CONCRETE BRIDGES

1·1. Introduction

Concrete bridges are almost universally used for both highways and rail-roads. Their durability, rigidity, economy and ease with which pleasing appearance can be obtained make them suitable for this purpose.

1·2. Types of Bridges

There are numerous types of bridges. For a particular site condition more than one type of bridge may be possible. The type of bridge, most suitable for a particular site, can be decided after rough calculations and rough estimates of cost of construction and maintenance.

The following types of bridges are in general use :

(*i*) **Simply supported slab or girder bridges.** Fig. 1·1 shows the sectional elevation of this type of bridge. It is suitable for spans upto 8 metres. If prestressed concrete girders are used, spans can go upto 50 metres.

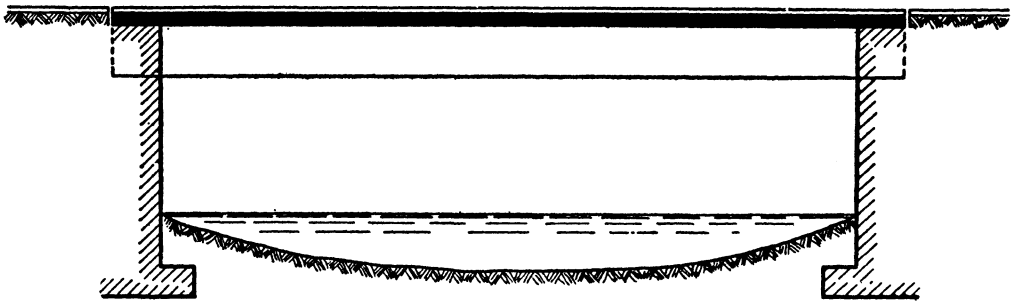


Fig. 1·1 Simply Supported Bridge

(*ii*) **Balanced cantilever bridges.** Fig. 1·2 shows the sectional elevation of balanced cantilever bridge. It can be used for spans varying from 8 metres to 20 metres. This type of bridge can be used both for reinforced concrete and prestressed concrete construction. Being a statically determinate structure, it is easier to analyse. Much longer spans are possible with prestressed concrete construction.

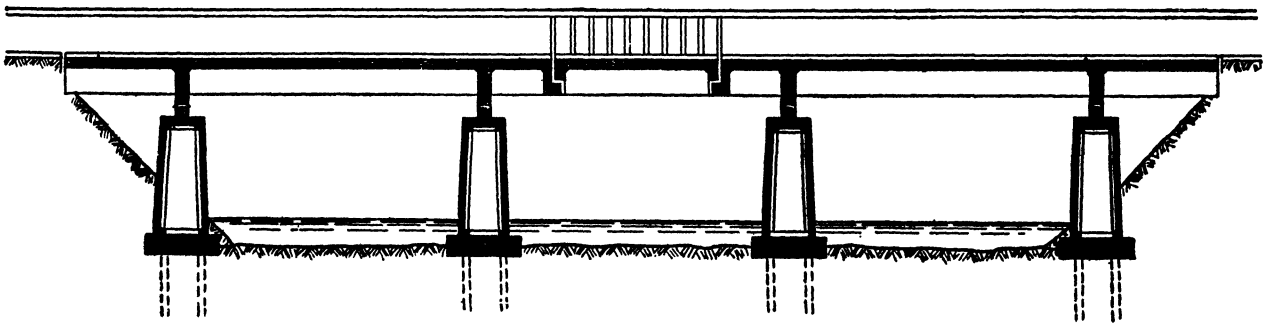


Fig. 1·2 Balance Cantilever Bridge.

(*iii*) **Continuous bridges.** Fig. 1·3 shows the sectional elevation of continuous girder bridge. It is used for large spans and where unyielding foundations are available.

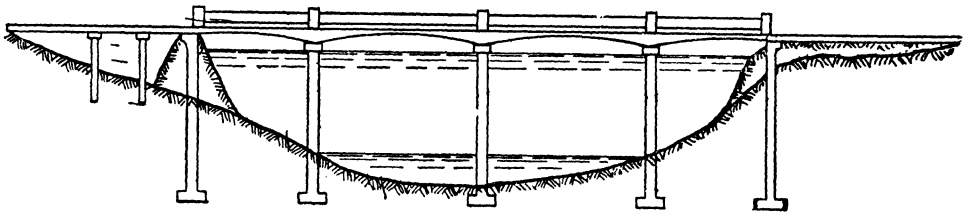


Fig. 1·3. Continuous Bridge.

(*iv*) **Arch bridges.** For very large spans the use of girder bridges becomes uneconomical. For large spans arch bridges can be economically used. These bridges can

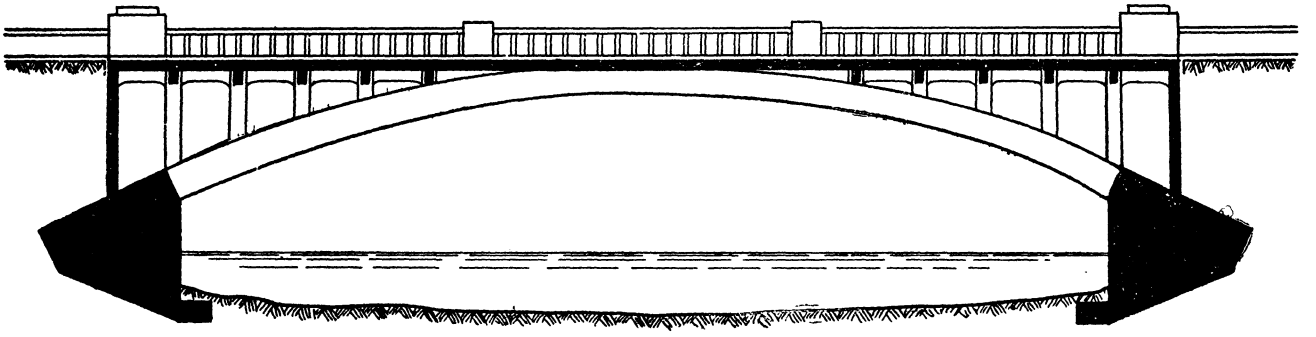


Fig 1·4. Open spandrel ribbed arch.

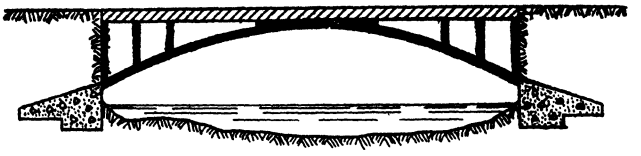


Fig. 1·5. Open spandrel arch.

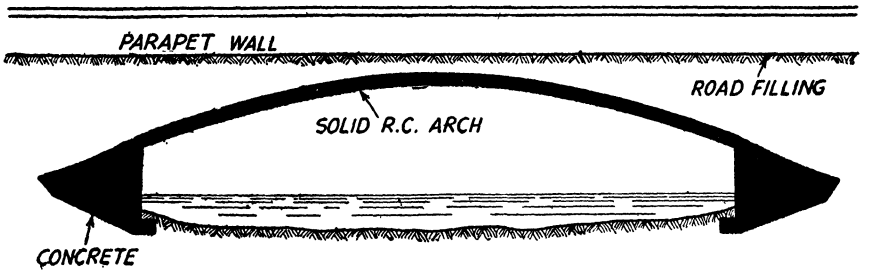


Fig. 1·6. Filled Spandrel Barrel Arch.

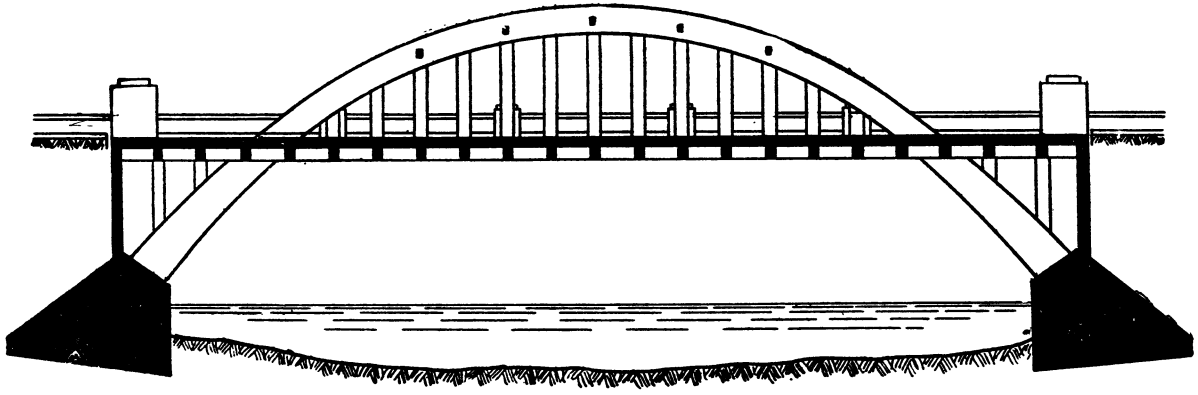


Fig. 1·7 Ribbed arch with partially hung decking.

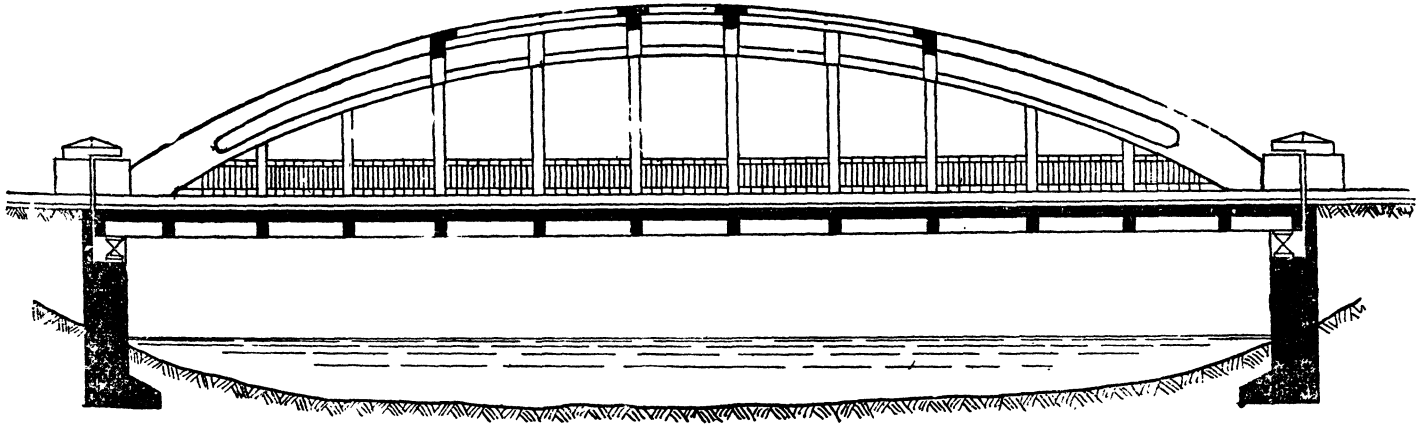


Fig. 1·8 Bowstring Girder Bridge.

be economically used upto spans of about 200 m. Another chief advantage of these bridges is their graceful and pleasing appearance.

The arches may be in barrel form or in the form of ribs. In the case of barrel form the deck is generally supported on earth filling placed on the arch slab and retained by spandrel walls. This type of arch is also known as filled span lrel arch. In the case of arch rib the deck is supported on the columns which are in turn supported on arch ribs. Such an arch is called open spandrel arch.

The types of reinforced concrete arches used are :

1, Three hinged arch. 2. Two hinged arch. 3. Fixed arch.

Fig. 1·4, 1·5, 1·6 and 1 7 show various types of arch bridges. Fig. 1·8 shows bow-string bridge which essentially consists of two hinged arch with a tie beam.

(*v*) **Rigid frame bridges.** In rigid frame bridges the horizontal deck slab is made monolithic with the vertical abutment walls. The construction is suitable for spans upto 20 metres. Generally this type of bridge is not found econonomical for spans less than 10 metres. Fig. 1·9 shows the sectional elevation for this type of bridge.

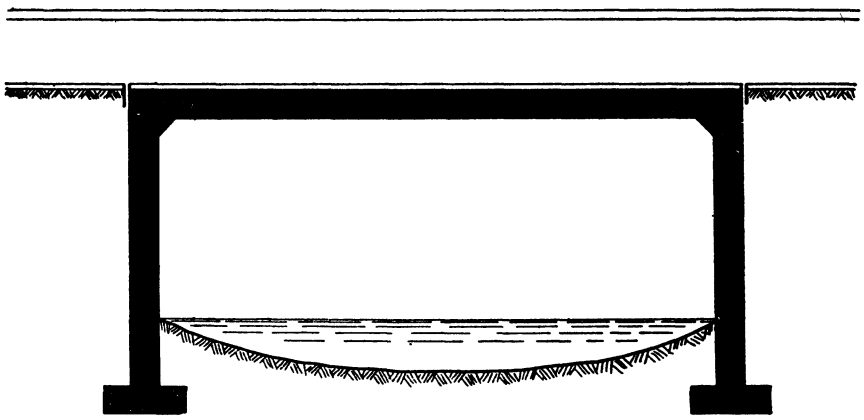


Fig. 1·9. Rigid Frame Bridge.

1·3. Economic Span Length.

The cost of a bridge is the total cost of sub-structure and super-structure. For small variation in span, the cost of pier will not be affected. The cost of deck slab will be the same for any small variation in span length. The cost of main girders and cross-girders will be directly proportional to the span. If 'L' is the total length of the bridge and 'n' is the number of piers, span $l = \frac{L}{n+1}$. The cost of sub-structure will be the cost of abutments which will not vary, plus cost of piers. Let C_A be the cost of one abutment and C_P be the cost of one pier. Let C_T be the cost of main girders and cross-girders per unit length and C_F be the cost of flooring per unit length,

Total cost $$C = C_A + nC_P + LC_F + LC_T$$

$$C_T \propto l$$

$$\therefore \quad C_T = kl$$

$$n = \frac{L}{l} - 1$$

$$\therefore \qquad C = C_A + C_P \frac{L}{l} - C_P + LC_F + Lkl$$

For cost to be minimum $\frac{dC}{al} = 0$

$$\therefore \qquad -C_P \frac{L}{l^2} + Lk = 0$$

$$\therefore \qquad \frac{C_P}{l} = kl = C_T$$

$$\therefore \qquad C_P = lC_T$$

When the cost of pier is equal to the cost of main girders and cross-girders of one span, the total cost of the bridge will be minimum.

1·4. Types of Loading.

For the purposes of computing maximum stresses in any girder or member of a bridge the following loads shall be taken into account:

(*i*) Dead load.
(*ii*) Live load.
(*iii*) Impact effect.
(*iv*) Centrifugal force.
(*v*) Wind load.
(*vi*) Lateral loads such as nosing effect in railway bridges and horizontal forces on parapets.
(*vii*) Longitudinal forces and
(*viii*) Seismic forces.

In addition to the stresses caused by above loading, the following stresses shall be taken into account where applicable :

(*a*) Secondary stresses.
(*b*) Temperature stresses and
(*c*) Erection stresses.

1·5. Dead Load.

The dead load is the weight of the structure and any permanent loads fixed thereon. The dead load initially assumed shall be checked after the design is completed and the design shall be revised if the actual calculated dead load exceeds the assumed dead load by more than 2½% or if the assumed dead load effect on a member varies from the actual dead load effect to such an extent as to adversely affect the design of such member.

Dead load on Foot Bridges and Highway Bridges. The weight of floor system for bridges can be estimated. The weight of a component is assumed and checked after design. For example, for a bridge flooring of R.C.C. supported on main girders, when designing R.C.C. flooring, its dead weight is assumed which is checked after designing the flooring. The correction in dead weight is made if necessary and the dead weight of cross beams is assumed when designing these and varified after design. Same procedure is adopted for main girders.

1·6. Live Load.

(*a*) **Foot Bridges and Footways.** For all parts of bridge floors accessible only to pedestrians and for all footways the loading shall be 400 kg/m², where crowd loads are likely to occur the loading shall be 500 kg/m².

Kerbs 600 mm. or more in width shall be designed for the above loads. If the kerb width is less than 600 mm. no live load shall be applied.

The main girders, trusses, arches or other members supporting the footways shall be designed for the following live loads per square metre of footway area.

(*i*) For effective spans of 7·5 metres or less 400 kg/m² and for crowded locations 500 kg/m².

(*ii*) For effective spans of over 7·5 metres, but not exceeding 30 metres, the intensity of load shall be determined according to the formula—

$$P = P' - \left[\frac{40L - 300}{9}\right]$$

(*iii*) For effective spans of over 30 metres, the intensity of load shall be determined according to the formula—

$$P = \left(P' - 260 - \frac{4800}{L}\right)\left(\frac{165 - W}{15}\right)$$

where P' = 400 kg/m.² or 500 kg/m.², as the case may be

P = the live load in kg/m².

L = the effective span of the main girder, truss or arch in metres.

W = the width of the footway in metres.

Where vehicles mount the footway, each part of the footway shall be capable of carrying a wheel load of 4 tonnes which shall be deemed to include impact, distributed over a contact area of 300 mm. in diameter. The working stress shall be increased by 25%.

(*b*) **For Road Bridges**

I.R.C. 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 be obtained under class *A* loading.

I.R.C. class A loading. This loading is to be normally adopted on all roads in which prominent bridges and culverts are constructed.

I.R.C. class B loading. This loading is to be normally adopted for temporary structures and for bridges in specified areas. Structures with timber spans are to be regarded as temporary structures for this clause.

The standard wheeled or tracked vehicles or trains of vehicles are illustrated in Figs. 1·10 to 1·12. The trailers attached to the driving unit are not to be considered as detachable.

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 a kerb and between two passing or crossing vehicles are not encroached upon.

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.

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

The spaces on the carriageway left uncovered by the standard train or vehicle shall not be assumed as subject to any additional live load.

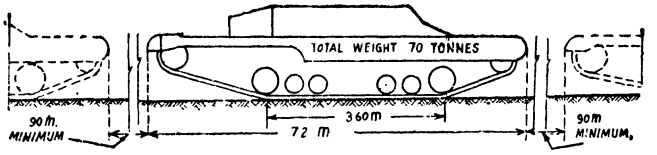


Fig. 1·10(*a*)

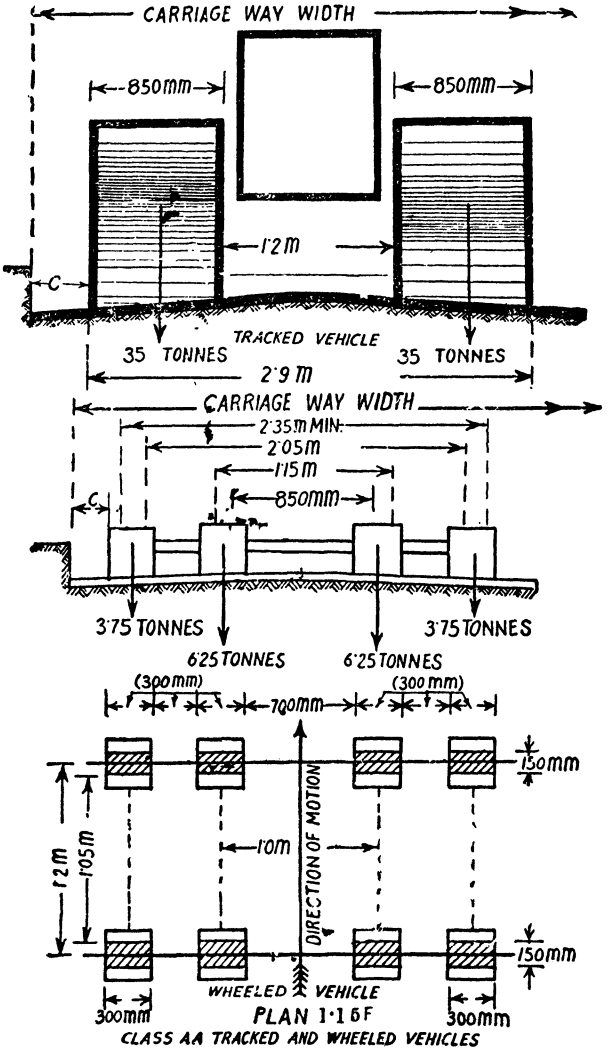


Fig. 1·10 *b*)

Notes :

1. The nose to tail spaceing between two successive vehicles shall not be less than 90 m.

2. For multi-lane bridges and culverts, one train of class *AA*, tracked or wheeled vehicles, whichever creates severer condition, shall be considered for every two traffic lane width.

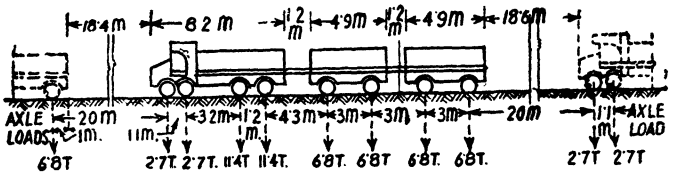
No other live load shall be considered on any part of the said 3-lane width carriageway of the bridge when the above mentioned train of vehicles is crossing the bridge.

3. The maximum loads for the wheeled vehicle shall be 20 tonnes for a single axle.

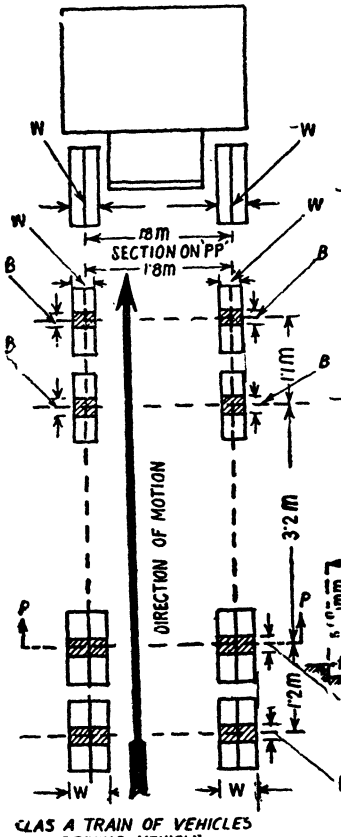
40 tonnes for a bogie of two axles spaced not more than 1·2 m. centres.

4. The minimum clearance between the road face of the kerb and the outer edge of the wheel or track *C*, shall be as under :

Carriageway width	*Minimum value of C*
Single lane bridge	
3·8 m and above	0·3 m.
Multiple lane bridges	
Less than 5·5 m	0·6 m.
5·5 m. or above	0·2 m.



CLASS A TRAIN OF VEHICLES



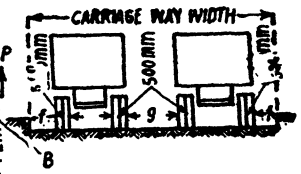
CLAS A TRAIN OF VEHICLES DRIVING VEHICLE

Notes :

1. The nose to tail distance between successive trains shall not be less than 18 4 m.
2. No other live load shall cover any part of the carriageway when a train of vehicles (or trains of vehicles in a multilane bridge) is crossing the bridge.
3. The ground contact area of the wheels shall be as under :

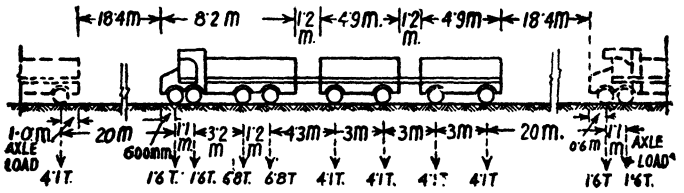
Axle load tonnes	*Ground contact area*	
	B in mm.	*W in mm,*
11·4	250	500
6·8	200	300
2·7	150	200

4. The minimum clearance *f* between the outer edge of the wheel and the roadway face of the kerb, and the minimum clearance *g*, between the other edges of passing or crossing vehicles, on multilane bridges shall be as given below :

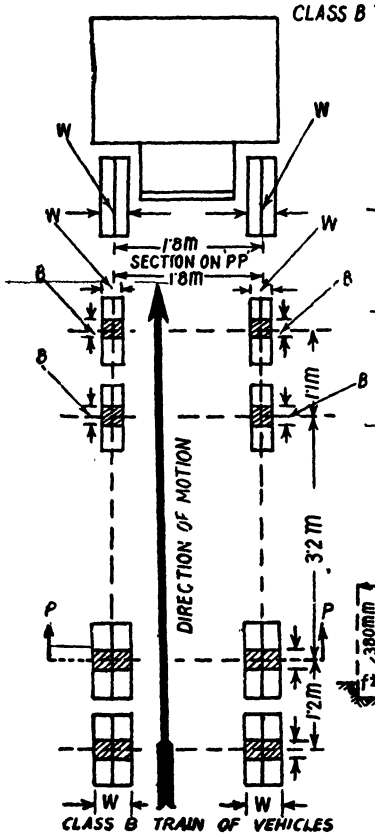


Clear carriageway width	*g*	*f*
5·5 m. to 7·5 m.	Uniformly increasing from 0·4 to 1·2 m.	0·15 m. for all carriageway widths
Above 7·5 m.	1·2 m.	

Fig. 1·11



CLASS B TRAIN OF VEHICLE



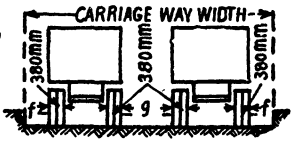
CLASS B TRAIN OF VEHICLES

Notes :

1. The nose to tail distance between successive trains shall not be less than 18·4 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 the bridge.
3. The ground contact area of the wheels shall be as under :

Axle load tonnes	*Ground contact area*	
	B in mm.	*W in mm.*
6·8	200	380
4·2	150	300
1·6	125	175

4. 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 multilane shall be as given below :



Clear carriageway width	*f*	*g*
5·5 m. to 7·5 m.	Uniformly increasing from 0·4 to 1·2 m.	0·15 m. for all carriageway widths
Above 7·5 m.	1·2 m.	

Fig. 1·12

1·7. Impact Effect

No allowance for impact is to be made for foot bridges.

Road Bridges.

(1) **For road bridges for class A or class B loading.** The impact fraction shall be determined from the following formula which is applicable for spans between 3 m. and 45 m.

For R.C. Bridges $I = \frac{4 \cdot 5}{6 \cdot 0 + L}$

where L is the span in metres.

For spans 3 m. and less impact factor will be 50% for concrete bridges. Graph in Fig. 1·13 may be used for determining impact factor.

For road bridges for class AA loading.

(*a*) For spans less than 9 m.

(*i*) For tracked vehicle	25% for spans upto 5 m. linearly reducing to 10 per cent for spans of 9 m.
(*ii*) For wheeled vehicles	25 per cent.

(*b*) For spans of 9 m. or more.

Reinforced concrete bridges.

Tracked vehicle	10 per cent upto a span of 40 m. and in accordance with the curve indicated in Fig. 1·13 for spans in excess of 40 m.
Wheeled vehicle	25 per cent for spans upto 12 m. and in accordance with the curve indicated in Fig. 1·13 for spans in excess of 12 m.

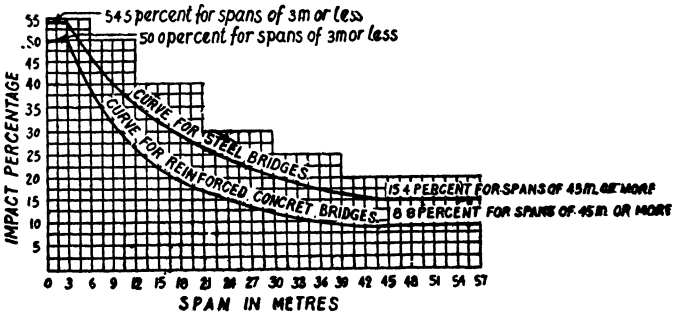


Fig. 1·13

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 one-half of what is prescribed above.

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
(*c*) For calculating the pressure on the portion of the structure more than 3 m. below the bed block.	decreasing uniformly ...zero

1·8. Centrifugal Force

For road bridges centrifugal force shall be determined from the following formula :

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

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

W = Live load (1) in tonnes in case of wheel loads each wheel load being considered as acting over the ground contact specified in article 16·7 and (2) in tonnes per linear metre in case of a uniformly distributed live load.

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

R = The radius of curvature in metres.

The centrifugal force shall be considered to act at a height of 1·2 m. above the level of the carriages.

1·9. Wind Loads

All structures shall be designed for the lateral wind forces given below. These forces shall be considered to act horizontally and in such a direction that the resultant stresses in the member under consideration are maximum.

The wind force on structure shall be assumed as a horizontal force of the intensity specified in Table 1·1 and acting on an area calculated as follows :

(*a*) For a deck structure :

The area of the structure as seen in elevation including the floor system and railings.

(*b*) For a through or half through structure.

The area of the elevation of the windward truss as specified in (*a*) above plus half the area of elevation above the deck level of all other trusses or girders.

The pressure given in the Table 1·1 shall be doubled for bridges situated in areas such as the Kathiawar, Peninsula and Orissa coasts.

Table 1·1. Wind pressures and wind velocities

H	*V*	*P*	*H*	*V*	*P*
0	80	40	30	147	141
3	98	58	38	154	151
6	107	73	46	159	166
9	115	88	54	163	176
12	123	98	62	168	185
15	128	107	76	175	200
18	133	112	91	181	210
21	138	122	106	186	224
24	143	127	122	190	234
27	145	136			

where, H = The average height in metres or the exposed surface above the mean retarding surface (ground or bed level or water level).
V = Horizontal velocity of wind in kilometres per hour at height H.
P = Horizontal wind pressure in kg. per sq. metre at height H.

The lateral wind force against any exposed moving load shall be considered as acting at 1·5 m. above the roadway and shall be assumed to have the following values—

Highway bridges ordinary ...300 kg. per linear metre.

Highway bridges carrying tramway ...450 kg. per linear metre.

The bridges shall not be considered to be carrying any live load when the wind velocity at deck level exceeds 130 km. per hour.

The total assumed wind force shall not be less than 450 kg. per linear metre in the plane of the loaded chord and 225 kilograms per linear metre in the plane of the unloaded chord on through or half through truss, latticed or other similar spans and not less than 450 kg. per linear metre on deck spans.

A wind pressure of 240 kilograms per square metre of the unloaded structure shall be used if it produces greater stresses than those produced by the combined wind forces as stated above.

1·10. Lateral Loads.

Forces on Parapets. Railings or parapets shall have a minimum height above the adjacent roadway or footway surface, of 4 m. less one half the horizontal width of the top rail or top of the parapet. They shall be designed to resist a lateral horizontal force and a vertical force each of 150 kg/m. applied simultaneously at the top of the railing or parapet. These forces shall also be considered in the design of main structural members of the bridges provided with foot paths. Where, however, foot paths are not provided, these forces need not be considered in the design of the main structural member.

Kerbs. Kerbs shall be designed for lateral loading of 750 kg/m. run of the kerb applied horizontally at the top of the kerb. This load need not be taken for the design of supporting structure.

1·11. Longitudinal Forces

Road Bridges

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

(*a*) Tractive effect caused through acceleration of the driving wheels ;
(*b*) Braking effect resulting from the application of brakes to braked wheels ; and
(*c*) Frictional resistance offered to the movement of free bearings due to change of temperature or any other cause.

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

(*a*) In the case of single lane or two lane bridge : twenty per cent of the first train load plus ten per cent of the loads of the succeeding trains or part thereof, the train loads in one lane only being considered for this purpose.

(*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.

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 accounted for.

The longitudinal force at any free bearing shall be limited to the sum of dead and live load reactions at the bearing multiplied by the appropriate coefficient of friction. The coefficient of friction at the bearings shall be assumed to have the following values :

For roller bearings	...0·03
For sliding bearings of hard copper alloy	...0·15
For sliding bearings of steel on cast iron or steel on steel	...0·25
For sliding bearing of steel on ferro asbestos	...0·20

The longitudinal force at the fixed bearing shall be taken as the algebraic sum of the longitudinal forces at the free bearings in the bridge unit under consideration and the force due to braking effect on the wheels as mentioned above.

The effect 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.

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.

1·12. Seismic Loads

Seismic loads need only be taken account if the local conditions so require and the allowance for horizontal acceleration shall depend on such local conditions, but shall not exceed 0·12 of gravity.

1·13. Frictional Resistance of Expansion Bearings

Where the frictional resistance of the expansion bearings has to be taken into account, the following coefficients shall be assumed in calculating the amount of friction bearings—

For roller bearings	...0·03
For sliding bearing of steel on cast iron or steel bearings	...0·25
For sliding bearings of steel on ferro asbestos	...0·20

1·14. Secondary Stresses

Secondary stresses fall into two groups—

(*a*) Stresses which are the result of eccentricity of connections and off-panel point loading (for example, loads rolling direct on chords, self weight of member and wind load on member). Secondly stresses due to unknown eccentricities arising out of inaccuracies in fabrication are already allowed for in the factor of safety.

(*b*) Stresses which are the result of the elastic deformation of the structure combined with the rigidity of the joints.

1·15. Temperature Effect

Where any portion of the structure is not free to expand or contract under variation of temperature, allowance shall be made for the stresses resulting from this condition. The coefficient of expansion shall be taken as $11{\cdot}7 \times 10^{-6}$ per 1°C.

1·16. Erection Forces and Effects

The weight of all permanent and temporary materials together with all other forces and effects which can operate on any part of the structure during erection shall be taken into account.

1·17. Width of Roadway and Footway

(*i*) For high level bridges constructed for the use of road traffic only, the width of roadway shall not be less than 3·8 m. for a single lane bridge and shall be increased by a minimum of 3 m. for every additional lane of traffic on a multiple lane bridges. Road bridges shall be either one lane, two lane or four lanes. Three lane bridges shall not be constructed. In the case of four lane or multiple of two lanes, a central verge of at least 1·2 metres shall be provided.

(*ii*) For bridges constructed for the use of combined road and tramway traffic, the roadway widths given in (*i*) above shall be increased by 4 m. for a single track tramway and by 7·6 m. for a double track tramway.

(*iii*) When a footway is provided its width shall not be less than 1·5 m.

1·18. General Design Requirements

1. Bar Sizes. (*i*) The maximum size of the reinforcement shall be 45 mm. diameter or a section of equivalent area, unless a bigger size is permitted by the competent authority.

(*ii*) Excepting for wiremesh and similar reinforcement the diameter of any reinforcing bar, including transverse ties, spirals, stirrups and all secondary reinforcement shall generally be not less than 6 mm.

(*iii*) The diameter of the longitudinal reinforcement bars in column shall not be less than 12 mm.

(*iv*) The diameter of wires under tensile stress in connected mesh and similar reinforcement in slabs shall be not less than 3 mm.

2. Curtailment of Bars. (*i*) To prevent large changes in the moment of resistance, the points at which the bars are curtailed shall be suitably spread.

(*ii*) In simply supported spans, at least 20 per cent of the steel required to resist the maximum bending moment shall be carried over to the supports along the tension side of the beam or slab. For a span continuous beyond a support, at least 10 per cent of the steel required to resist the maximum positive bending moment shall be similarly carried over the support.

3. Curved or sloped reinforcement. Where the alignment of the reinforcement deviates from the normal to the plane of bending, as in the case of a beam with curved or sloped soffit, only the area of the reinforcement effective in the direction normal to the plane of bending shall be considered.

4. Lateral Support in Beams. In flanged beams with length between adequate lateral restraints greater than thirty times the width of compression flange, the permissible compressive stress in concrete shall be reduced by a factor equal to $1{\cdot}5 - \frac{L}{60B}$ where L is the unsupported length and B is the width of the compression flange of the beam.

5. Deep Beams. If the depth of the web of beam exceeds 1·5 m. skin longitudinal reinforcement of an area of at least 0·5% of the web area shall be provided on each face. Spacing of such bars shall not exceed 200 mm.

6. Depth of Slab. The depth of slab in a panel is governed by maximum positive bending moment in the panel. Generally same thickness is provided for all panels. If the negative bending moment at the support is greater than the positive bending moment, it may be economical to provide slab with haunches at junction with girders. The length of the haunch should not be less than 3 times its depth.

7. Main Reinforcement in Slab. Main reinforcement of slab consists of top and bottom reinforcement placed in the direction of span. Reinforcement to resist negative bending moment is also required in central part of each span and the amount of this reinforcement should not be less than one-third of the bottom reinforcement in that span. The negative bending moment is produced in the span when adjacent span is loaded.

8. Distribution Reinforcement. (*i*) For solid slabs spanning in one direction, distributing bars shall be provided at right angles to the main tensile bars to provide for the lateral distribution of the loads. The distribution reinforcement shall be such as to produce a resisting moment in the direction perpendicular to the span equal to 0·3 times the moment due to concentrated live loads plus 0·2 times the moment due to other loads such as dead load, shrinkage, temperature, etc.

(*ii*) In cantilever slabs, the distribution steel, to resist a moment equal to that of 0·3 times live load moment plus 0·2 times dead load moment shall be provided half at top and half at the bottom of the slab.

(*iii*) The pitch of the distributing bars shall be not greater than thrice the effective depth of the slab or 450 mm. whichever is less.

9. Slabs provided with cantilevers. In girder bridges it is generally economical to provide deck cantilevering out from the outermost girders. This has the following advantages.

(*i*) Positive bending moment in the exterior span is reduced and thus the distribution of moments in exterior spans is made similar to interior spans.

(*ii*) The loads on all girders are equalized.

(*iii*) The number of girders are reduced without increasing the spacing of the girders.

(*iv*) The length of the supporting piers is reduced as the distance between outermost girders is reduced, which governs the length of the pier.

The cantilever loads produce bending moments not only in cantilevers but also in other spans. As the slab is monolithic with heavy girders the cantilever bending moments are partly resisted by the torsional resistance of the outermost girders and partly transferred to the slab. The amount of bending moment transferred to girder depends on the torsional rigidity of the girder and number of cross beams.

10. Distribution of live loads on longitudinal beams.

(*a*) When longitudinal beams are connected together by transverse members like deck slab, cross girders, diaphragms, soffit slab, etc., the distribution of bending moments between longitudinals shall be calculated by one of the following methods :

(*i*) Finding the reactions on the longitudinals assuming the support of deck slab as unyielding. This method is applicable where there are only two longitudinals.

(*ii*) Distributing the loads between longitudinals by Courbon's method strictly within its limitations, *i.e.* when the effective width of the deck is less than half the span and when the stiffness of cross girders is very much greater than that of the longitudinals, and

(*iii*) Distributing the loads between longitudinals by any rational method of grid analysis, *e.g.*, the method of harmonic analysis as given by Hendry and Jaeger or Morrice and Little's version of the isotropic plate theory of Guyon and Massonet, etc.

(*b*) In calculating the shear force on sections of longitudinal beams, wheel loads or track loads shall be allocated to respective longitudinal beams by any rational method. Alternately, the following method shall be followed.

(*i*) For loads at or within 5·5 metres of either supports—

The reaction on the longitudinal beams shall be the greatest of the results obtained by

(*a*) assuming the deck slab simply supported or continuous as the case may be, the supports being assumed unyielding ; and

(*b*) following are of the three methods as used for distribution of bending moments.

(*ii*) For loads more than 5·5 metres away from either supports—

Distribution of the loads between the longitudinals for the purpose of finding shearing forces shall be assumed to be the same as for bending moments.

The dispersion of live load along the span length through the wearing coat, deck slab and filling shall not be considered.

11. Dispersion of the Live Loads on Transverse Beams.

Dispersion of live loads along the span length through the wearing coat, deck slab and filling shall not be considered.

12. Distribution of Live Loads on Intermediate Transverse Floor Beams.

Distribution of loads between longitudinal beams for the purpose of finding bending moments and shears in intermediate transverse floor beams shall be made by one of the methods as used for distribution of bending moments.

13. T-beams and L-beams.

For *T*-beams and *L*-beams, the slab shall be considered as an integral part of the beam if adequate bond and shear resistance is provided at the junction of the slab and the web of the beam.

For the purpose of calculation of stress at any section of a T-beam or L-beam or in the calculation of its moment of inertia, the effective width of slab, to function as the compression flange of the beam, shall be the least of the following :

(I) *In case of T-beams*

(*i*) One-fourth the effective span of the beam.

(*ii*) The distance between the centres of the ribs of the beam.

(*iii*) The breadth of the rib plus twelve times the thickness of the slab.

(II) *In case of L-beams.*

(*i*) One-tenth the effective span of the beam.

(*ii*) The breadth of the rib plus one-half the clear distance between the ribs.

(*iii*) The breadth of the rib plus four times the thickness of the slab.

If T-form or L-form is used only for the purpose of providing additional compression area, such as the continuous beams over supports, the flange thickness shall not be less than one-half the width of the web and a total flange width not more than four times the width of the web. For effective stress transfer, it is desirable to splay the junction of the web and flange so as to form an angle of not less than 110° as shown in Fig. 1·14.

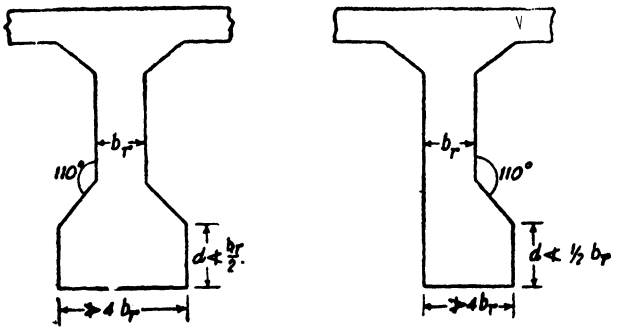


Fig. 1·14

Where the principal reinforcement in a slab which is considered as the flange if a T-beam or L-beam is parallel to the beam, transverse reinforcement shall be provided at the top of the flange. This reinforcement shall be equal to sixty per cent of the main reinforcement of the slab at its mid-span unless it is specially calculated. The length of such reinforcing bars shall be as indicated in Fig. 1·15.

For T-beams or L-beams with spans over 10 m. in length, properly designed diaphragms shall be placed at suitable points at the discretion of the designer. The spacings of such diaphragms shall not be more than thirty times the minimum thickness of the web. Cross girders monolithic with deck slab shall be provided at the bearings and also at the ends of cantilevers.

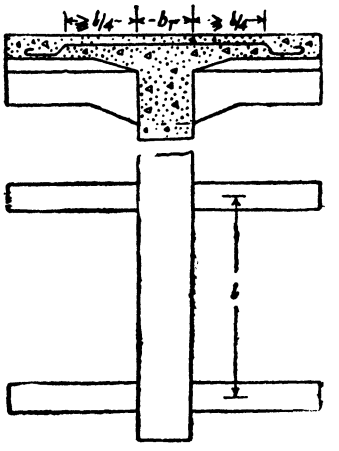


Fig. 1·15

14. Increase in permissible stresses. (*i*) When the effect of temperature, shrinkage and creep is taken into consideration, the permissible stresses should not be exceeded in the case of determinate structures but may be exceeded by 15 per cent in the case of indeterminate structures.

(*ii*) When the effect of wind forces is taken into consideration, the permissible stresses may be exceeded by 25%.

(*iii*) When the worst combination of loads met with during erection is considered, the permissible working st sses may be exceeded by 25%.

(*iv*) When the effect of seismic forces is also considered the permissible working stresses may be exceeded by 50%.

15. Balanced cantilever and continuous construction

(*i*) **Moments.** The section where the dead load moment is of the sign opposite to that of live-load moment the design bending moment for this condition of loading shall be taken as equal to the live load moment plus 0·7 times dead load moment with proper signs. This shall be in addition to moments expected under normal loads. However at least 20% of the maximum negative reinforcement at the support shall be carried out right across the span even it not required by design calculations.

(*ii*) **Articulations.** For beams resting on articulations, to prevent damage to articulation the design of the bearings shall be such that it will not induce concentrated edge stresses and will be able to allow angular deflection of the cantilevers and suspended spans.

(*iii*) **Distribution of reinforcement in tensile flanges.** When the flanges of T-beams are subjected to tensile stresses in longitudinal direction (zones of negative moments), the bars of the principal reinforcement shall be distributed not only above the web but also in a suitable position in the flange, on either side of the web.

16. Cover. (*a*) The thickness of concrete cover (exclusive of plaster or other decorative finish) shall be as follows :-

(*i*) At each end of a reinforcing bar, not less than 25 mm. nor less than the diameter of such bar ;

(*ii*) For a longitudinal reinforcing bar in a column, or beam not less than 40 mm. nor less than the diameter of such bar.

(*iii*) In no case shall the cover to any reinforcement including stirrups or binders be less than 25 mm. or less than the diameter of such bar.

In case of bars which are not round, the diameter shall be taken as the diameter of a circle giving an equivalent area.

(*b*) The cover of concrete in direct contact with earth faces shall be 25 mm. more than that specified above.

(*c*) For reinforced concrete members, totally immersed in sea water, the cover shall be 40 mm. and members subjected to sea spray, the cover of concrete shall be 50 mm. more than that specificed in (*a*).

(*d*) Increased cover thickness, as may be decided by the designer, shall be provided when surfaces of concrete members are exposed to the action of harmful chemicals, acid vapours saline atmosphere, sulphurous smoke (as in case of bridges over steam operated railways) etc. It may however be noted that dense and strong concrete is a better guarantee of protecting the reinforcement from the harmful effects of chemicals than increasing the thickness of cover.

(*e*) For controlled concrete with specified cube strength greater than 200 kg./cm^2, the increase in thickness specified in (*b*), (*c*) and (*d*) may be reduced by half.

17. Web Reinforcement. Where web reinforcement is not required to carry, shear, minimum reinforcement of area not less than 0·15% of the area of the web in plan shall be provided.