions of waterway in lean section, and minimum clearance requirements are specified by IWAI. The latest requirements (2009) are shown in **Annex E.**

Table 21: Vessel Displacement Tonnage

Class of Waterway	I	II	III	IV & V	VI & VII
DWT (in Tonnes)	200	600	1000	2000	4000

Note: *The total displacement tonnage of Self Propelled Vehicle (SPV) equals the weight of the barge when empty plus the weight of the ballast and cargo (DWT) being carried by the barge. The displacement tonnage for barge tows shall equal the displacement tonnage of the tug/tow barge plus the combined displacement of number of barges in the length of the tow as shown in **Annex E**.*

6) In determining barge impact loads, consideration shall also be given to the relationship of the bridge to :

a) Waterway geometry.

b) Size, type, loading condition of barge using the waterway, taking into account the available water depth, and width of the navigable channel.

c) Speed of barge and direction, with respect to water current velocities in the period of the year when barges are permitted to ply.

d) Structural response of the bridge to collision.

7) In navigable portion of waterways where barge collision is anticipated, structures shall be :

a) Designed to resist barge collision forces, or

b) Adequately protected by designed fenders, dolphins, berms, artificial islands, or other sacrificial devices designed to absorb the energy of colliding vessels or to redirect the course of a vessel, or

c) A combination of (a) and (b) above, where protective measures absorb most of the force and substructure is designed for the residual force.

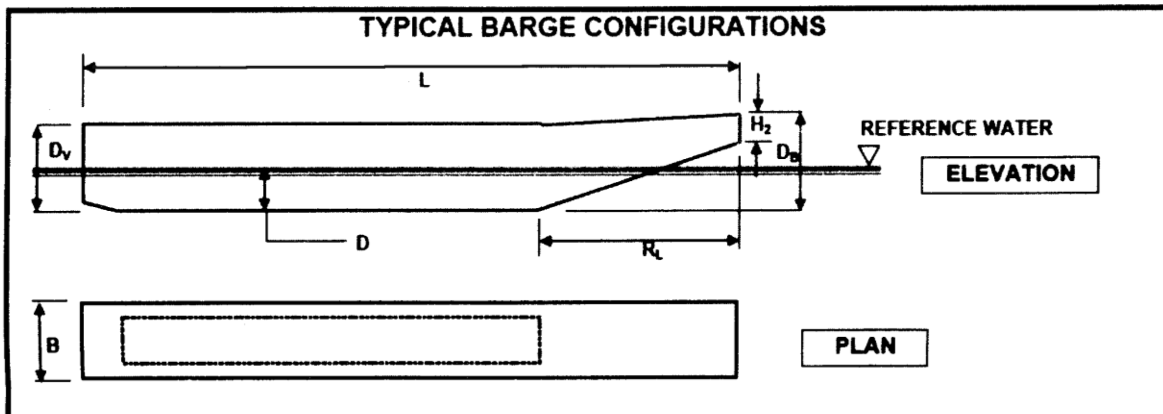
8) In non-navigable portion of the waterways, the possibility of smaller barges using these portions and likely to cause accidental impact shall be examined from consideration of the available draft and type of barges that ply on the waterway. In case such possibility exists, the piers shall be designed to resist a lower force of barge impact caused by the smaller barges as compared to the navigational span.

9) For navigable waterways which have not been classified by IWAI, but where barges are plying, one of Class from I & VI should be chosen as applicable, based on the local survey of crafts plying in the waterway. Where reliable data is not available minimum Class-I shall be assigned.

220.2 Design Barge Dimensions

A design barge shall be selected on the basis of classification of the waterway. The barge characteristics for any waterway shall be obtained from IWAI (Ref. A**nnex E**).

The dimensions of the barge should be taken from the survey of operating barge. Where no reliable information is available, the same may be taken from **Fig. 24**



BARGE DIMENSIONS AS PER IWAI

Notation	Class of Waterway	Unit	I	II	III	IV	V	VI	VII
L	Length	m	32.0	45.0	58.0	70.0	70.0	86.0	86.0
B	Width	m	5.0	8.0	9.0	12.0	12.0	14.0	14.0
D	Depth of loaded draft	m	1.0	1.2	1.5	1.8	1.8	2.5	2.5
D_B	Depth of Bow	m	1.5	1.8	2.2	2.7	2.7	3.7	3.7
D_V	Depth of Vessel	m	1.4	1.6	2.1	2.5	2.5	3.4	3.4
H_L	Head of log height	m	0.3	0.4	0.4	0.5	0.5	0.7	0.7
R_L	Bow Rake Length	m	3.3	4.6	6.0	7.2	7.2	8.9	8.9

Fig. 24: Typical Barge Dimensions

220.3 Checking in Dimensional Clearances for Navigation and Location of Barge Impact Force

Fig. 25 shows the position of bridge foundations and piers as well as the position of the barge in relation to the actual water level. The minimum and maximum water levels within which barges are permitted to ply are shown schematically. These levels should be decided by the river authorities or by authority controlling the navigation.

The minimum navigable level will be controlled by the minimum depth of water needed for the plying of barges. The maximum level may be determined by the maximum water velocity in which the barges may safely ply and by the available vertical clearances below the existing (or planned) structures across the navigable water.

The minimum vertical clearance for the parabolic soffit shall be reckoned above the high flood level at a distance/section where the minimum horizontal clearance from pier face is chosen.

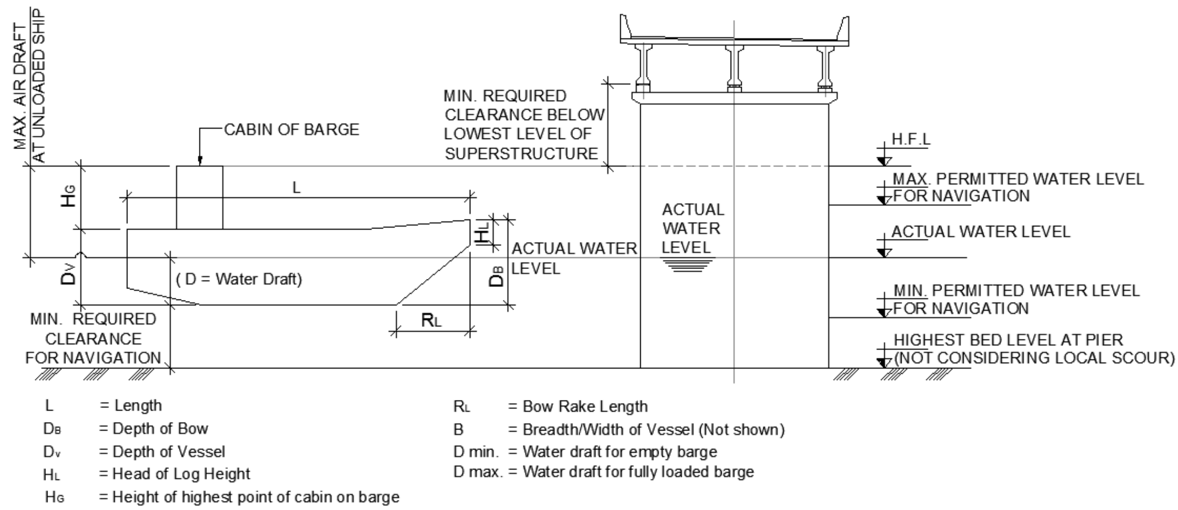


Fig. 25: Factors Deciding Range of Location of Impact Force

Use of Fig. 25:

1) **For checking Minimum Clearance below Bridge Deck :**

   a) $H_G+(D_V-D_{min})$: is maximum projection of the highest barge component above actual water level (e.g. including projecting equipment over top of cabin like radar mast)

   b) Highest Level of Barge: $H_G+(D_V-D_{min})$ + maximum permitted water level for navigation (This may be decided by water current velocity). Minimum specified clearance should be checked with reference to this level and lowest soffit level of bridge.

2) **For determining lowest position of barge with respect to bridge pier.**

   a) Maximum depth of submergence = D_{max} = Maximum Water Draft.

   b) Minimum level permitted for navigation = Level at which minimum clearance required for navigation between bed level and lowest part of barge (at D_{max}) is available.

3) **For determining range of pier elevations between which barge impact can take place anywhere:**

   a) Highest Level=Maximum water level permitted for navigation + (D_B-D_{min}).

b) Lowest Level=Minimum water level permitted for navigation + D_B-D_{max}).

c) Height over which impact force P_B acts = H_L as defined in **Fig. 25**.

220.4 Design Barge Speed

The speed at which the barge collides against the components of a bridge depends upon to the barge transit speed within the navigable channel limits, the distance to the location of the bridge element from the centre line of the barge transit path and the barge length overall (LOA). This information shall be collected from the IWAI. In absence of any data, a design speed of 6 knots (i.e. 3.1 m/sec) for unladen barge and 4 knots (i.e. 2.1 m/sec) for laden barge may be assumed for design for both upstream and downstream directions of traffic.

220.5 Barge Collision Energy

$$KE = 500 \times C_H \times W \times V^2$$

Where

W = Barge Displacement Tonnage (T)

V = Barge Impact Speed (m/s)

KE = Barge Collision Energy (N-m)

C_H = Hydrodynamic coefficient

= 1.05 to 1.25 for Barges depending upon the under keel clearance available

- In case underkeel clearance is more than 0.5 x Draft, C_H =1.05;
- In case underkeel clearance is less than 0.1 x Draft, C_H =1.25.
- For any intermediate values of underkeel clearance, linear interpolation shall be done.

Note : *The formula of kinetic energy is a standard kinetic energy, equation KE =* $\frac{1}{2} MV_1^2 C_H$ *Mass, M =* $\frac{W}{g}$ *where W is the weight of barge and* C_H *is the hydro dynamic effect representing mass of the water moving together with the barge. Substitution value in proper units in K.E. formula yields the equation given in the draft.*

220.6 Barge Damage Depth, 'a_B'

$$a_B = 3100 \times ([1+1.3 \times 10^{(-7)} KE]^{0.5} - 1),$$

Where

a_B = Barge blow damage depth (mm)

220.7 Barge Collision Impact Force, 'P_B'

The barge collision impact force shall be determined based on the following equations:

For a_B <100 mm, $P_B = 6.0 \times 10^4 \times (a_B)$, in N

For a_B ≥100 mm, $P_B = 6.0 \times 10^6 + 1600 \times (a_B)$, in N

220.8 Location & Magnitude of Impact Force in Substructure & Foundation, 'P_B'

All components of the substructure, exposed to physical contact by any portion of the design barge's hull or bow, shall be designed to resist the applied loads. The bow overhang, rake, or flair distance of barges shall be considered in determining the portions of the substructure exposed to contact by the barge. Crushing of the barge's bow causing contact with any setback portion of the substructure shall also be considered.

Some of the salient barge dimensions to be checked while checking for the navigational clearances are as follows

The design impact force for the above cases is to be applied as a vertical line load equally distributed along the barge's bow depth, H_2 defined with respect to the reference water level as shown in **Fig.25**. The barge's bow is considered to be raked forward in determining the potential contact area of the impact force on the substructure.

220.9 Protection of Substructure

Protection may be provided to reduce or to eliminate the exposure of bridge substructures to barge collision by physical protection systems, including fenders, pile cluster, pile-supported structures, dolphins, islands, and combinations thereof.

Severe damage and/or collapse of the protection system may be permitted, provided that the protection system stops the Barge prior to contact with the pier or redirects the barge away from the pier. In such cases, the bridge piers need not be designed for Barge Impact. Specialist literature shall be referred for design of protection structures.

Flexible fenders or other protection system attached to the substructure help to limit the damage to the barge and the substructure by absorbing part of impact (kinetic energy of collision). For the design of combined system of pier and protection system, the design forces as obtained from Clause **220.7** shall be used in absence of rigorous analysis.

220.10 Load Combination

The barge collision load shall be considered as an accidental load and load combination shall conform to the provisions of IRC:6. Barge impact load shall be considered only under Ultimate Limit State. For working load/allowable stress condition, allowable stress may be increased by 50 percent.

The probability of the simultaneous occurrence of a barge collision together with the maximum flood need not be considered. For the purpose of load combination of barge collision, the maximum flood level may be taken as the mean annual flood level of previous 20 years, provided that the permissible maximum current velocities for the barges to ply are not exceeded. In such event maximum level may be calculated backward from the allowable current velocities. The maximum level of scour below this flood level shall be calculated by scour formula in Clause **703.3.1** of IRC: 78. However, no credit for scour shall be taken for verifying required depth for allowing navigation.

221 SNOW LOAD

The snow load of 500 kg/m^3 where applicable shall be assumed to act on the bridge deck while combining with live load as given below. Both the conditions shall be checked independently:

a) A snow accumulation upto 0.25 m over the deck shall be taken into consideration, while designing the structure for wheeled vehicles.

b) A snow accumulation upto 0.50 m over the deck shall be taken into consideration, while designing the structure for tracked vehicles.

c) In case of snow accumulation exceeding 0.50 m, design shall be based on the maximum recorded snow accumulation (based on the actual site observation, including the effect of variation in snow density). No live load shall be considered to act along with this snow load.

222 VEHICLE COLLISION LOADS ON SUPPORTS OF BRIDGES, FLYOVER SUPPORTS AND FOOT OVER BRIDGES

222.1 General

222.1.1 Bridge piers of wall type, columns or the frames built in the median or in the vicinity of the carriageway supporting the superstructure shall be designed to withstand vehicle collision loads. The effect of collision load shall also be considered on the supporting elements, such as, foundations and bearings. For multilevel carriageways, the collision loads shall be considered separately for each level.

222.1.2 The effect of collision load shall not be considered on abutments or on the structures separated from the edge of the carriageway by a minimum distance of 4.5 m and shall also not be combined with principal live loads on the carriageway supported by the structural members subjected to such collision loads, as well as wind or seismic load. Where pedestrian/cycle track bridge ramps and stairs are structurally independent of the main highway-spanning structure, their supports need not be designed for the vehicle collision loads.

Note: *The tertiary structures, such as lighting post, signage supports etc. need not be designed for vehicle collision loads.*

222.2 Material factor of safety and Permissible overstressing in foundation

For material factor of safety under collision load reference shall be made to the provision in IRC: 112 for concrete and IRC: 24 for steel. For permissible overstressing in foundation, refer provision of IRC: 78

222.3 Collision Load

222.3.1 The nominal loads given in **Table 22** shall be considered to act horizontally as Vehicle Collision Loads. Supports shall be capable of resisting the main and residual load components acting simultaneously. Loads normal to the carriageway below and loads parallel to the carriageway below shall be considered to act separately and shall not be combined.

Table 22: Nominal Vehicle collision Loads on Supports of bridges

	Load normal to the carriageway below (ton)	Load parallel to the carriageway below (ton)	Point of Application on Bridge Support
Main load component	50	100	At the most severe point between 0.75 and 1.5 m above carriageway level
Residual load component	25 (10)	50 (10)	At the most severe point between 1 m and 3 m above carriageway level

Note : *Figures within brackets are for FOBs.*

222.3.2 The loads indicated in Clause **222.3.1**, are assumed for vehicles plying at velocity of about 60 km/hour. In case of vehicles travelling at lesser velocity, the loads may be reduced in proportion to the square of the velocity but not less than 50 percent.

222.3.3 The bridge supports shall be designed for the residual load component only, if protected with suitably designed fencing system taking into account its flexibility, having a minimum height of 1.5 m above the carriageway level.

223 INDETERMINATE STRUCTURES AND COMPOSITE STRUCTURES

Effects due to creep, shrinkage and temperature, etc. should be considered for statically indeterminate structures or composite members consisting of steel or concrete prefabricated elements and cast-in-situ components for which specialist literature may be referred to.

Annex A

(*Clause* **201.2)**

(TO BE INSERTED IN A3)

Annex A

(*Clause **201.2***)

HYPOTHETICAL VEHICLES FOR CLASSIFICATION OF VEHICLES AND BRIDGES (REVISED)

NOTES FOR LOAD CLASSIFICATION CHART

1) The possible variations in the wheel spacings and tyre sizes, for the heaviest single axles-cols. (f) and (h), the heaviest bogie axles-col. (j) and also for the heaviest axles of the train vehicle of cols. (e) and (g) are given in cols. (k), (l), (m) and (n). The same pattern of wheel arrangement may be assumed for all axles of the wheel train shown in cols. (e) and (g) as for the heaviest axles. The overall width of tyre in mm may be taken as equal to [150+(p-1) 57], where “p” represents the load on tyre in tonnes, wherever the tyre sizes are not specified on the chart.

2) Contact areas of tyres on the deck may be obtained from the corresponding tyre loads, max. tyre pressures (p) and width of tyre treads.

3) The first dimension of tyre size refers to the overall width of tyre and second dimension to the rim diameter of the tyre. Tyre tread width may be taken as overall tyre width minus 25 mm for tyres upto 225 mm width, and minus 50 mm for tyres over 225 mm width.

4) The spacing between successive vehicles shall not be less than 30 m. This spacing will be measured from the rear-most point of ground contact of the leading vehicles to the forward-most point of ground contact of the following vehicle in case of tracked vehicles. For wheeled vehicles, it will be measured from the centre of the rear-most axle of the leading vehicle to the centre of the first axle of the following vehicle.

5) The classification of the bridge shall be determined by the safe load carrying capacity of the weakest of all the structural members including the main girders, stringers (or load bearers), the decking, cross bearers (or transome) bearings, piers and abutments, investigated under the track, wheel axle and bogie loads shown for the various classes. Any bridge upto and including class 40 will be marked with a single class number-the highest tracked or wheel standard load class which the bridge can safely withstand. Any bridge over class 40 will be marked with a single class number if the wheeled and tracked classes are the same, and with dual classification sign showing both T and W load classes if the T and W classes are different.

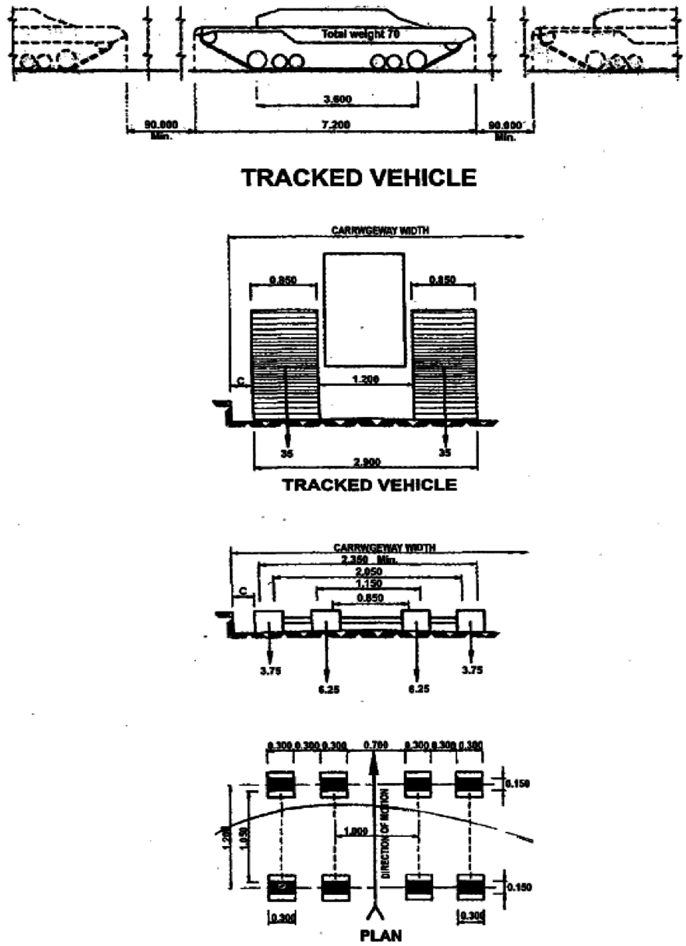
6) The calculations determining the safe load carrying capacity shall also allow for the effects due to impact, wind pressure, longitudinal forces, etc., as described in the relevant Clauses of this Code.

7) The distribution of load between the main girders of a bridge is not necessarily equal and shall be assessed from considerations of the spacing of the main girders, their torsional stiffness, flexibility of the cross bearers, the width of roadway and the width of the vehicles, etc., by any rational method of calculations.

8) The maximum single axle loads shown in columns (f) and (h) and the bogie axle loads shown in column (j) correspond to the heaviest axles of the trains, shown in columns (e) and (g) in load-classes upto and including class 30-R. In the case of higher load classes, the single axle loads and bogie axle loads shall be assumed to belong to some other hypothetical vehicles and their effects worked out separately on the components of bridge deck.

9) The minimum clearance between the road face of the kerb and the outer edge of wheel or track for any of the hypothetical vehicles shall be the same as for Class AA vehicles, when there is only one-lane of traffic moving on a bridge. If a bridge is to be designed for two-lanes of traffic for any type of vehicles given in the Chart, the clearance may be decided in each case depending upon the circumstances.

NOTES FOR LOAD CLASSIFICATION CHART



Wheeled Vehicle

Fig. A-1: Class AA Tracked and Wheeled Vehicles (Clause 204.1)

Notes :

1) *The nose to tail spacing between two successive vehicles shall not be less than 90m*.
2) *For multi-lane bridges and culverts, each Class AA loading shall be considered to occupy two lanes and no other vehicle shall be allowed in these two lanes. The passing/crossing vehicle can only be allowed on lanes other than these two lanes. Load combination is as shown in Table 6.*
3) *The maximum loads for the wheeled vehicle shall be 20 tonne for a single axle or 40 tonne for a bridge of two axles spaced not more than 1.2 m centres.*
4) *Class AA loading is applicable only for bridges having carriageway width of 5.3 m and above (i.e. 1.2 x 2 + 2.9 = 5.3). The minimum clearance between the road face of the kerb and the outer edge of the wheel or track, 'C', shall be 1.2 m.*
5) *Axle loads in tone. Linear dimensions in metre.*

Annex B

(*Clause* ***202.3*****)**

COMBINATION OF LOADS FOR LIMIT STATE DESIGN

1. Loads to be considered while arriving at the appropriate combination for carrying out the necessary checks for the design of road bridges and culverts are as follows :

1) Dead Load
2) Snow load (See note i)
3) Superimposed dead load such as hand rail, crash barrier, foot path and service loads.
4) Surfacing or wearing coat
5) Back Fill Weight
6) Earth Pressure
7) Primary and secondary effect of prestress
8) Secondary effects such as creep, shrinkage and settlement.
9) Temperature effects including restraint and bearing forces.
10) Carriageway live load, footpath live load, construction live loads.
11) Associated carriageway live load such as braking, tractive and centrifugal forces.
12) Accidental forces such as vehicle collision load, barge impact due to floating bodies and accidental wheel load on mountable footway
13) Wind
14) Seismic Effect
15) Construction dead loads such as weight of launching girder, truss or cantilever construction equipments
16) Water Current Forces
17) Wave Pressure
18) Buoyancy

Notes:

i) The snow loads may be based on actual observation or past records in the particular area or local practices, if existing

ii) The wave forces shall be determined by suitable analysis considering drawing and inertia forces etc. on single structural members based on rational methods or model studies. In case of group of piles, piers etc., proximity effects shall also be considered.

2. Combination of Loads for the Verification of Equilibrium and Structural Strength under Ultimate State

Loads are required to be combined to check the equilibrium and the structural strength under ultimate limit state. The equilibrium of the structure shall be checked against overturning, sliding and uplift. It shall be ensured that the disturbing loads (overturning, sliding and uplifting) shall always be less than the stabilizing or restoring actions. The structural strength under ultimate limit state shall be estimated in order to avoid internal failure or excessive deformation. The equilibrium and the structural strength shall be checked under basic, accidental and seismic combinations of loads.

3. Combination Principles

The following principles shall be followed while using these tables for arriving at the combinations:

i) All loads shown under Column 1 of **Table B.1** or **Table B.2** or **Table B.3** or **Table B.4** shall be combined to carry out the relevant verification.

ii) While working out the combinations, only one variable load shall be considered as the leading load at a time. All other variable loads shall be considered as accompanying loads. In case if the variable loads produce favourable effect (relieving effect) the same shall be ignored.

iii) For accidental combination, the traffic load on the upper deck of a bridge (when collision with the pier due to traffic under the bridge occurs) shall be treated as the leading load. In all other accidental situations the traffic load shall be treated as the accompanying load.

iv) During construction the relevant design situation shall be taken into account.

4. Basic Combination

4.1 For Checking the Equilibrium

For checking the equilibrium of the structure, the partial safety factor for loads shown in Column No. 2 or 3 under **Table B.1** shall be adopted.

4.2 For Checking the Structural Strength

For checking the structural strength, the partial safety factor for loads shown in Column No. 2 under **Table B.2** shall be adopted.

5. Accidental Combination

For checking the equilibrium of the structure, the partial safety factor for loads shown in Column No. 4 or 5 under **Table B.1** and for checking the structural strength, the partial safety factor for loads shown in Column No. 3 under **Table B.2** shall be adopted.

6. Seismic Combination

For checking the equilibrium of the structure, the partial safety factor for loads shown in Column No. 6 or 7 under T**able B.1** and for checking the structural strength, the partial safety factor for loads shown in Column No. 4 under **Table B.2** shall be adopted.

7. Combination of Loads for the Verification of Serviceability Limit State

Loads are required to be combined to satisfy the serviceability requirements. The serviceability limit state check shall be carried out in order to have control on stress, deflection, vibration, crack width, settlement and to estimate shrinkage and creep effects. It shall be ensured that the design value obtained by using the appropriate combination shall be less than the limiting value of serviceability criterion as per the relevant code. The rare combination of loads shall be used for checking the stress limit. The frequent combination of loads shall be used for checking the deflection, vibration and crack width. The quasi-permanent combination of loads shall be used for checking the settlement, shrinkage creep effects and the permanent stress in concrete.

7.1 Rare Combination

For checking the stress limits, the partial safety factor for loads shown in Column No. 2 under **Table B.3** shall be adopted.

7.2 Frequent Combination

For checking the deflection, vibration and crack width in prestressed concrete structures, partial safety factor for loads shown in column no. 3 under **Table B.3** shall be adopted.

7.3 Quasi-permanent Combinations

For checking the crack width in RCC structures, settlement, creep effects and to estimate the permanent stress in the structure, partial safety factor for loads shown in Column No. 4 under **Table B.3** shall be adopted.

8. Combination for Design of Foundations

For checking the base pressure under foundation and to estimate the structural strength which includes the geotechnical loads, the partial safety factor for loads for 3 combinations shown in **Table B.4** shall be used.

The material safety factor for the soil parameters, resistance factor and the allowable bearing pressure for these combinations shall be as per relevant code.

Table B.1 Partial Safety Factor for Verification of Equilibrium

Loads	Basic Combination		Accidental Combination		Seismic Combination	
	Overturning or Sliding or uplift Effect	Restoring or Resisting Effect	Overturning or Sliding or uplift Effect	Restoring or Resisting Effect	Overturning or Sliding or uplift Effect	Restoring or Resisting Effect
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1. Permanent Loads:						
1.1 Dead Load, Snow load (if present), SIDL except surfacing, Backfill weight, settlement, creep and shrinkage effect	1.1	0.9	1.0	1.0	1.1	0.9
1.2 Surfacing	1.35	1.0	1.0	1.0	1.35	1.0
1.3 Prestress and Secondary effect of prestress	(Refer Note 5)					
1.4 Earth pressure	1.5	1.0	1.0	1.0	1.0	1.0
2. Variable Loads:						
2.1 Carriageway Live load, associated loads (braking, tractive and centrifugal) and pedestrian load						
a) As leading load	1.5	0	0.75	0	-	-
b) As accompanying load	1.15	0	0.2	0	0.2	0
c) Construction live load	1.35	0	1.0	0	1.0	0
2.2 Thermal Load						
a) As leading load	1.5	0	-	-	-	-
b) As accompanying load	0.9	0	0.5	0	0.5	0
2.3 Wind Load						
a) As leading load	1.5	0	-	-	-	-
b) As accompanying load	0.9	0	-	-	-	-
2.4 Live Load Surcharge effects as accompanying load	1.2	0	-	-	-	-
3. Accidental Effects:						
3.1 Vehicle collision (or)	-	-	1.0	-	-	-
3.2 Barge Impact	-	-	1.0	-	-	-
3.3 Impact due to floating bodies	-	-	1.0	-	-	-
4. Seismic Effect						
(a) During Service	-	-	-	-	1.5	-
(b) During Construction	-	-	-	-	0.75	-
5. Construction condition:						
5.1 Counter Weights:						
a) When density or self-weight is well defined	-	0.9	-	1.0	-	1.0
b) When density or self-weight is not well defined	-	0.8	-	1.0	-	1.0
5.2 Construction Dead Loads (such as Wt. of launching girder, truss or Cantilever Construction Equipments)	1.05	0.95	-	-	-	-
5.3 Wind Load						
a) As leading load	1.5	0	-	-	-	-
b) As accompanying load	1.2	0	-	-	-	-
6. Hydraulic Loads: (Accompanying Load):						
6.1 Water current forces	1.0	0	1.0	0	1.0	-
6.2 Wave Pressure	1.0	0	1.0	0	1.0	-
6.3 Hydrodynamic effect	-	-		-	1.0	-
6.4 Buoyancy	1.0	-	1.0	-	1.0	-

Notes:

1) *During launching the counterweight position shall be allowed a variation of ± 1 m for steel bridges.*
2) *For Combination principles refer Para 3.*
3) *Thermal effects include restraints associated with expansion/contraction due to type of construction (Portal frame, arch and elastomeric bearings), frictional restraint in metallic bearings and thermal gradients. This combination however, is not valid for the design of bearing and expansion joint.*
4) *Wind load and thermal load need not be taken simultaneously unless otherwise required to cater for local climatic condition,*
5) *Partial safety factor for prestress and secondary effect of prestress shall be as recommended in the relevant codes.*
6) *Wherever Snow Load is applicable, Clause **221** shall be referred for combination of snow load and live load.*
7) *For repair, rehabilitation and retrofitting, the load combination shall be project specific.*
8) *For calculation of time period and seismic force, dead load, SIDL and appropriate live load as defined in Clause **219.5.2**, shall not be enhanced by corresponding partial safety factor as given in **Table B.1** and shall be calculated using unfactored loads.*
9) *For dynamic increment and decrements of lateral earth pressure under seismic condition Clause **214.1.2** shall be referred to.*

Table B.2 Partial Safety Factor for Verification of Structural Strength

Loads	Ultimate Limit State		
	Basic Combination	Accidental Combination	Seismic Combination
(1)	(2)	(3)	(4)
1. Permanent Loads:			
1.1 Dead Load, Snow load (if present), SIDL except surfacing			
a) Adding to the effect of variable loads	1.35	1.0	1.35
b) Relieving the effect of variable loads	1.0	1.0	1.0
1.2 Surfacing			
a) Adding to the effect of variable loads	1.75	1.0	1.75
b) Relieving the effect of variable loads	1.0	1.0	1.0
1.3 Prestress and Secondary effect of prestress	(Refer Note 2)		
1.4 Back fill Weight	1.5	1.0	1.0
1.5 Earth Pressure			
a) Adding to the effect of loads	1.5	1.0	1.0
b) Relieving the effect of loads	1.0	1.0	1.0
2. Variable Loads:			
2.1 Carriageway Live load and associated loads (braking, tractive and centrifugal) and Footway live load			
a) As leading load	1.5	0.75	-
b) As accompanying load	1.15	0.2	0.2
c) Construction live load	1.35	1.0	1.0
2.2 Wind Load construction during service			
a) As leading load	1.5	-	-
b) As accompanying load	0.9	-	-
2.3 Live Load Surcharge effects (as accompanying load)	1.2	0.2	0.2
2.4 Construction Dead Loads (such as Wt. of launching girder, truss or Cantilever Construction Equipment)	1.35	1.0	1.35
2.5 Thermal Loads			
a) As leading load	1.5	-	-
b) As accompanying load	0.9	0.5	0.5
3. Accidental effects:			
3.1 Vehicle collision (or)	-	1.0	-
3.2 Barge Impact (or)	-	1.0	-
3.3 Impact due to floating bodies	-	1.0	-
4. Seismic effect			
(a) During Service	-	-	1.5
(b) During Construction	-	-	0.75
5. Hydraulic Loads (Accompanying load):			
5.1 Water current forces	1.0	1.0	1.0
5.2 Wave Pressure	1.0	1.0	1.0
5.3 Hydrodynamic effect	-	-	1.0
5.4 Buoyancy	0.15	0.15	1.0

Notes:

1) *For combination principles, refer Para 3.*
2) *Partial safety factor for prestress and secondary effect of prestress shall be as recommended in the relevant codes.*
3) *Wherever Snow Load is applicable, Clause* ***221*** *shall be referred for combination of snow load and live load.*
4) *For calculation of time period and seismic force, dead load, SIDL and appropriate live load as defined in Clause* ***219.5.2****, shall not be enhanced by corresponding partial safety factor as given in* ***Table B.2*** *and shall be calculated using unfactored loads.*
5) *Thermal loads indicated, consists of either restraint effect generated by portal frame or arch or elastomeric bearing or frictional force generated by bearings as applicable.*
6) *For dynamic increment and decrements of lateral earth pressure under seismic condition Clause 214.1.2 shall be referred to.*

Table B.3 Partial Safety Factor for Verification of Serviceability Limit State

Loads	Rare Combination	Frequent Combination	Quasi- permanent Combination
(1)	(2)	(3)	(4)
1. Permanent Loads:			
1.1 Dead Load, Snow load if present, SIDL except surfacing	1.0	1.0	1.0
1.2 surfacing			
a) Adding to the effect of variable loads	1.2	1.2	1.2
b) Relieving the effect of variable loads	1.0	1.0	1.0
1.3 Earth Pressure	1.0	1.0	1.0
1.4 Prestress and Secondary Effect of prestress	(Refer Note 4)		
1.5 Shrinkage and Creep Effect	1.0	1.0	1.0
2. Settlement Effects			
a) Adding to the permanent loads	1.0	1.0	1.0
b) Opposing the permanent loads	0	0	0
3. Variable Loads:			
3.1 Carriageway load and associated loads (braking, tractive and centrifugal forces) and footway live load			
a) Leading Load	1.0	0.75	-
b) Accompanying Load	0.75	0.2	0
3.2 Thermal Load			
a) Leading Load	1.0	0.60	-
b) Accompanying Load	0.60	0.50	0.5
3.3 Wind Load			
a) Leading Load	1.0	0.60	-
b) Accompanying Load	0.60	0.50	0
3.4 Live Load surcharge as accompanying load	0.80	0	0
4. Hydraulic Loads (Accompanying loads) :			
4.1 Water Current	1.0	1.0	-
4.2 Wave Pressure	1.0	1.0	-
4.3 Buoyancy	0.15	0.15	0.15

Notes *:*

1) *For Combination principles, refer Para 3.*
2) *Thermal load includes restraints associated with expansion/ contraction due to type of construction (Portal frame, arch and elastomeric bearings), frictional restraint in metallic bearings and thermal gradients. This combination however, is not valid for the design of bearing and expansion joint.*
3) *Wind load and thermal load need not be taken simultaneously unless otherwise required to cater for local climatic condition,*
4) *Partial safety factor for prestress and secondary effect of prestress shall be as recommended in the relevant codes.*
5) *Where Snow Load is applicable, Clause **221** shall be referred for combination of snow load and live load.*

Table B.4 Partial Safety Factor for Checking the Base Pressure and Design of Foundation

Loads	Combination (1)	Combination (2)	Seismic Combination	Accidental Combination
(1)	(2)	(3)	(4)	(5)
1. Permanent Loads:				
1.1 Dead Load, Snow load (if present), SIDL except surfacing and Back Fill	1.35	1.0	1.35	1.0
1.2 SIDL surfacing	1.75	1.0	1.75	1.0
1.3 Prestress Effect	(Refer Note 4)			
1.4 Settlement Effect	1.0 or 0	1.0 or 0	1.0 or 0	1.0 or 0
1.5 Earth Pressure				
a) Adding to the effect of loads	1.50	1.30	1.0	1.0
b) Relieving the effect of loads	1.0	0.85	1.0	1.0
2. Variable Loads:				
2.1 All carriageway loads and associated loads (braking, tractive and centrifugal) and footway live load				
a) Leading Load	1.5	1.3	0.75 (if applicable) or 0	0.75 (if applicable) or 0
b) Accompanying Load	1.15	1.0	0.2	0.2
2.2 Thermal Load as accompanying load	0.90	0.80	0.5	0.5
2.3 Wind Load				
a) Leading Load	1.5	1.3	-	
b) Accompanying Load	0.9	0.8	0	0
2.4 Live Load surcharge as Accompanying Load (if applicable)	1.2	1.0	0.2	0.2
3. Accidental Effect or Seismic Effect				
a) During Service	-	-	1.5	1.0
b) During Construction	-	-	0.75	0.5
4. Construction Dead Loads (such as Wt. of launching girder, truss or Cantilever Construction Equipments)	1.35	1.0	1.0	1.0
5. Hydraulic Loads:				
5.1 Water Current	1.0 or 0	1.0 or 0	1.0 or 0	
5.2 Wave Pressure	1.0 or 0	1.0 or 0	1.0 or 0	
5.3 Hydrodynamic effect	-	-	1.0 or 0	
6. Buoyancy:				
a) For Base Pressure	1.0	1.0	1.0	
b) For Structural Design	0.15	0.15	0.15	

Notes :

1) *For combination principles, refer para 3.*
2) *Where two partial factors are indicated for loads, both these factors shall be considered for arriving at the severe effect.*
3) *Wind load and thermal load need not be taken simultaneously unless otherwise required to cater for local climatic condition.*

4) *Partial safety factor for prestress and secondary effect of prestress shall be as recommended in the relevant codes.*
5) *Wherever Snow Load is applicable, Clause **221** shall be referred for combination of snow load and live load.*
6) *For repair, rehabilitation and retrofitting the load combination shall be project specific.*
7) *For calculation of time period and seismic force, dead load, SIDL and appropriate live load as defined in Clause **219.5.2**. shall not be enhanced by corresponding partial safety factor as given in **Table B.4** and shall be calculated using unfactored loads.*
8) *At present the combination of loads shown in **Table B.4** shall be used for structural design of foundation only. For checking the base pressure under foundation, load combination given in IRC:78 shall be used. **Table B.4** shall be used for checking of base pressure under foundation only when relevant material safety factor and resistance factor are introduced in IRC:78.*
9) *For dynamic increment and decrement, Clause **214.1.2** on lateral earth pressure under seismic condition shall be referred to.*
10) *Thermal loads indicated, consists of either restraint effect generated by portal frame or arch or elastomeric bearing or frictional force generated by bearings as applicable.*

Annex C

*(Clause **209.3.3**)*

Wind Load Computation on Truss bridge Superstructure

C-1.1 Superstructures without live load: The design transverse wind load F_T shall be derived separately for the areas of the windward and leeward truss girder and deck elements. Except that F_T need not be derived considering the projected areas of windward parapet shielded by windward truss, or vice versa, deck shielded by the windward truss, or vice versa and leeward truss shielded by the deck.

The area A_1 for each truss, parapet etc. shall be the solid area in normal projected elevation. The area A_1 for the deck shall be based on the full depth of the deck.

C-1.2 Superstructures with live load: The design transverse wind load shall be derived separately for elements as specified in **C-1** and also for the live load depth. The area A_1 for the deck, parapets, trusses etc. shall be as for the superstructure without live load. The area A_1 for the live load shall be derived using the appropriate live load depth.

C-1.3 Drag Coefficient C_D for all Truss Girder Superstructures

a) Superstructures without live Load :

The drag coefficient C_D for each truss and for the deck shall be derived as follows:

- For a windward truss C_D shall be taken from **Table C-1**.
- For leeward truss of a superstructure with two trusses, drag coefficient shall be taken as ηC_D, values of shielding factor η are given in **Table C-2**. The solidity ratio of the truss is the ratio of the effective area to the overall area of the truss.
- Where a superstructure has more than two trusses, the drag coefficient for the truss adjacent to the windward truss shall be derived as specified above. The coefficient for all other trusses shall be taken as equal to this value.
- For Deck Construction, the drag coefficient shall be taken as 1.1.

b) Superstructure with live load:

The drag coefficient C_D for each truss and for the deck shall be as for the superstructure without live load. C_D for the unshielded parts of the live load shall be taken as 1.45.

Table C-1: Force Coefficients for Single Truss

Solidity ratio (ϕ)	Drag Coefficient C_D for		
	Built-up Sections	Rounded Members of Diameter (d)	
		Subcritical flow ($dV_z < 6m^2/s$)	Supercritical flow ($dV_z \geq 6m^2/s$)
0.1	1.9	1.2	0.7
0.2	1.8	1.2	0.8
0.3	1.7	1.2	0.8
0.4	1.7	1.1	0.8
0.5	1.6	1.1	0.8

Notes:

1) *Linear interpolation between values is permitted.*
2) *The solidity ratio of the truss is the ratio of the net area to overall area of the truss*

Table C-2: Shielding Factor η for Multiple Trusses

Truss Spacing Ratio	Value of η for Solidity Ratio				
	0.1	0.2	0.3	0.4	0.5
<1	1.0	0.90	0.80	0.60	0.45
2	1.0	0.90	0.80	0.65	0.50
3	1.0	0.95	0.80	0.70	0.55
4	1.0	0.95	0.85	0.70	0.60
5	1.0	0.95	0.85	0.75	0.65
6	1.0	0.95	0.90	0.80	0.70

Notes:

1) *Linear interpolation between values is permitted.*
2) *The truss spacing ratio is the distance between centers of trusses divided by depth of the windward truss.*

Annex D
(*Clause **219.5***)

SIMPLIFIED FORMULA FOR TIME PERIOD

The fundamental natural period T (in seconds) of pier/abutment of the bridge along a horizontal direction may be estimated by the following expression:

$$T = 2.0\sqrt{\frac{\mathrm{D}}{1000\mathrm{F}}}$$

Where,

D	=	Appropriate dead load of the superstructure and live load in kN
V	=	Horizontal force in kN required to be applied at the centre of mass of superstructure for one mm horizontal deflection at the top of the pier/ abutment for the earthquake in the transverse direction; and the force to be applied at the top of the bearings for the earthquake in the longitudinal direction.

Annex E

(*Clause* **220.1**)

CLASSIFICATION OF INLAND WATERWAYS IN INDIA

Table E-1: Class of Waterway, Dimension for Barge & Minimum Navigational Clearances

Class of Waterway	Tonnage (DWT) of SPV (T)	Barge Units			Minimum Dimensions of Navigational Channels in Lean Seasons					Minimum Clearances for cross structure		
		Dimension of Single Barge (LxBxD) (m)	Dimension of Barge Units (LxBxD) (m)	Tonnage of Barge Units (DWT) (T)	Rivers		Canals		Radius at Bend (m)	Horizontal Clearance		Vertical Clearance** (m)
					Depth* (m)	Bottom Width (m)	Depth* (m)	Bottom Width (m)		Rivers (m)	Canals (m)	
I	100	32x5x1.0	80x5x1.0	200	1.20	30	1.50	20	300	30	20	4.0
II	300	45x8x1.2	110x8x1.2	600	1.40	40	1.80	30	500	40	30	5.0
III	500	58x9x1.5	141x9 x1.5	1000	1.70	50	2.20	40	700	50	40	7.0
IV	1000	70x12x1.6	170x12x1.8	2000	2.00	50	2.50	50	800	50	50	10.0
V	1000	70x12x1.6	170x24x1.8	4000	2.00	80	-	-	800	80	-	10.0
VI	2000	86x14x2.5	210x14x2.5	4000	2.75	80	3.50	60	900	80	60	10.0
VII	2000	86x14x2.5	210x26x2.5	8000	2.75	100	-	-	900	100	-	10.0

Notes:

1) *SPV : Self Propelled Vehicle : L-Overall Length ; B-Beam Width; D-Loaded Draft*
2) *Minimum Depth of Channel should be available for 95% of the year*
3) *The vertical clearance shall be available in at least 75% of the portion of each of the spans in entire width of the waterway during lean season.*
4) *Reference levels for vertical clearance in different types of channels is given below :*
   - A) *For rivers, over Navigational High Flood Level (NHFL), which is the highest Flood level at a frequency of 5% in any year over a period of last twenty years*
   - B) *For tidal canals, over the highest high water level*
   - C) *For other canals, over designed for supply level*