IRC CODES:

S.N.	CODE NO.	TITLE OF THE PUBLICATION
1	IRC:2-1968	Route Marker Signs for National Highways (First Revision)
2	IRC:3-1983	Dimensions & Weights of Road Division Vehicles (First Revision)
3	IRC:5-1998	Standard Specifications & Code of Practice for Road Bridges, Section I – General Features of Design (Seventh Revision)
4	IRC:8-1980	Type Designs for Highway Kilometre Stones (Second Revision)
5	IRC:9-1972	Traffic Census on Non – Urban Roads (First Revision)
6	IRC:14-1977	Recommended Practice for location and Layout of Roadside Motor-Fuel Filling and Motor – Fuel Filling-cum-Service Stations (Second Revision)
7	IRC:16-1989	Specification for Priming of Base Course with Bituminous Primers (First Revision)
8	IRC:17-1965	Tentative Specification for Single Coat Bituminous Surface Dressing
9	IRC:18-2000	Design Criteria for Prestressed Concrete Road Bridges (Post-Tensioned Concrete) (Third Revision)
10	IRC:22-1986	Standard Specifications and code of Practice for Road Bridges, Section VI – Composite Construction (First Revision)
11	IRC:24-2001	Standard Specifications and code of Practice for Road Bridges, Section V – Steel Road Bridges (Second Revision)
12	IRC:33-1969	Standard Procedure for Elevation and Condition Surveys of Stabilised Soil Roads
13	IRC:34-1970	Recommendations for Road Construction in Waterlogged Areas
14	IRC:37-2001	Guidelines for the Design of Flexible Pavements (Second Revision)
15	IRC:39-1986	Standards for Road-Rail Level Crossings (First Revision)
16	IRC:49-1973	Recommended Practice for the Pulverization of Black Cotton Soils for Lime Stabilisation
17	IRC:51-1992	Guidelines for the Use of Soil Lime Mixers in Road Construction
18	IRC:53-1982	Road Accident Dorms A-1 and 4 (First revision)
19	IRC:54-1974	Lateral and Vertical Clearances at Underpasses for Vehicular Traffic
20	IRC:58-1963	Guidelines for the Design of Rigid Pavements for Highways (First Revision)
21	IRC:63-1976	Tentative Guidelines for the Use of Low Grade Aggregates and Soil Aggregate Mixtures in Road Pavement Construction
22	IRC:66-1976	Recommended Practice for Sight Distance on Rural Highways
23	IRC:72-1978	Recommended Practice for Use and Upkeep of Equipment, tools and Appliances for Bituminous Pavement Construction

IRC

ROUTE MARKER SIGNS
FOR
NATIONAL HIGHWAYS

(*First Revision*)

Published by

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ROUTE MARKER SIGNS FOR NATIONAL HIGHWAYS

1. INTRODUCTION

1.1. Planting of route marker signs along National Highways is considered advantageous for more than one reason. Type designs for National Highway route marker were developed initially in the Roads Wing of the Ministry of Transport, Government of India and discussed at the Chief Engineers' meeting held in April 1952. The design finalised in light of these discussions was issued by the Consulting Engineer (Road Development) to the Government of India for general adoption and also published as an Indian Roads Congress Standard in 1953.

1.2. Consequent upon switchover to metric system in the country, it became imperative to metricise the Standard. The metricisation was initially considered by a Subcommittee of the Indian Roads Congress dealing with matters concerning roads. Later on, it was examined by the Specifications and Standards Committee (personnel given on inside front cover) along with a general revision of the Standard at its meetings held in 1967. Besides rationalisation of the various dimensions and values into metric units, certain other important changes have also been incorporated in this revised edition which was approved by the Executive Committee at their meeting held on 24th September, 1968 and finally by the Council at their meeting held at Bombay on 2nd November, 1968.

2. DESIGN

2.1. A National Highway route marker sign shall consist of a shield painted on a rectangular plate 450 mm by 600 mm. The design is given in Plate 1.

2.2. The sign shall have a yellow background and the lettering and border shall be in black. The yellow colour shall conform to "Canary Yellow, Indian Standard Colour No. 309". Materials of the paint shall be in accordance with the requirements specified by the Indian Standards Institution.

2.3. The size, shape and spacing of the letters and numerals shall conform to those given in Fig. 1 and Plates 1 and 5.

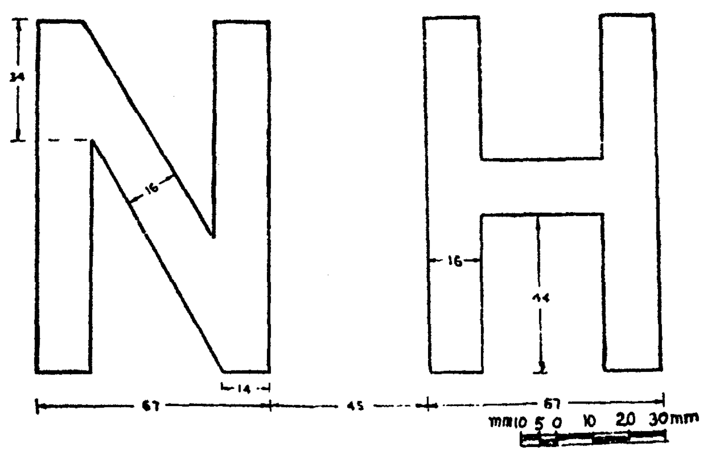


Fig.1: STANDARD LETTERS N AND H OF 100 mm HEIGHT
(All dimensions are in millimetres)

3. LOCATION

3.1. The sign shall be erected on National Highways ahead of their intersections with other important roads, immediately after the intersections as confirmatory route markers, at suitable locations through built-up areas, and at such other points that may be considered necessary to guide the throuh traffic.

3.2. The sign shall be erected as indicated **in** the drawing titled "Arrangement for Erection of National Highway Route **Marker Signs"**, Plate 2.

3.3. On roads without kerbs, the sign shall be erected with a clear distance of 2 to **3 metres between the post and the** edge of the carriageway. On roads **with kerbs,** the sign post shall not be less than **60 mm away from the** edge of the kerb. To avoid specular reflection from the sign face, the **sign shall** be turned slightly away from the road as indicated in Plate 2.

3.4. The distance (along the National Highway) of the sign from the junction, on either side of it, **shall be** 100 to 150 **metres. Also, it shall be fixed on the** left hand side as one approaches the junction.

4. DEFINITION PLATE

4.1. When the sign is erected in advance of **a junction, the** direction which the National Highway takes at **the junction shall be indicated** on a definition plate of the size 300 mm by 250 mm **fixed** below the shield as shown in Plate 2.

4.2. Background colour of the definition plate shall be the same as of the shield (Clause 2.2.). The border and arrow shall be in black.

4.3. Some type designs of arrows for use on the definition plate in different situations are given in Plate 3.

5. ROUTE MARKER ASSEMBLY AT JUNCTIONS WITH NUMBERED ROUTES

5.1. When a numbered route intersects or takes off from a National Highway, indication about the number of the intersecting route may be provided by erecting, ahead of the intersection, its route marker sign along with the marker of the National Highway route being travelled upon. Such auxiliary markers shall be mounted on the same post as carrying the regular route marker and be accompanied by a definition plate carrying a single or a double-headed arrow pointing in the general direction or directions in which that route may be followed.

5.2. The manner of putting up such assemblies is illustrated through two examples given in Plate 4.

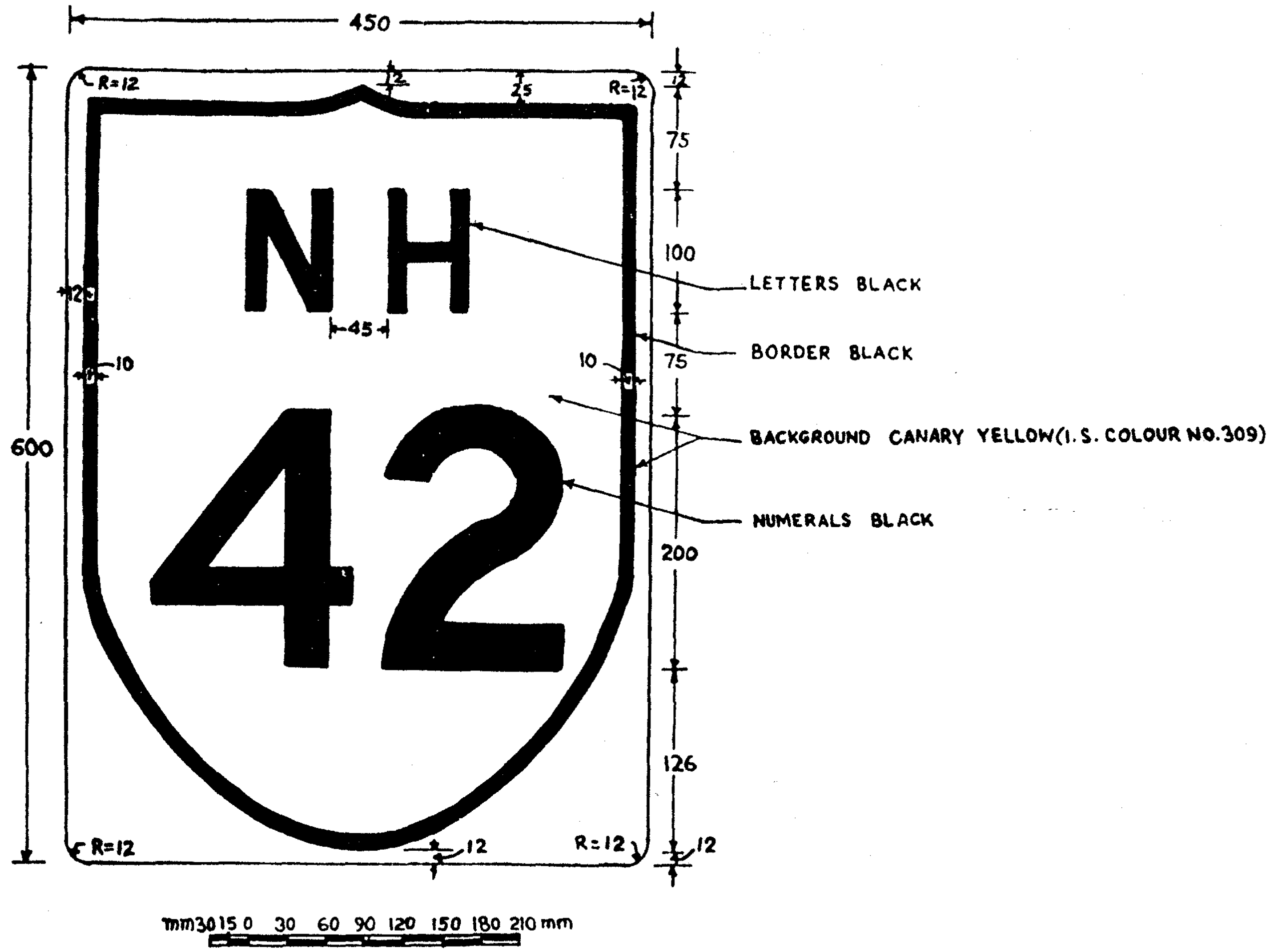
6. COLOUR OF BACK OF SIGN AND POST

Similar to other traffic signs, the reverse side of all route marker signs should be painted in unobtrusive grey, Indian Standard Colour No. 630. The sign post should be painted in 25 cm bands, alternately black and white, with the lowest band next to the ground being black.

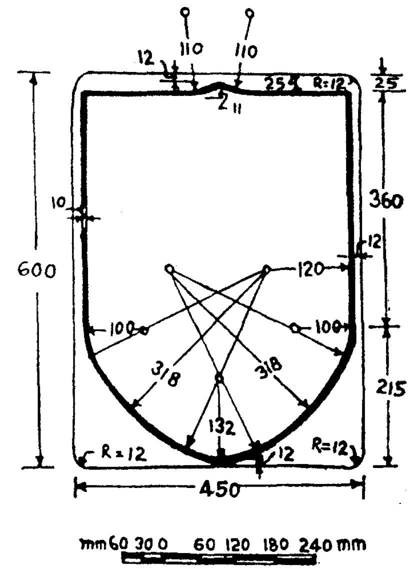
7. MATERIALS

The sign may be of either enamelled or painted steel plate.

PLATE 1

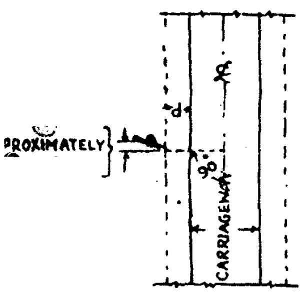


All dimensions are in millimetres

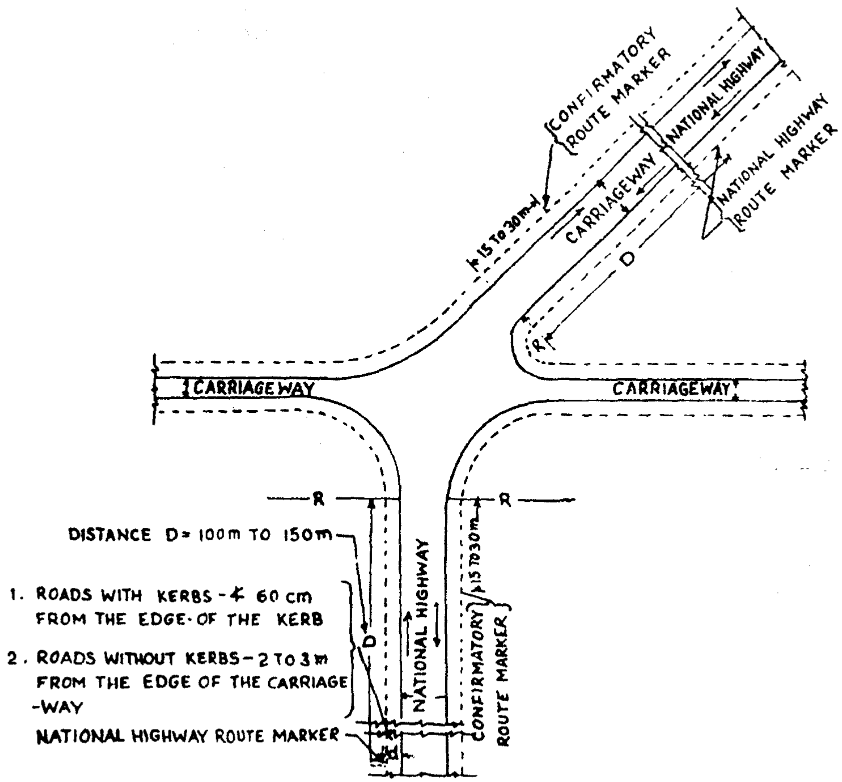


NATIONAL HIGHWAY
ROUTE MARKER SIGN

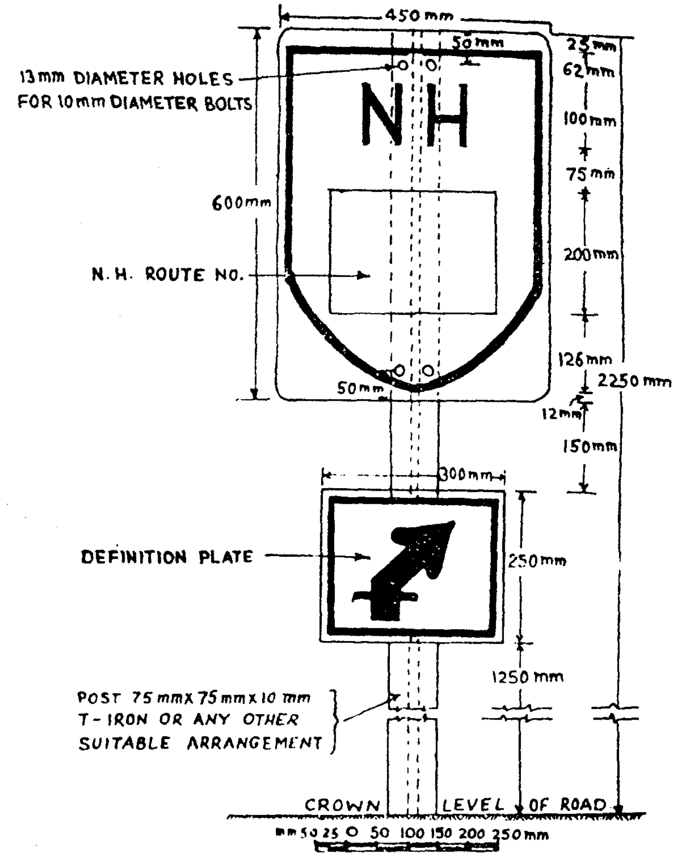
PLATE 2



POSITIONING OF THE SIGN



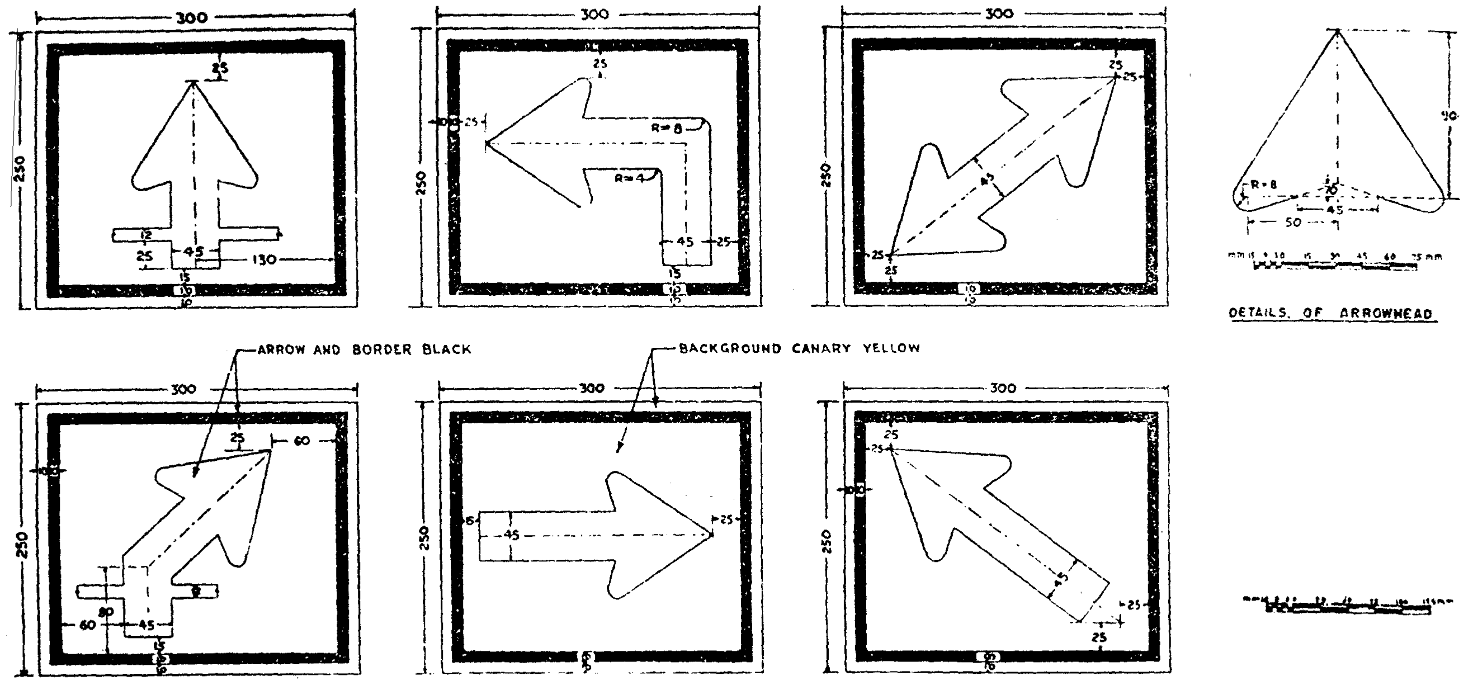
PLAN SHOWING LOCATION OF ROUTE MARKER SIGN AT A JUNCTION



ASSEMBLY OF ROUTE MARER SIGN

ARRANGEMENT FOR ERECTION OF NATIONAL HIGHWAY ROUTE MARKER SIGN

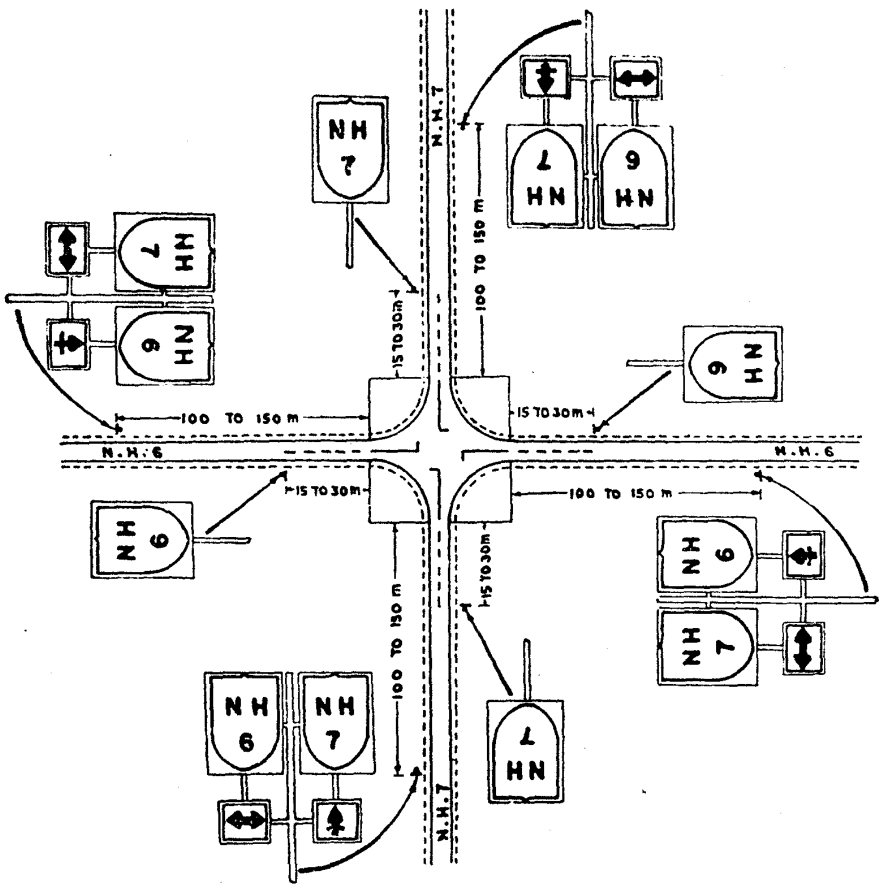
PLATE 3



NOTES :

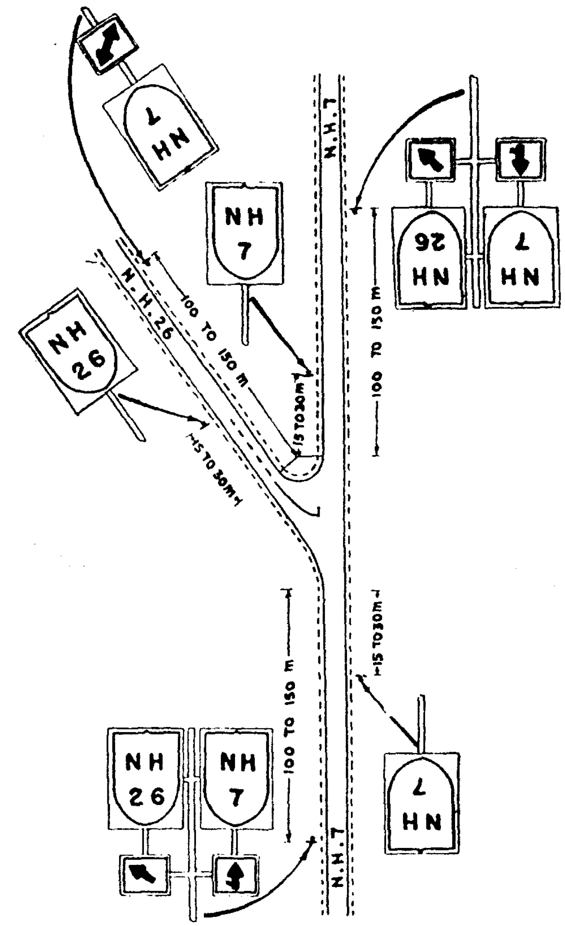
1. ALL DIMENSIONS ARE IN MILLIMETRES
2. BACKGROUND COLOUR SHALL BE CANARY YELLOW (IS COLOUR NO. 309) AND THE BORDER AND ARROW IN BLACK
3. ARROWS GIVEN IN THIS PLATE ARE ONLY EXAMPLES. FOR OTHER SITUATIONS, ARROWS COULD BE SUITABLY EVOLVED ON SIMILAR LINES.

DETAILS OF TYPICAL ARROWS FOR USE ON THE DEFINITION PLATE



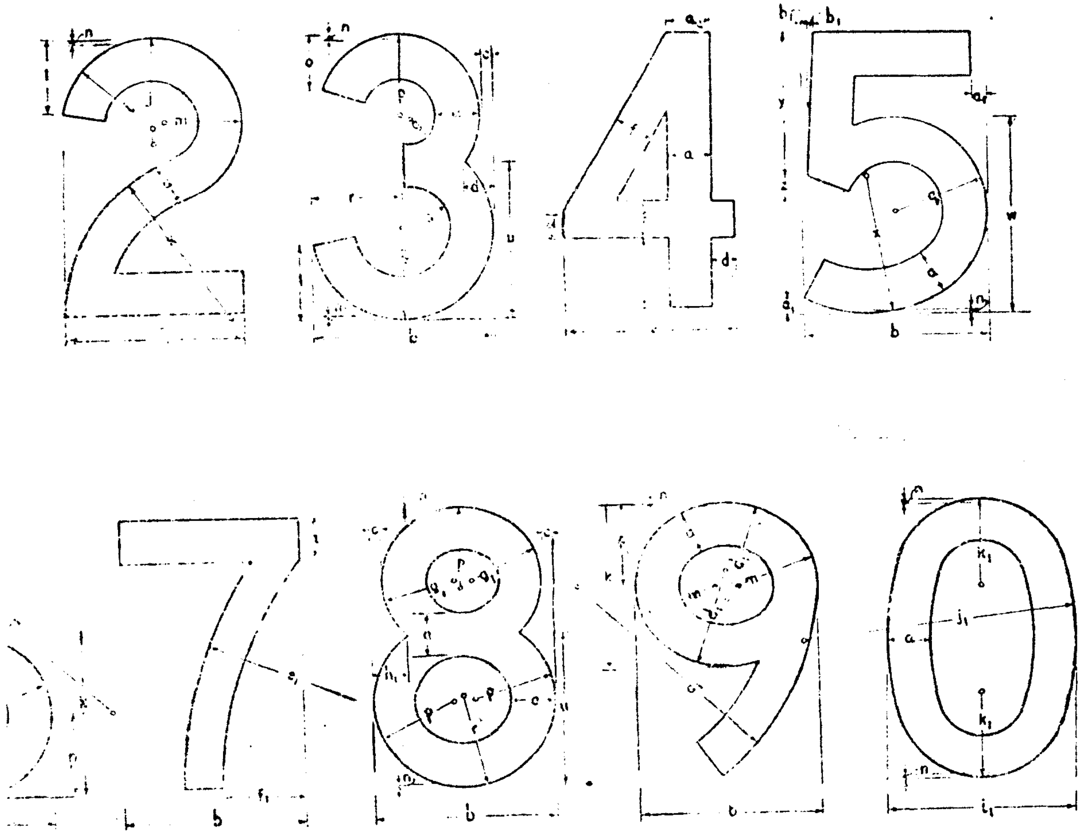
Example 1

NOTE : THE EXAMPLES GIVEN ABOVE ARE FOR THE CASE WHEN THE INTERSECTING ROUTE IS A NATIONAL HIGHWAY. THE ROUTE MARKER ASSEMBLY WILL BASICALLY BE THE SAME WHEN THE INTERSECTING ROUTE IS A STATE ROUTE OR OF SOME OTHER CLASSIFICATION, THE ONLY DIFFERENCE BEING IN THE SHIELD.



Example 2

ROUTE MARKER ASSEMBLY AT JUNCTIONS OF NATIONAL HIGHWAYS WITH OTHER NUMBERED ROUTES



DIMENSIONS IN MILLIMETRES FOR 20 cm HIGH NUMERALS											
a	a_1	b	b_1	c	c_1	d	d_1	e	e_1	f	f_1
31	11	134	5	8	173	19	72	25	300	28	59
g_1	h	h_1	i	i_1	j	j_1	k	k_1	l	m	n
52	13	23	66	141	78	200	122	64	53	58	3
	p	q	r	s	t	u	v	w	x	y	z
	61	67	69	30	50	113	128	144	102	103	9

SPACING BETWEEN STANDARD NUMERALS OF 20 cm HEIGHT

TABLE 1
NUMERAL TO NUMERAL CODE NUMBER

PRECEDING NUMERAL	FOLLOWING NUMERAL		
	1,5	2,3,6 8,9,0	4,7
1	1	1	2
2	1	2	2
3	1	2	2
4	2	2	4
5	1	2	2
6	1	2	2
7	2	2	4
8	1	2	2
9	1	2	2
0	1	2	2

TABLE 2
SPACING BETW
NUMERALS IN m

CODE NUMBER	SPA
1	
2	
4	

NOTE:-
TO DETERMINE THE PRO
SPACING BETWEEN NUMER
OBTAIN THE CODE NUMBER
TABLE 1 AND ENTER TABL
FOR THAT CODE NUMBE
SPACING IS MEASURED H
ZONTALLY FROM THE EXT
RIGHT EDGE OF THE PREC
NUMERAL TO THE EXTREM
EDGE OF THE FOLLOWING

STANDARD NUMERALS OF 20 cm HEIG

IRC : 3-1983

DIMENSIONS AND WEIGHTS
OF
ROAD DESIGN VEHICLES

(First Revision)

THE INDIAN ROADS CONGRESS

1983

MEMBERS OF THE SPECIFICATIONS AND STANDARDS COMMITTEE

DIMENSIONS AND WEIGHTS OF ROAD DESIGN VEHICLES

(First Revision)

Published by
THE INDIAN ROADS CONGRESS
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1983

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Reprinted : November, 1954
First Revision : August, 1983

Printed at PRINTAID, New Delhi-20

DIMENSIONS AND WEIGHTS OF ROAD DESIGN VEHICLES

1. INTRODUCTION

1.1. The object of framing this Standard is to lay down a basis for designing road components. The dimensions and weights of vehicles are cardinal factors in the design of road elements. The width of the design vehicle has a bearing on the width of traffic lanes and that of shoulders. The height of the vehicle affects the clearance to be provided in designing road under-bridges, electrical service lines, and other overhead structures. The overall length of the vehicle (including trailer and semi-trailer combinations) has to be taken into consideration in designing horizontal curves and vertical curves, as also in framing safety regulations for passing and overtaking. The axle load affects the design of the thickness of pavement, whereas the total weight of the vehicle governs limiting gradients.

1.2. The Indian Roads Congress Standard on Dimensions and Weights of Road Design Vehicles was first published in January, 1954. When the question of metricization of this standard was taken up, it was felt that since by that time considerable changes had taken place in the design and construction of motor vehicles and concept of geometric and structural design of the highway system both in this country and abroad, there was need of its wholesale revision.

Accordingly, a revised draft for the Standard was prepared by L.R. Kadiyali. This was modified in the Ministry of Shipping and Transport (Roads Wing) considering the current amendments to the Indian Motor Vehicles Act 1939 and the latest trends on the subject both in this country and abroad. The modified document was considered by the Specifications and Standards Committee in their meeting held at New Delhi on the 24th May, 1983. The draft as approved with a few changes by the Specifications and Standards Committee was later approved by the Executive Committee and the Council in their meetings held on the 21st July and the 21st August 1983 respectively for being published as a standard of the Indian Roads Congress.

2. SCOPE

2.1. The Standard shall be applied in designing all road elements except culverts and bridges, the latter being governed by the IRC Bridge Codes.

2.2. For purposes of this Standard, three types of commercial vehicles have been recognised :

(i) Single unit
(ii) Semi-trailer
(iii) Truck-trailer combination.

The selection of the vehicle type for design of a road would depend upon terrain conditions, economic justification, importance of the road and similar other considerations.

As a general guide, roads in steep and mountainous terrain need not be designed for truck-trailer combination and may only be designed for single unit vehicle and, where economically feasible, for semi-trailers.

Subject to the above, such of the maximum dimensions and weights out of those specified here shall be used that have the severest effect in the design of any road component. All road components, to be newly built or improved, shall be so designed that they are initially adequate or capable of being made adequate subsequently when the necessity arises, for the movement of vehicles conforming to this Standard and selected for design of the road.

3. DEFINITIONS

3.1. Axle

The common axis of rotation of one or more wheels, whether power driven or freely rotating, and whether in one or more segments, and regardless of the number of wheels carried thereon.

3.2. Axle Group

An assemblage of two or more consecutive axles considered together in determining their combined load effect on a pavement structure.

3.3. Gross Weight

The weight of a vehicle and/or vehicle combination without load plus the weight of any load thereon.

3.4. Length, Overall

The total longitudinal dimension of any vehicle or combination of vehicles, including any load or load-holding devices thereon.

3.5. Height, Overall

The total vertical dimension of any vehicle above the ground surface including any load and load holding device thereon.

3.6. Semi-Trailer

A vehicle designed for carrying persons or property and drawn by a truck-tractor on which part of its weight and load rests.

3.7. Single Axle

An assembly of two or more wheels whose centres are in one transverse vertical plane or may be included between two parallel transverse vertical planes one metre apart extending across the full width of the vehicles.

3.8. Tandem Axle

Any two or more consecutive axles whose centres are more than 1.2 m but not more than 2.5 m apart and are individually attached to and/or articulated from a common attachment to the vehicle including a connecting mechanism to equalise the load between axles.

3.9. Tandem Axle Weight

The total weight transmitted to the road by two or more consecutive axles whose centres may be included between parallel transverse vertical planes spaced not less than 1.2 m but not more than 2.5 m apart, extending the full width of the vehicle.

3.10. Trailer

A vehicle designed for carrying persons or goods and drawn by a motor vehicle which carries no part of the weight and load of the trailer on its own wheels.

3.11. Truck

A motor vehicle designed, used, or maintained primarily for the transportation of goods.

3.12 Truck-Tractor

A motor vehicle designed for drawing other vehicles, but not for a load other than part of the weight of the vehicle and load drawn.

3.13. Truck-Trailer Combination

A truck or a tractive unit with a trailer.

3.14. Width Overall

The total outside transverse dimension of a vehicle including any load or load holding devices thereon, but excluding approved safety devices and tyre bulge due to load.

4. NOTATIONS FOR VEHICLE TYPES

The Figure shows the outline of the vehicle types covered by this Standard. The first digit indicates the number of axles of the truck or truck-tractor. The letter "S" indicates a semi-trailer and the letter immediately following an "S" indicates the number of axles on the semi-trailer. Any digit other than the first in a combination, when not preceded by "S" indicates a trailer and the number of its axles. For instance, a 2-S2 combination is a two-axle truck-tractor with a tandem-axle semi-trailer. Combination 2-2 is a two-axle truck with a two-axle trailer.

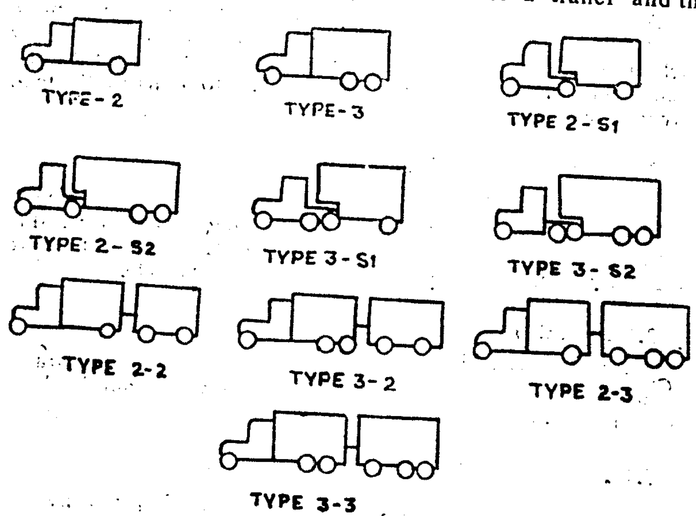

Fig. Vehicle Types

5. DIMENSIONS OF ROAD DESIGN VEHICLES

5.1. Width

No vehicle shall have a width exceeding 2.5 m.

5.2. Height

No vehicle other than a double-decker bus shall have a height exceeding 3.8 m for normal application and 4.2 m when carrying ISO series 1 freight containers. Double decker buses may, however, have a height not exceeding 4.75 m.

5.3. Length

5.3.1. The maximum overall length of a single unit truck, exclusive of front and rear bumpers, having two or more axles, shall be 11 m.

5.3.2. The maximum overall length of a single unit bus, exclusive of front and rear bumpers, having two or more axles shall be 12 m.

5.3.3. The maximum overall length of a truck-tractor semi-trailer combination, exclusive of front and rear bumpers, shall be 16 m.

5.3.4. The maximum overall length of a truck-trailer combination, exclusive of front and rear bumpers, shall be 18 m.

5.3.5. No combination of vehicles shall comprise more than two vehicles.

6. MAXIMUM PERMISSIBLE WEIGHTS

6.1. Single Axle Weight

The total gross weight imposed on the highway by a single axle fitted with dual wheels shall not exceed 10.2 tonnes. In the case of axles with single wheels, the axle weight shall not exceed 6 tonnes.

6.2. Tandem Axle Weight

The total gross weight imposed on the highway by two axles in tandem articulated from a common attachment to the vehicle or individually attached to the vehicles and spaced not less then 1.2 m but not more than 2.5 m apart, shall not exceed 18 tonnes.

6.3. Maximum Permissible Gross Weight

The maximum permissible gross weight for a given vehicle or vehicle combination would be equal to the sum of the individual single axle and tandem axle weights indicated above. For typical vehicles, maximum permissible gross weights are given in the Table.

TABLE : MAXIMUM PERMISSIBLE GROSS WEIGHTS AND MAXIMUM AXLE WEIGHTS OF TRANSPORT VEHICLES

Vehicle type	Maximum gross weight (tonnes)	Maximum axle weight (tonnes)			
		Truck/Tractor		Trailer	
		FAW	RAW	FAW	RAW
Type 2 (Both axles single tyre)	12	6	6		
Type 2 (FA-Single tyre RA-Dual tyre)	16.2	6	10.2		
Type 3	24	6	18 (TA)		
Type 2-S1	26.4	6	10.2		10.2
Type 2-S2	34.2	6	10.2		18 (TA)
Type 3-S1	34.2	6	18 (TA)		10.2
Type 3-S2	42	6	18 (TA)		18 (TA)
Type 2-2	36.6	6	10.2	10.2	10.2
Type 3-2	44.4	6	18 (TA)	10.2	10.2
Type 2-3	44.4	6	10.2	10.2	18 (TA)
Type 3-3	52.2	6	18 (TA)	10.2	18 (TA)

FA — Front Axle
RA — Rear Axle
FAW — Weight on Front Axle
RAW — Weight on Rear Axle
TA — Tandem axle fitted with 8 tyres.

IRC:5-1998

STANDARD SPECIFICATIONS
AND
CODE OF PRACTICE
FOR
ROAD BRIDGES

SECTION I

General Features of Design

(Seventh Revision)

THE INDIAN ROADS CONGRESS
1998

STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION I

General Features of Design

(Seventh Revision)

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1998

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Reprinted : November, 1969
Fifth Revision : December, 1970
Reprinted : June, 1973
Reprinted : October, 1975
Reprinted : July, 1977 (Incorporates Amendment No. 1-February, 1977)
Reprinted : June, 1980 (Incorporates Amendment No. 2-November, 1977 and Amendment No.3-January, 1979)
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Reprinted : September, 1993.
Reprinted : January, 1996
Seventh Revision : December, 1998
Reprinted : September, 2000

Printed at M/s Dee Kay Printers, New Delhi-110015
(1000 copies)

MEMBERS OF THE BRIDGE SPECIFICATIONS AND STANDARDS COMMITTEE
(As on 12.3.97)

* ADG(B) being not in position. The meeting was presided by Shri A.D. Narain, DG(RD) & Addl. Secretary to the Govt. of India, Ministry of Surface Transport

CONTENTS

Section I
GENERAL FEATURES OF DESIGN

INTRODUCTION

The Bridge Code in outline form was originally drafted in 1944-45 by the Bridges Sub-Committee. The Code was redrafted by the Office of the Consulting Engineer (Roads) in consultation with the members of the Bridges Committee and was circulated to the Chief Engineers of the Public Works Departments of all States in India. It was also discussed at the Indian Roads Congress Session at Jaipur held in 1946. An expanded Bridges Committee modified the draft in the light of the comments received from the Chief Engineers of States, the discussions at the Jaipur Session and the discussions at the Bridges Committee meeting held from time to time and this Code was first published in January, 1956.

Some changes had later been approved by the Bridges Committee in the light of subsequent discussions at the Committee meetings. The Second and Third Revisions were published including the approved changes.

The Executive Committee of the Indian Roads Congress approved the publication of the Fourth Revision in metric units. This Code was revised by the Bridges Committee, and was later published as the Fifth Revision.

Subsequently the Sixth revision was brought out based on the provisions contained in IRC:78-1983, Standard Specifications and Code of Practice for Road Bridges, Section VII - Foundations and substructure.

The General Design Features Committee (B-2) (personnel given below) in its meeting held on 21.11.96 finalised the draft "General Features of Designs" (7th Revision).

A.D. Narain	..	Convenor
A.K. Banerjee	..	Member-Secretary

Members

S.S. Chakraborty	G.R. Haridas
D.T. Grover	M.K. Mukherjee
D.K. Kanhere	A. Chattopadhyaya
S.B. Kulkarni	P.L. Manickam
M.R. Kachhwaha	CE (Design), PWD, Gujarat
A.K. Saxena	P.K. Saikia
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Vijay Kumar	A.K. Mookerjee
Dr. B.P. Bagish	Dr. G.P. Saha

Corresponding Members

B.C. Roy	S. Sengupta

Ex-Officio Members

The President, IRC (M.S. Guram)	The DG(RD) (A.D. Narain)

The Secretary, IRC
(S.C. Sharma)

The Draft was approved by the Bridge Specifications and Standards Committee and the Executive Committee in their meetings held on 12.3.97 and 29.3.97 respectively.

The draft was approved by the Council in their meeting held at Aizawl on 17.4.97.

This publication is meant to serve as a guide for engineers, engaged in the design and/or construction of road bridges. The provisions herein shall be used with discretion and care shall be taken to ensure that the stability and soundness of the structures designed and/or constructed as per these provisions are satisfactory.

The design and construction of road bridges require an extensive and thorough 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.

100. SCOPE

This Code deals with the general features of design of road bridges and the recommendations of this Code shall apply to all types of bridges constructed for use by road traffic or other moving loads.

101. DEFINITIONS

The following definitions shall be applicable for the purpose of this and other sections of the IRC Standard Specifications and Code of Practice for Road Bridges.

101.1. Bridge

Bridge is a structure having a total length of above 6 metres between the inner faces of the dirt walls for carrying traffic or other moving loads over a depression or obstruction such as channel, road or railway. These bridges are classified as minor and major bridges as per classification given below :

(a) Minor Bridge : A minor bridge is a bridge having a total length of upto 60 m.

(b) Major Bridge : A major bridge is a bridge having a total length of above 60 m

Bridges shall be graded as important essentially on the basis of the seriousness of the consequences of their distress/failure and the extent of remedial measures involved.

101.1.1. Culvert

Culvert is a cross-drainage structure having a total-length of 6 metres or less between the inner faces of the dirt walls or extreme ventway boundaries measured at right angles thereto.

101.1.2. Foot bridge

A foot bridge is a bridge exclusively used for carrying pedestrians, cycles and animals.

101.1.3. High level bridge

A high level bridge is a bridge which carries the roadway above the highest flood level of the channel.

101.1.4. **Submersible bridge/vented causeway**

A submersible bridge/vented causeway is a bridge designed to be overtopped during floods.

101.2. **Channel**

A channel means a natural or artificial water course.

101.3. **Clearance**

Clearance is the shortest distance between the boundaries at a specified position of a bridge structure.

101.4. **Freeboard**

Freeboard at any point is the difference between the highest flood level after allowing for afflux, if any, and the formation level of road embankment on the approaches or top level of guide bunds at that point.

101.5. **Highest Flood Level (H.F.L.)**

Highest flood level is the level of the highest flood ever recorded or the calculated level for the design discharge.

101.6. **Low Water Level (L.W.L)**

Low water level is the level of the water surface obtaining generally in the dry season and shall be specified in case of each bridge.

101.7. **Length of a Bridge**

The length of a bridge structure will be taken as the overall length measured along the centre line of the bridge between inner faces of dirtwalls.

101.8. **Linear Waterway**

Linear waterway of a bridge is the width of the waterway between the extreme edges of water surface at the highest flood level measured at right angles to the abutment faces.

101.9. **Effective Linear Waterway**

Effective linear waterway is the total width of the waterway of the bridge at HFL minus the effective width of obstruction. The effective width of obstruction is to be worked out as per Clause No. 104.6.

101.10. **Safety Kerb**

A safety kerb is a roadway kerb for occasional use of pedestrian traffic.

101.11. **Width of Carriageway**

The width of carriageway is the minimum clear width measured at right angles to the longitudinal centre line of the bridge between the inside faces of roadway kerbs or wheel guards.

101.12. **Width of Footway or Safety Kerb**

The width of footway or safety kerb shall be taken as the minimum clear width any where within a height of 2.25 metres above the surface of the footway or safety kerb, such width being measured at right angles to the centre line of the bridge. Fig. 1.

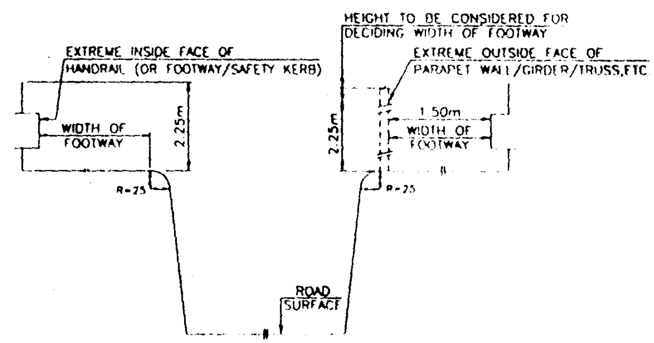


Fig. 1. Width of Footway (Clause 101.12)

101.13. **Super Elevation (Cant or Banking)**

Super elevation is the transverse inclination given to the cross-section of a carriageway on a horizontal curve in order to reduce the effects of centrifugal force on a moving vehicle.

102. COLLECTION OF DATA

All detailed information for a complete and proper appreciation of the bridge project shall be included in the project documents. Generally, the following information shall be furnished.

102.1. General Data including Maps, Plans and Topographical Features

102.1.1. An index map to a suitable small scale (toposheets in scale one cm to 500 m or 1/50,000 would do in most cases) showing the proposed location of the bridge, the alternative sites investigated and rejected, the existing means of communication, the general topography of the country, and the important towns, villages etc. in the vicinity.

102.1.2. A contour survey plan of the stream showing all topographical features and extending upstream and downstream of any of the proposed sites, to the distances shown below, (or such other greater distances as the engineer responsible for the design may direct) and to a sufficient distance on either side to give a clear indication of the topographical or other features that might influence the location and design of the bridge and its approaches. All sites for crossings worth consideration shall be shown on the plan.

102.1.2.1. 100 m for catchment areas less than 3 square km (scale not less than 1 cm to 10 m or 1/1000)

102.1.2.2. 300 m for catchment areas of 3 to 15 square km (scale not less than 1 cm to 10 m or 1/1000)

102.1.2.3. One and a half km or the width between the banks, whichever is more, for catchment areas more than 15 square km (scale not less than 1 cm to 50 m or 1/5000)

102.1.2.4. In case of meandering rivers, the provisions made in Clauses 102.1.2.1, 102.1.2.2 and 102.1.2.3 may be suitably reviewed.

Note: In difficult country and for crossings over artificial channels, the engineer responsible for the design may permit discretion to be used regarding these limits of distances, provided that the plans give sufficient information on the course of the channel and the topographical features near the bridge site.

102.1.3. A site plan, to a suitable scale, showing details of the site selected and extending not less than 100 metres upstream and downstream from the centre line of the crossing and covering the approaches to a sufficient distance, which in the case of a major bridge, shall not be less than 500 metres on either side of the channel. In case the river is meandering in the vicinity of the bridge site, the course of the river extending a suitable distance not less than two loops on either side of the proposed crossing shall be plotted on the site plan. The following information shall be indicated on the site plan.

102.1.3.1. The name of the channel or bridge and of the road and the identification number allotted to the crossing with the location (in kilometres) at the centre of crossing.

102.1.3.2. The direction of flow of water at maximum discharge and, if possible, the extent of deviation at lower discharges.

102.1.3.3. The alignment of existing approaches and of the proposed crossing and its approaches.

102.1.3.4. The angle and direction of skew if the crossing is aligned on a skew.

102.1.3.5. The name of the nearest inhabited identifiable locality at either end of the crossing on the roads leading to the site.

102.1.3.6. Reference and R.L. of permanent stations and bench marks used for layout duly connected to G.T.S. benchmark, wherever available.

102.1.3.7. The location and identification number of the cross-section and longitudinal section taken within the scope of the site plan, and the exact location of their extreme points.

102.1.3.8. The location of trial pits or borings, each being given an identification number and connected to the datum.

102.1.3.9. The location of all nullahs, buildings, places of worship, wells, burial grounds, outcrops of rocks and other possible obstructions, which may affect the approach alignment.

102.1.4. Cross-section of the channel at the site of the proposed crossing and a few cross-sections at suitable distances both upstream and downstream (at least two cross-sections, one upstream and other downstream of the proposed site), all to a horizontal scale not less than 1 cm to 10 m (1/1000) and vertical scale of not less than 1 cm to 1 m (1/100) recording the bed levels alongwith the corresponding flood levels and indicating the following information.

102.1.4.1. The bed levels upto the top of banks and the ground levels to a sufficient distance beyond the edges of the channels, with levels at intervals sufficiently close to give a clear outline of markedly uneven features of the bed or ground showing right and left banks and names of villages on each side.

102.1.4.2. The nature of the existing surface soil in bed, banks and approaches and the location and depth of trial pits or borings with their respective identification number.

102.1.4.3. The highest flood level and the low water level.

102.1.4.4. For tidal streams, record of the tidal information, over as long a period as possible, including any local information specific to the site of work. The form given below is recommended for presenting such a record.

Highest high water (HHW)
Mean high water springs (MHWS)
Mean high water (MHW)
Mean high water neaps (MHWN)
Mean sea level (MSL)
Mean low water neaps (MLWN)
Mean low water (MLW)
Mean low water springs (MLWS) Chart Datum
Lowest low water (LLW)
In coastal areas which are prone to cyclone and storms, increase in water level due to storm surge.
For bridges across sea maximum wave height above MSL.

102.1.5. A longitudinal section of the channel, showing the site of the bridge with the highest flood level, the low water level (also the highest high tide level and the lowest low tide level for tidal channels), and the bed levels at suitably spaced intervals along the approximate centre line of the deep water channel between the approximate points to which the survey plan required in Clause 102.1.2 extends. The horizontal scale shall be the same as for the survey plan and the vertical scale not less than 1 cm to 10 m or 1/1000.

102.2. Alternative Bridge Sites and Selection of a particular Bridge Site

102.2.1. A brief description of the reasons for selection of a particular site for the crossing accompanied, if necessary, with typical cross-sections of the channel at alternative sites investigated and rejected.

102.2.2. The cardinal principles to be kept in view at the time of selection of a particular bridge site including river training works are to provide a suitable crossing consistent with safety and economy and acceptable detour from the existing road alignment. The following shall be the guiding considerations in this regard :-

i) Bridge upto a length of 60 m

The location shall generally be governed by the approach alignment with minimum shifting for improvement of geometrics, if required, unless there are special bridge design problems.

ii) Bridges having total lengths between 60 m and 300 m.

Requirement of a suitable bridge site and proper alignment of approaches should be considered together and the most suitable site selected.

iii) Bridges having total lengths in excess of 300 m.

The requirement of the most suitable site shall have over-riding consideration and the site so selected shall regulate the approach alignments.

102.3. Hydraulic Data for particular Bridge Site Selected

102.3.1. The size, shape and surface characteristics of the catchment including percolation and interception.

102.3.2. The slope of the catchment, both in longitudinal and cross directions.

102.3.3. The possibility of subsequent changes in the catchment like aforestation, deforestation, urban development, extention of or reduction in cultivated area etc.

102.3.4. Storage areas in the catchment, artificial or natural.

102.3.5. The intensity, frequency, duration and distribution of rainfall in the catchment giving maximum in 24 hours and in one hour and average annual rainfall characteristics alongwith relevant meteorological records.

102.3.6. Hydrographs for one or more years, if possible, and in the absence of such data, fluctuations of the water level observed during different months of the year.

102.3.7. The highest flood level and the year of its occurrence delimiting the areas flooded. If the flood level is affected by backwater, details of the same.

102.3.8. A chart of the period of high flood levels for as many years as the relevant data has been recorded.

102.3.9. The influence of afflux on areas in the vicinity likely to be affected.

102.3.10. Low water level.

102.3.11. The design discharge (Clause 103), the linear waterway (Clause 104) and corresponding average velocity of flow.

102.3.12. The observed maximum depth of scour with corresponding level and details of obstruction or any other special causes responsible for the scour

102.3.13. History of hydraulic functioning of existing bridge, if any, under flood like distribution of flow, general direction of river course through the structure, afflux, extent and magnitude of flood, effect of backwater, if any, aggradation/degradation of the bed, evidence of scour, damage to structure and adjacent property, maintenance problems and records of any other bridges across the same river in the vicinity etc. These observations may be supplemented by photographic documentation.

102.4. Geological and Seismological Data for particular Bridge Site Selected

102.4.1. The nature and properties of the existing soil in bed, banks and approaches, with trial pit or bore hole sections showing the levels,

nature and properties of the various strata to a sufficient depth below the level suitable for foundations and the safe intensity of pressure on the foundation soil (as far as practicable, the spacing of trial pits or bore holes shall be such as to provide a full description of all substrata layers along the whole length and width of the crossing).

102.4.2. Proneness of the site to artesian condition, earthquake disturbance and its magnitude.

102.5. Sub-surface Data

Sub-surface exploration, sampling, in-situ testing and laboratory tests for determining the design parameters for the bridge foundation shall be carried out in accordance with Clause 704 of I.R.C. Bridge Code, Section VII (IRC:78).

102.6. Environmental Data

Information regarding usual annual temperature range, susceptibility to severe storms, cyclones, tidal effects etc., and probable wind velocity, rainfall characteristics, indication of period of rainy seasons, relative humidity and salinity or presence of harmful chemicals in the subsoil, water and environment.

102.7. Loading and Other Data

102.7.1. The load for which the bridge is to be designed shall be as per relevant clauses of IRC:6 with any specific variation from those clauses, if required to cover special load conditions.

102.7.2. Special local conditions like traffic intensity and pattern to enable the designer to fix the loading to be adopted for the footpath and to fix number of traffic lanes required.

102.7.3. Utilities or services, if any, to be carried over the bridge and if so, nature thereof (e.g. Telephone Cables, Water Conduits, Gas Pipes, etc.) and relevant information regarding size, arrangement, etc.

102.7.4. The minimum vertical and horizontal clearances required for any special requirement like navigation, aggradation of the bed, etc., and the basis on which it is suggested.

102.7.5. An index map showing location of rail and road bridges, if any, crossing the same channel or its tributaries within a reasonable distance of the proposed bridge and a note (with sketches or drawings) giving important details of such bridges.

102.7.6. A note stating whether large trees and rolling debris etc., are likely to float down the channel at the proposed bridge site.

102.7.7. Details of protective works, including guidebunds, if any, provided for structures across the same stream, upstream or downstream alongwith data of their behaviour, depth of scour etc.

102.8. Any other additional information including further details regarding floods and bridges in the vicinity alongwith their performance which may be considered essential for complete and proper appreciation of the project.

103. DETERMINATION OF DESIGN DISCHARGE

103.1. The design discharge for which the waterway of the bridge is to be designed, shall be based on maximum flood discharge of 50 years' return cycle. In case where the requisite information is not available, the design discharge shall be the maximum estimated discharge determined by consideration of the following or any other rational method.

103.1.1. From the records available, if any, of discharge observed in the stream at the site of the bridge, or at any other site in its vicinity.

103.1.2. From the rainfall and other characteristics of the catchment :

i) By the use of an empirical formula, applicable to that region, or

ii) By a recognised method, provided it is possible to evaluate for the region concerned the various factors employed in that method.

103.1.3. By the area velocity method with the help of hydraulic characteristics of the channel.

103.1.4. By unit Hydrograph Method (See Appendix-1). Flood estimation reports in respect of total 21 climatic sub-zones (Appendix 1(a)) in the country have been prepared based on the hydro-meteorological data collected for selected catchments of areas varying from 25 to 1500 sq.km. and are available with the Director, Hydrology (small catchments), Central Water Commission, Sewa Bhavan,

R.K. Puram, New Delhi. The methodology recommended in the relevant sub-zone report pertaining to a particular region may be followed for assessment of maximum discharge for the design of bridges.

103.2. Where possible, more than one method shall be adopted, results compared, and the maximum discharge fixed by judgement by the engineer responsible for the design. The bridge shall be designed for this maximum discharge. However, for catchment areas covered by sub-zones mentioned in Appendix 1(a), the maximum discharge shall be assessed on the basis of the flood estimation report for the said sub-zone.

103.3. Freak flood discharges or exceptional discharges of high intensity due to the failure of a dam or tank constructed upstream of the bridge need not be catered for and the maximum estimated discharge from the catchment area or normal peak flood discharge from the dam/spillway (to be ascertained from the Irrigation Authorities), whichever is more, shall be considered for design of the bridge.

Note: In cases where the design discharge cannot be properly quantified and in spill zones of rivers known for freak floods, the abutments may be designed as abutment piers to leave scope for future extension.

104. DETERMINATION OF LINEAR WATERWAY AND EFFECTIVE LINEAR WATERWAY

104.1. For artificial channels (irrigation, navigation and drainage), the effective linear waterway should be adequate to pass the full discharge at designed velocity but concurrence shall invariably be obtained from the authority controlling the channel. If it is proposed to flume the channel at the site of the bridge, this fluming shall be subject to the consent of the same authority and in accordance with the essential requirements.

104.2. For non-meandering channels in alluvial beds but with well defined banks and for all natural channels in beds with rigid inerodible boundaries, the linear waterway shall be the distance between banks at that water surface elevation, at which the designed maximum discharge determined in accordance with Clause 103, can be passed without creating harmful afflux.

104.3. For natural channels in alluvial beds and having undefined banks, the effective linear waterway shall be determined from the design discharge using some accepted rational formula at the discretion of the engineer responsible for the design. One such formula for regime conditions is :

$$W = C\sqrt{Q}$$

where

W = regime width in metres (equal to effective linear waterway under regime condition)

Q = the design maximum discharge in m^3/sec;

C = a constant usually taken as 4.8 for regime channels but it may vary from 4.5 to 6.3 according to local conditions.

104.4. If the river is of a flashy nature and the bed does not submit readily to the scouring effects of the flood, the waterway should be determined by the area velocity method taking into account the design flood level and its waterspread, the characteristics of the bed materials and the water surface slope.

104.5. In cases of bridges located in tidal zones, where it is decided to adopt measures likely to affect the volume of the tidal flow and other characteristics of the tide, it shall be ensured that no port or harbour or other installations in the proximity of the bridge are adversely affected.

104.6. For calculating the effective linear waterway (as defined in Clause 101.9), the width of obstruction due to each pier shall be taken as the mean submerged width of the pier and its foundation upto the normal scour level. The obstruction at the ends due to the abutments or pitched slopes duly protected shall be ignored.

104.7. For unstable meandering rivers flowing through a number of subchannels separated by land or shallow section of nearly stagnant water and having width much in excess of the regime width, it is necessary to constrict the channel by providing training works to prevent the main channel from wandering about freely and for minimising the resultant oblique attack on bridge foundations and approaches. The extent of constriction and the design of training works in such cases should, preferably, be decided on the basis of model studies, keeping in view the ultimate economy, safety, durability and aiming at optimal recurring maintenance needs of the structure.

104.8. **Effect of presence of Dams, Barrages, Wiers, Sluice Gates etc.**

104.8.1. Presence of dams, barrages, wiers, sluice gates etc., on the rivers affect their hydraulic characteristics like causing obliquity and

concentration of flow, scour, silting of bed, change in flow levels, bed levels etc. These effects shall be considered in the design of bridges depending upon whether the proposed site of the bridge is upstream or downstream of a dam or a barrage, or wier etc.

104.8.2. Since the above parameters depend on many factors which are varying from site to site, no uniform guidelines can possibly be laid down. Such problems may be jointly taken up with the concerned Departments and suitable provision made in the bridge design.

105. SPACING AND LOCATION OF PIERS AND ABUTMENTS

105.1. Piers and abutments shall be so located as to make the best use of the foundation conditions available.

105.2. Keeping in view Clause 105.1 above, the number of supports and their locations shall be so fixed as to provide the most economical design of the bridge and at the same time satisfy special requirements, if any, for navigation, railways or other crossings in consultation with the concerned authorities, floating logs or debris and bridge aesthetics, etc.

105.3. The alignment of the piers and abutments shall, as far as possible, be parallel to the mean direction of flow in the channel, as well as the direction of other piers and abutments in the vicinity, but provision shall be made against harmful effects on the stability of the bridge structure and on the maintenance of the channel banks, contiguous to the bridge due to any temporary variations in the direction and velocity of the current.

105.4. Placing a pier at the deepest portion of an active channel may be avoided by suitably adjusting the number and length of the spans.

106. VERTICAL CLEARANCES

106.1. In the case of a channel, vertical clearance is usually the height from the design highest flood level with afflux of the channel to the lowest point of the bridge superstructure at the position along the bridge where clearance is being denoted.

106.2. Clearance shall be allowed according to navigational or anti-obstruction requirements in consultation with the concerned authorities. Where these considerations do not arise, the vertical clearance shall ordinarily be as follows :

106.2.1. For openings of high level bridges, which have a flat soffit or soffit with a very flat curve, the minimum clearance shall be in accordance with the following table. The minimum clearance shall be measured from the lowest point of the deck structure inclusive of main girder in the central half of the clear opening unless otherwise specified.

Discharge in m^3/sec	Minimum vertical clearance in mm.
Upto 0.3	150
Above 0.3 & upto 3.0	450
Above 3.0 & upto 30.0	600
Above 30.0 & upto 300	900
Above 300 & upto 3000	1200
Above 3000	1500

106.2.2. For arched openings of high level bridges having overhead decking, the clearance below the crown of the arch shall not be less than one tenth of the maximum depth of water plus one-third of the rise of the arch intrados.

106.2.3. In structures provided with metallic bearings, no part of the bearings shall be at a height less than 500 mm above the design highest flood level taking into account afflux.

106.2.4. In the case of artificial channels having controlled flows and carrying no floating debris, the engineer responsible for design may, at his discretion, provide less vertical clearance than that specified in Clauses 106.2.1 & 106.2.2 above.

106.2.5. In the case of bridges in sub-mountainous region and across aggrading rivers, silting of the bed of the river should also be taken into consideration while fixing the vertical clearance.

107. FREEBOARD

107.1. The freeboard for the approaches to high level bridges shall not be less than 1750 mm.

107.2. For aggrading rivers in Himalayan foot-hills and flood-prone areas of North-Eastern States, North Bengal etc., the freeboard shall be suitably increased.

108. RESTRICTED WATERWAYS

Restriction of the waterway as determined by Clause 104 may be done giving careful consideration to the resulting effects based on site conditions in the individual cases.

When the waterway is restricted to such an extent that the resultant afflux will cause the channel to discharge at erosive velocities, protection against damage by scour shall be afforded by providing deep foundations, curtain or cut-off walls, rip-rap, bed pavement, bearing piles, sheet piles or other suitable means. Likewise, embankment slopes adjacent to all structures subjet to erosion shall be adequately protected by pitching, revetment walls or other suitable measures.

109. OBSTRUCTIONS AND RIVER TRAINING

Obstruction in the channel bed likely to divert the current or cause undue disturbed flow or scour and thereby endanger the safety of the bridge shall be removed as far as practicable from within a distance upstream and downstream of the bridge not less than the length of the bridge subject to a minimum of 100 metres in each direction. Attention shall be given to river training and protection of banks over such lengths of the river as required.

110. DETERMINATION OF THE MAXIMUM DEPTH OF SCOUR

110.1. The probable maximum depth of scour to be taken for the purpose of designing foundations for piers, abutments and river training works shall be estimated after considering all local conditions over a reasonable period of time. The following may help in deciding the maximum scour depth.

110.1.1. Wherever possible, soundings for the purpose of determining the depth of scour shall be taken in the vicinity of the site proposed for the bridge. Such soundings are best taken during or immediately after a flood before the scour holes have had time to silt up appreciably. Allowance shall be made in the observed depth for increased scour resulting from :

i) the design discharge being greater than the flood discharge.

ii) the increase in velocity due to obstruction in flow caused by construction of the bridge.

iii) the increase in scour in the proximity of piers and guide bunds.

110.1.2. Discharge for Design of Foundations and Protection Works

To provide for an adequate margin of safety, the foundation and protection works shall be designed for a larger discharge which should be a per cent over the design discharge given in Clause 103, for which reference may be made to the relevant provision contained in IRC:78 (Bridge Code Section VII).

110.1.3. The following theoretical method may be adopted when dealing with the natural channels flowing in non-co-herent alluvium for the estimation of mean depth of scour 'Dsm' in metres.

$$Dsm = 1.34 \left[\frac{D_b^2}{k_{sf}} \right]^{1/3}$$

where

D_b = the discharge in Cumecs per metre width. The value of 'D_b' shall be the maximum of the following :

i) the total design discharge divided by the effective linear waterway between abutments or guide bunds, as the case may be.

ii) the value obtained taking into account any concentration of flow through a portion of the waterway assessed from the study of the cross-section of the river. Such modification of the value may not be deemed applicable to minor bridges of length upto 60 m.

iii) actual observations, if any.

k_{sf} = the silt factor for a representative sample of the bed materials obtained upto the maximum anticipated scour level and is given by the expression $1.76\sqrt{dm}$ where 'dm' is the weighted mean diameter of the bed material in mm.

Note : i) The effective linear waterway shall be determined in accordance with Clause 104.6 and in no case shall exceed the value assessed as per Clause 104.3.

ii) A typical method of determining 'dm' is set forth in Appendix-2.

iii) The value of 'k_{sf}' for various grades of bed materials normally encountered are given below for general guidance only

Type of bed material	weighted mean diameter of particle in mm, dm	Value of silt factor ksf
fine silt	0.081	0.500
fine silt	0.120	0.600
fine silt	0.158	0.700
medium silt	0.233	0.850
standard silt	0.323	1.000
medium sand	0.505	1.250
coarse sand	0.725	1.500
fine bajri & sand	0.988	1.750
heavy sand	1.290	2.000

The value to be adopted for the purpose of design should be determined after laboratory testing of the representative samples of bed materials collected during the sub-soil exploration.

110.1.4. If a river is of flashy nature and the bed does not lend itself readily to the scouring effect of floods, the formula for Dsm given in the Clause 110.1.3 shall not apply. In such cases, the maximum depth of scour shall be assessed from actual observations.

110.1.5. For bridges located across streams having bouldery beds, there is yet no rational formula for determining scour depth. However, the formula given in Clause 110.1.3 may be applied with a judicious choice of value for D_b and K_{sf} and the results compared with the actual observations at site or from experiences on similar structure nearby and their performance and decision taken based on sound engineering judgement. If a pucca floor at bed is provided, it is essential to check the hydraulic performance of these structures under various flow conditions to ensure that a standing wave is not formed on the downstream side which may result in very heavy scour. It is also essential to check the usual scour that may take place downstream of a bed flooring and to make adequate provision for the same. If it is not possible to increase the waterway and avoid the formation of a standing wave, a depressed pucca floor on the downstream may be provided to contain the standing wave within the floor.

111. KERBS

111.1 The section given in Fig. 2 is indicative and shall be generally adopted for the road kerb. For bridges across deep gorges, major rivers, open sea, breakwaters etc. where crash barriers are not provided, the road kerb shall be considered as fully unsurmountable. In such cases, the kerb section shown in Fig. 2 shall be suitably modified.

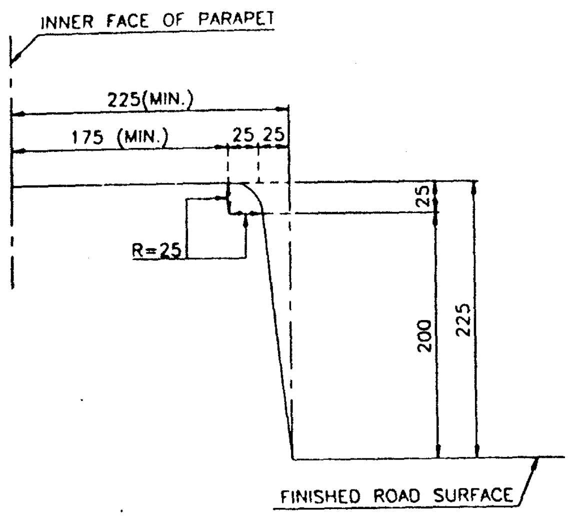


Fig. 2. Outline of Road Kerb (Clause 111.1)
(All Dimensions are in Millimetres)

111.2. The section of the kerb should be so designed that it would be safe for vertical and horizontal loads as per relevant Clauses in IRC:6.

111.3. A safety kerb will have the same outline as that of a roadway kerb except that the top width shall not be less than 750 mm.

112. WIDTH OF CARRIAGEWAY, FOOTPATH AND MEDIAN

112.1. For high level bridges constructed for the use of road traffic only, the width of carriageway shall not be less than 4.25 m for a single lane bridge and 7.5 m for a two-lane bridge and shall be increased by 3.5 m for every additional lane of traffic for a multiple lane bridge. Road bridges shall provide for either one lane, two lanes or multiple of two lanes. Three-lane bridges with two directional traffic shall not be constructed. If a median/central verge is constructed in a wide bridge thus providing two separate carriageways, the carriageway on each side of the verge shall provide for at least two lanes of traffic and width thereof shall individually

comply with the minimum requirements stipulated above. The width of central/ verge/median, when provided, shall not be less than 1.2 metres.

In addition, cross-sections of 2-lane and multi lane bridges shall satisfy the following :

i) For all minor bridges of total length upto 60 m, width between the outermost faces of the bridge shall be equal to the full formation width of the approaches subject to a minimum of 10 m for hill roads/other district roads and 12 m for other cases.

ii) For two lane bridges having total length more than 60 metres in non-urban situations, the width of the bridge shall provide for 7.5 m carriageway plus a minimum of 1.5 m wide footpath on either side, wherever required.

iii) For two lane bridges having total length more than 60 m in urban situations, the overall width between the outermost faces of the bridge shall be equal to the full formation width of the approaches.

iv) For multi-lane bridges, in both urban and non-urban situations, the overall width between the outermost faces of the bridge shall be the same as the full formation width of the approaches. Wherever footpaths are provided, their width shall not be less than 1.5 m. The width of the median in the bridge portion shall be kept same as that in the approaches.

v) For bridges on expressways, the provisions in sub clause (iv) shall be satisfied and the carriageway width shall not be less than the width of carriageway in the approaches plus hard shoulders.

112.2. For bridges carrying combined road and tramway or any other special type of traffic, the widths indicated in Clause 112.1 shall be modified to suit these special requirements.

112.3. Vented causeways/submersible bridges shall provide for at least two lanes of traffic as specified in Clause 112.1 above unless one lane of traffic is specially permitted in the design.

112.4. For a bridge on a horizontal curve, the roadway width shall be increased suitably to conform to the requirements stipulated in the relevant IRC Road Standards.

112.5. When a footpath is provided, its width shall not be less than 1.5 metres. For urban and populated areas having large concentration of pedestrian traffic, the width of the footpath shall be suitably increased.

113. SUPER ELEVATION

113.1. The super elevation on the deck of a bridge on a horizontal curve shall be provided in accordance with the relevant IRC Road Standards.

113.2. Due allowance shall be made for the effect of superelevation on the stresses in the various members of the bridge.

113.3. If there is a change of gradient on the bridge deck, suitable vertical curve shall be introduced conforming to the stipulations contained in IRC:SP-23.

114. CLEARANCES

114.1. The minimum horizontal clearance shall be the clear width and the minimum vertical clearance the clear height available for the passage of traffic.

114.2. The minimum horizontal and vertical clearances for single lane and multiple lane bridges with vehicular traffic shall be as shown in Fig. 3.

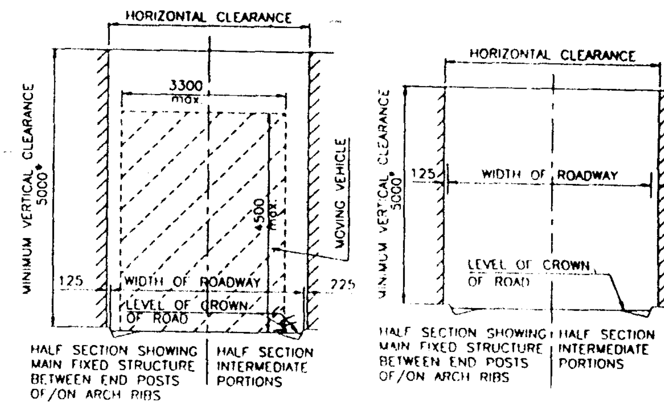
114.3. For Road Over Bridges across railway lines, horizontal and vertical clearance shall be governed by the requirements of the Railways as per their specifications.

114.4. Unless otherwise specified, bridges shall have all their parts constructed to secure the minimum clearances for traffic given in Fig. 3.

114.5. For footways and cycle tracks, a minimum vertical clearance of 2.25 metres shall be provided.

114.6. For a bridge constructed on a horizontal curve with superelevated road surface, the horizontal clearance shall be increased on the side of the inner kerb by an amount equal to 5 metres multiplied by the superelevation. The minimum vertical clearance shall be measured from the superelevated level of the roadway. Extra horizontal clearance required

for the superelevation will be over and above the increase in width required on a curve under Clause 112.4.



(a) Single lane bridge (b) Multiple lane bridge

* In urban area provide 5500mm

FIG. 3. Clearance Diagram (Clause 114.2)
(All Dimensions are in Millimetres)

114.7. Vertical and lateral clearances at underpasses shall be provided in accordance with the stipulations contained in IRC:54-Lateral and Vertical Clearances at Underpasses for Vehicular Traffic.

115. RAILINGS, PARAPETS OR GUIDE POSTS

115.1. High Level Bridges

115.1.1. Substantial railings or parapets along each side of the bridge shall be provided for the protection of traffic. Consideration shall be given to the architectural features of the railing or parapet to obtain proper proportioning of its various members and its harmony with the structure and the environment as a whole. For bridges situated in severe marine environment, solid wall type parapets along each side of the bridge shall be preferred for better serviceability. Consideration shall be given also to avoid, as far as is consistent with safety and appearance, obstruction of the view from the passing motor cars.

115.1.2. Railings or parapets shall have a minimum height above the adjacent roadway or footway safety kerb surface of 1.1 metres less one half the horizontal width of the top rail or top of the parapet. For bridges exceeding 300 metres in length, the height of railings, determined in the manner stipulated above, shall be increased by 100 mm. The forces to be considered in design shall be as per relevant stipulations of IRC:6. For R.O.Bs across railway lines, these requirements shall be governed by those for railways' safety.

115.1.3. Where a road provided with cycle tracks goes over a bridge and the cycle track is located immediately next to the bridge railing or parapet, the height of the railing or parapet shall be kept 15 cm higher than that reqired as per Clause 115.1.2 above.

115.1.4. The clear distance between the lower rail and the top of the kerb shall not exceed 150 mm unless the space is filled by vertical or inclined members, the clear distance between which is not more than 150 mm. The strength of the lower rail shall be at least as great as that of the top rail. The space between lower rail and the top rail shall be filled by means of vertical, horizontal or inclined members, the clear distance between which shall be fixed with due regard to the safety of persons and animals using the structure.

115.1.5. Guard rails shall be provided at the high approaches. The design, layout and materials chosen for the rails shall suitably blend with the surroundings.

115.2. Submersible Bridges

115.2.1. Railings shall be either collapsible or removable.

115.2.2. Collapsible railing shall be used where it is necessary to put up the railings immediately when the bridge is opened to traffic after a submerging flood has receded. Care shall be taken in the structural design of these railings to ensure that they sit well in their grooves and are not liable to be dislodged by floods.

115.2.3. Removable type of railings may be adopted when there is no danger to the traffic using the bridge for short period without railings. Care shall be taken in the structural design of these railings to ensure that the various members are interchangeable and can be easily removed and refitted.

115.2.4. Collapsible or removable railings shall be designed to resist as far as possible the same forces as specified in Clause 115.1.2 for railings or parapets on high level bridges.

115.3. Vented Causeways

Guide posts/stones may be used in lieu of railings, if the submergence of the road surface over the causeway is so frequent as to render the use of removable or collapsible railings unsatisfactory.

115.4. Crash Barriers

115.4.1. Suitably designed crash barriers shall be provided at the following situations to safeguard against errant vehicles :

i) Multi-lane bridges and bridges on expressways

ii) Flyovers and interchanges in urban situations

iii) R.O.Bs across railway lines

iv) Open sea, breakwaters, deep valleys/gorges

For other cases, decision may be taken by the appropriate authority duly considering the importance of the structure and the level of safety warranted.

115.4.2. Crash barriers, when provided, shall be essentially of the following types :

i) **Vehicle Crash Barrier** : Provided for bridges without footpaths to contain errant vehicles.

ii) **Combination Railing/Vehicle Pedestrian Crash Barrier** : Provided for bridges with footpaths to contain vehicles and safeguard pedestrians.

iii) **High Containment Barriers** : Provided mainly on bridges over busy railway lines, complex interchanges and other similar hazardous and high risk locations to contain errant vehicles and redirect them back into the traffic flow.

115.4.3. Typical shapes and dimensional details of crash barriers and their locations on the bridge decks with or without footpaths are shown in Fig. 4. These may be suitably modified and augmented depending on the developments in design and future functional requirements in individual cases.

115.4.4. Crash barriers shall be of metal or reinforced concrete and their design shall take into consideration the following factors :

i) Impact of vehicles colliding with the barrier

ii) Safety of occupants of a vehicle colliding with the barrier

iii) Safety of other vehicles near the collision site

iv) Safety of vehicles or pedestrians underneath the bridge

v) Aesthetics and freedom of view from passing vehicles

115.4.5. Crash barriers shall provide a smooth and continuous face on the traffic side and shall be suitably extended into the approaches. Exposed rail ends, posts and sharp changes in the geometry of the railings shall be avoided. Suitable reflective (luminous) devices shall be provided on the traffic face of the barrier at intervals to ensure adequate visibility during night and foggy conditions.

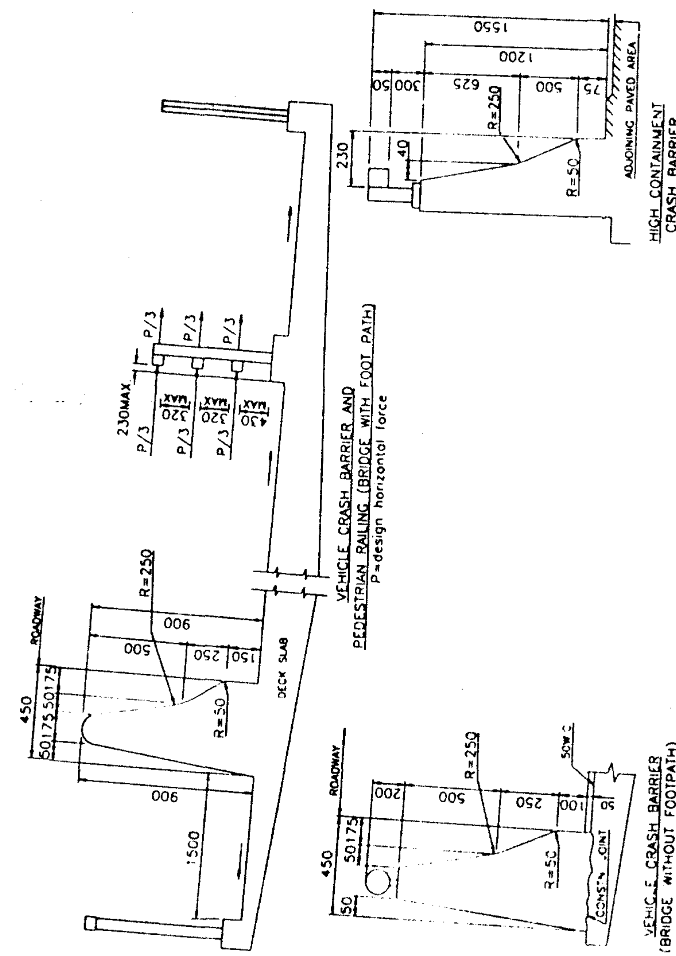


Fig. 4(a). Typical Sketches of Crash Barriers (Clause 115.4.3)
(All Dimensions are in Millimetres)

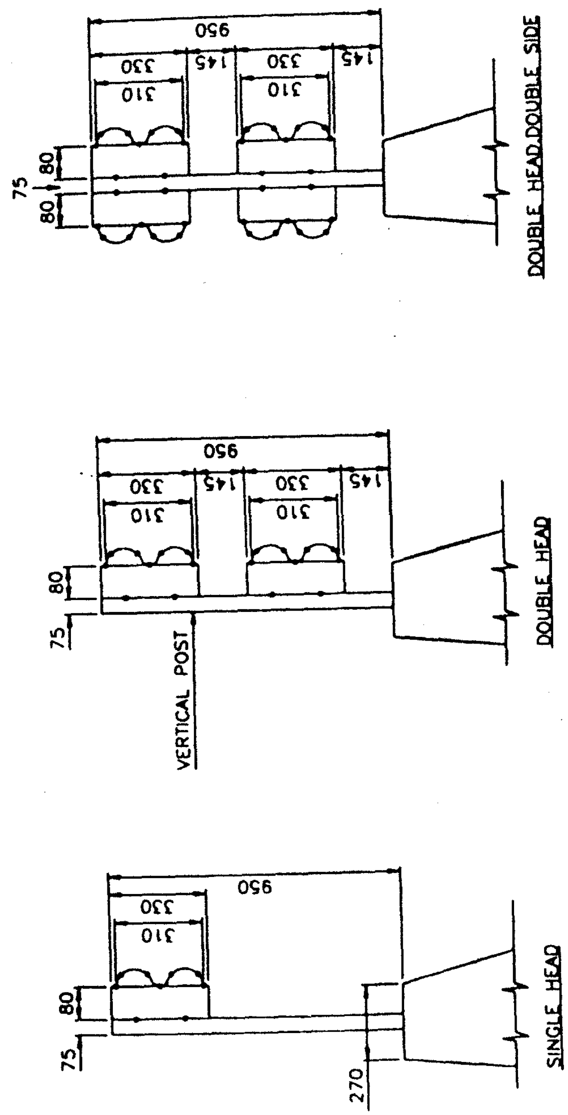


Fig. 4(b). Various Type of Crash Barriers (Clause 115.4.3)
(All Dimensions are in Millimetres)

116. DRAINAGE OF CARRIAGEWAY AND SURFACE FINISHES

116.1. The high level bridges may preferably be built in longitudinal gradient with suitably designed cross drains at abutment locations to facilitate proper drainage.

116.2. For drainage of a road over bridge/flyover, a suitably designed drainage arrangement should be provided. This may consist of vertical C.I. or rigid PVC pipes connecting the downspouts below the deck with funnels and along the pier upto ground level and eventually joined to the road drainage system. Suitable vertical recess in the piers may be provided to accommodate the drainage pipes rather than providing drip courses underneath the deck slab.

116.3. All carriageways and footpath surfaces shall have anti-skid characteristics.

117. ACCESS FOR INSPECTION AND MAINTENANCE

The design of the bridge structure shall be such as to provide for adequate access to all parts of the bridge to facilitate future inspection and maintenance operations.

118. ROAD SIGNS AND SIGNALS

118.1. General

i) All multi-lane bridges, complex interchanges and grade separated structures shall be provided with overhead signs and signals conforming to the provision contained in IRC:67. Non-luminous signs, however, shall not be permitted.

ii) The location of sign boards, barrier kerbs and supporting system should be designed and planned alongwith the project and details of the scheme shown in a drawing drawn to a scale of 1:500. Signages required on bridge decks shall be placed behind roadside barriers. The sign supports shall be structurally safe and properly anchored to the bridge deck. The bridge deck shall be designed to withstand all such loads.

iii) Provision shall be made to accommodate overhead signs or cantilever signs with requisite supporting system. If necessary, the median/railing area can be increased locally to accommodate the massive supports. Wherever signs are to be illuminated or signals to be operated, necessary arrangements shall be made for cable ducts. Typical arrangement of supports for overhead structures is shown in Fig. 5.

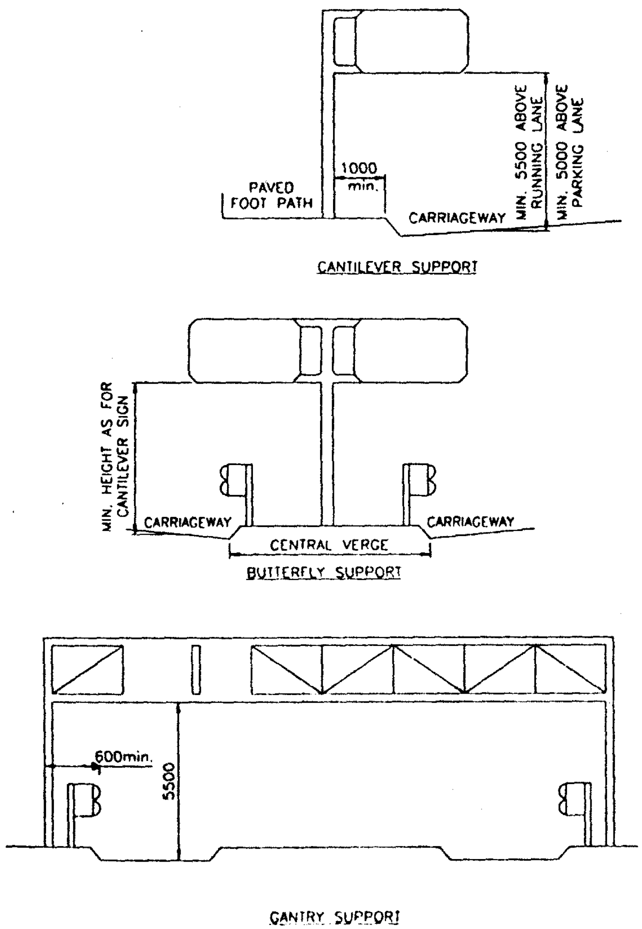


Fig. 5. Typical Arrangement of Supports for Overhead Structures
(CLause 118.1 (iii))
(All Dimensions are in Millimetres)

118.2. Vertical Clearance

The minimum vertical clearance above the roadway in any traffic lane upto the lowest point of the sign or any part of the signage structure or to lamps mounted below the sign shall be 5.5 m. In case of signages mounted over footway, shoulder or parking lane, the vertical clearance may be reduced to 5 m.

118.3. Lateral Clearance

The sign supports should be placed at a minimum 1.0 m clear distance behind the traffic face of the roadway kerbs. In longitudinal direction, it shall be placed at a minimum distance of 6 metre from the beginning of a traffic island in any high speed approach direction. On high speed roads, the support shall either be at 9 metre minimum clear distance from the edge of carriageway or adequately protected by ground fence and located at a minimum 0.6 metre clear distance behind the guard rail or parapet/crash barrier.

119. UTILITIES

Where required, provision shall be made for traction wire supports, poles or pillars for lights, trenches or other suitable places for the installation of electric or telephone conduits, water or gas pipes and other similar utilities or services with due care for durability and serviceability of the bridge and its approaches.

120. APPROACHES TO BRIDGES

120.1. The approaches on either side of a straight bridge shall have a minimum straight length of 15 metres and shall be suitably increased where necessary to provide for the minimum sight distance for the design speed. Minimum surfaced width of these straight lengths of approaches shall be equal to the carriageway width on the bridge.

Note: In difficult situations, the Engineer responsible for the design may at his discretion permit a reduction in the minimum straight length of approaches, provided reasons for making a departure from the Code are clearly recorded.

120.2. Where horizontal curves have to be provided on the approaches beyond the straight portion on either side, the minimum radius of curvature, the super elevation and transition length for various speed and the curve radii shall be provided in accordance with relevant stipulations contained in IRC:38.

120.3. If the approach is in filling, borrowpits should not be dug close to the embankment to avoid risk of parallel flow being developed which may endanger safety of the embankment. Suitable minimum distance from the toe of embankment and depth of borrow pits for the immediate approaches of the bridge may be specified for each case, depending on the size of the channel and topographical conditions of the area. In this connection, provision made in IRC:10 "Recommended Practice for Borrowpits for Road Embankments Constructed by Manual Operation" may also be kept in view.

120.4. If there is a change of gradient, suitable vertical curves shall be introduced conforming to relevant stipulations contained in IRC:23. A single vertical curve shall be provided for bridges having a total length upto 30 m.

120.5. Approaches to submersible bridges/vented causeways likely to be affected by floods shall be provided with suitably designed protective works.

121. BEARINGS AND EXPANSION JOINTS

121.1. Bearings for the bridges shall be designed for all movements and rotations as applicable and conform to the provision contained in IRC:83 Parts I & II.

121.2. To cater for expansion and contraction movements, suitably designed expansion joints shall be provided at the expansion ends of all spans and at other points where they may be necessary alongwith appropriate drainage arrangement. The number of such expansion joints shall be kept minimum, as far as practicable. Care shall be taken to ensure watertightness of the expansion joints.

122. BRIDGE FOUNDATIONS

The foundations for piers and abutments shall be at such depths that they are safe against scour and large impacts where necessary and are protected against it. They shall be taken down to a level sufficient to secure firm foundation from consideration of bearing capacity, overall stability and suitability of the strata at the founding level and upto sufficient depth below it. The foundations shall be designed in accordance with the provisions made in IRC:78.

123. ILLUMINATION OF BRIDGE DECK

123.1. Illumination for bridges, grade separators and interchanges shall be decided by the appropriate authority and conform to the following requirements :

i) Lighting masts may be of the conventional type of suitable heights not less than 5.5 metres. The design of the bridge should duly account for the above provision and the loads therefrom.

ii) The choice of lamps may be any of the following:

   a) Incandescent lamps
   b) Mixed incandescent and high pressure mercury vapour lamps
   c) High pressure mercury vapour lamps with clear or fluorescent bulbs
   d) Tubular fluorescent lamps
   e) Sodium vapour lamps
   f) Mercury-halide lamps and
   g) High Pressure sodium vapour lamps

iii) The arrangement of the masts i.e. its height and spacing should be such as to achieve a minimum level of illumination on the bridge deck of the order of 30 lux.

123.2. The installation, lighting arrangement, method of control, switches etc. shall all conform to the provision contained in IS:1944.

123.3. At highway interchanges, different lighting arrangements viz. low masts or high masts or combination of both may be considered and the one which gives optimum results from the view points of aesthetics, safety, illumination and ease of maintenance may be adopted. Light colour distinction at junctions to give an early warning signal to approaching vehicles may also be considered.

123.4. Illumination levels for the vehicular and pedestrian subways/underpasses may be kept same as those on the approaches at either end of the subway/underpass.

124. BRIDGE AESTHETICS

Visual forms of bridges, viaducts or flyover structures etc. should be selected with care to be in harmony with the general landscape with a view to maintaining the aesthetics of the surrounding. While planning such structures, the following general requirements may be kept in view :

i) Dimensions and proportions of various elements from the point of harmony and integration into the environment and being pleasing to the viewers.

ii) Symmetry of structure

iii) Surface of structure

iv) Form or appearance in totality

v) Abstract structural form of the bridge/viaduct viewed as an independent object.

APPENDIX-1
Clause 103.1.4

UNIT HYDROGRAPH METHOD

The Unit Hydrograph, frequently termed as the unit graph, is defined as the hydrograph of storm run-off at a given point in a river, resulting from an isolated rainfall of unit duration occuring uniformly over the catchment, and producing a unit run-off. The unit run-off adopted is 1 cm depth over a catchment area.

The term "Unit-Rainfall Duration" is the duration of rainfall excess resulting in the unit hydrograph. Usually, unit hydrographs are derived for specified unit durations, say, 6 hours, 12 hours, etc., and derived unit hydrographs for durations other than these are converted into unit hydrographs of the above unit durations. The duration selected should not exceed the period during which the storm is assumed to be approximately uniform in intensity over various parts of the catchment. A 6 hours unit duration is suitable and convenient for studies relating to catchments larger than 250 sq.km.

The unit hydrograph represents the integrated effects of all the basin constants, viz., drainage area, shape, stream pattern channel capacities, stream and land slopes.

The derivation and application of the unit hydrograph is based on the following principles :

1) All the characteristics of the catchment of a river are reflected in the shape of the hydrograph of run-off.

2) At a given point on a river for all storms having the same duration of rainfall excess above this point and uniformly distributed with respect to time, the discharge ordinates of the hydrograph are proportional to the total volumes of storm run-off. This implies that rainfall excess of say 2 cm within the unit of duration will produce a run-off hydrograph having ordinates twice as great as those of the unit hydrograph. Also, if individual hydrographs are obtained from separate periods of uniform rainfall excess that may occur throughout a storm period, and these are properly arranged with respect to time, the ordinates of the individual hydrographs can be added to give ordinates representing the total storm run-off hydrograph for the entire storm period.

Three methods are generally available for giving unit hydrographs at any point in a river.

i) By analysis of rainfall and run-off records for isolated unit storms;

ii) By analysis of the run-off compound hydrographs;

iii) By computation of synthetic unit hydrographs when sufficient rainfall and run-off data are not available.

The determination of design flood, after the unit hydrograph has been derived, involves the following steps :

a) Division of catchment into sub-areas, if necessary.

b) Derivation of design storm and its apportionment to sub-area.

c) Determination of minimum retention rate and calculation of rainfall excess of design storm.

d) Arrangement of design storm.

e) Application of rainfall excess to unit hydrographs for each sub-area.

f) Routing of flood for each sub-area to the point of collection of the whole catchment.

A rational determination of critical design storm for a catchment requires a comprehensive study of major storms recorded in the region and an evaluation of effects of local conditions upon rainfall rate. This is particularly necessary in the case of design storms covering a large area of several thousand square kms.

In the case of areas less than a few thousand square kms, certain assumptions can be made regarding rainfall patterns and intensity variations without being inconsistent with meteorological causes. They simplify design-storm estimation, but would entail high degree of conservation.

APPENDIX 1(a)
Clause 103.1.4

REPORTS BROUGHT OUT BY CENTRAL WATER COMMISSION

UNDER LONG TERM PLAN

Sl. No.	Name of Sub-zone	Sub-zone No.
1.	Chambal sub-zone	1(b)
2.	Betwa sub-zone	1(c)
3.	Sone sub-zone	1(d)
4.	Upper Indo-Ganga Plains sub-zone	1(e)
5.	Middle Ganga Plains sub-zone	1(f)
6.	Lower Gangetic Plains sub-zone	1(g)
7.	North Brahmaputra basin sub-zone	2(a)
8.	South Brahmaputra basin sub-zone	2(b)
9.	Mahi and Sabarmati sub-zone	3(a)
10.	Lower Narmada and Tapi sub-zone	3(b)
11.	Upper Narmada and Tapi sub-zone	3(c)
12.	Mahanadi sub-zone	3(d)
13.	Upper Godavari sub-zone	3(e)
14.	Lower Godavari sub-zone	3(f)
15.	Krishna & Pannar sub-zone	3(h)
16.	Kaveri river sub-zone	3(i)
17. 18. 19.	Eastern Coast sub-zones	4(a), 4(b) & 4(c)
20. 21.	West Coast region sub-zones	5(a) & 5(b)

APPENDIX-2
Clause 110.1.3

TYPICAL METHOD OF DETERMINATION OF WEIGHTED MEAN DIAMETER OF PARTICLES (dm)

Representative disturbed samples of bed materials shall be taken at every change of strata upto the maximum anticipated scour depth. The sampling should start from 300 mm below the existing bed. About 500 gms of each of the representative samples so collected shall be seived by a set of standard seives and the weight of soil retained in each seive is taken. The results thereof are then tabulated. A typical test result is shown below (Tables I & II)

TABLE-I

Seive Designation	Sieve Opening (mm)	Weight of Soil retained (gm)	Per cent retained
5.60 mm	5.60	0	0
4.00 mm	4.00	0	0
2.80 mm	2.80	16.90	4.03
1.00 mm	1.00	76.50	18.24
425 micron	0.425	79.20	18.88
180 micron	0.180	150.40	35.86
75 micron	0.75	41.00	9.78
Pan	-	55.40	13.21
	Total :	419.40	

TABLE-II

Sieve No. (1)	Average size (mm) (2)	Percentage of weight retained (3)	Column (2) x column (3) (4)
4.00 to 2.80 mm	3.40	4.03	13.70
2.80 to 1.00 mm	1.90	18.24	34.66
1.00 to 425 micron	0.712	18.88	13.44
425 to 180 micron	0.302	35.86	10.83
180 to 75 micron	0.127	9.78	1.24
75 micron & below	0.0375	13.21	0.495
			74.365

Weighted mean diameter $dm = \frac{74.365}{100}$

$= 0.74365$

Say 0.74

IRC:8-

TYPE DESIGNS FOR HIGHWAY KILOMETRE STONI

(Second Revision)

THE INDIAN ROADS CONGRESS
1995

TYPE DESIGNS
FOR
HIGHWAY KILOMETRE STONES

(Second Revision)

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TYPE DESIGNS FOR HIGHWAY KILOMETRE STONES

1. INTRODUCTION

1.1. The standard on Kilometre Stones was published by the Indian Roads Congress originally in 1960. It was reviewed by the Specifications and Standards Committee (personnel given on inside front and back cover) in October, 1979 with respect to the language of inscription and certain modifications were made.

1.2. The revised standard incorporating these changes was approved by the Council of Indian Roads Congress at their meeting held at Gauhati on 28th October, 1979 and is recommended for adoption on all roads in the country.

2. DESIGN OF KILOMETRE STONES

2.1. Kilometre stones shall correspond to the Type Designs shown in Plates 1, 2 and 3.

2.2. On National Highways, State Highways and Major District Roads, the kilometre stones used shall have two sizes:

(i) "ordinary kilometre stones" of smaller size vide Plate 1; and
(ii) "fifth kilometre stones" (i.e. kilometre stones installed after every five kilometres) of bigger size vide Plate 2.

2.3. On Other District Roads and Village Roads, the kilometre stones shall be uniformly of one size as shown in Plate 3.

3. MATERIALS

3.1. Kilometre stones may be made of suitable materials, available locally, such as hard stone, cement concrete etc.

4. SCRIPT AND SEQUENCE OF INSCRIPTION

4.1. Ordinary kilometre stones put up on National Highways, State Highways etc. (vide Plate 1) shall be inscribed either in Hindi or local language and indicate the name and distance to the next important (i.e. intermediate) town or the terminal/starting station as the case may be (see para 4.2). Fifth kilometre stones, on the other hand, shall be inscribed only in Roman and show the name and distance of the terminal/starting station as well as of the intermediate town (see para 4.2 and Plate 2).

4.2. The place names shall be inscribed in different scripts in the following sequence, only one script being used on any one kilometre stone:

Km No.	*Script for place names*	*Place to be shown*
0	Roman	Terminal/starting station and next important town
1	Hindi (Devnagri Script)	Next important town
2	Local Language	—do—
3	Hindi (Devnagri Script)	Terminal/starting station
4	Local Language	—do—
5	Roman	Terminal/starting station and next important town
6	Hindi (Devnagri Script)	Next important town

. . . .

. . . .

and so on, repeated in the same order.

4.3. The above order and weightage of a script may be modified by the Road Authority if considered necessary. For instance, where local script happens to be the same as Devnagri, instead of the sequence for ordinary kilometre stones given in para 4.2, the kilometrage to the next important town and the terminal/starting station could be shown on alternate stones.

4.4. On kilometre stones fixed on Other District Roads and Village Roads, the inscription may be in the official language (i.e. Hindi in Devnagri script) or the script of the recognised regional language at the discretion of the local Road Authority. Inscription in Roman script is not necessary unless the road leads to a place of tourist or archaeological interest or there are other good reasons.

4.5. In every case, the numerals on kilometre stones shall be inscribed in the international form of Indian numerals. Local or Devnagri numerals shall not be used vide Article 343 (i) of the Constitution of India.

4.6. On each kilometre stone, its number* shall be inscribed on the side of the stone facing the carriageway (see Plates 1, 2 and 3). In case of thin stones made of R.C.C. or some other material, the digits may be inscribed one below the other starting from the top. Thus 426 will be shown as:

4

2

6

5. SIZE, SHAPE AND SPACING OF LETTERS/NUMERALS

5.1. Letters and numerals on kilometre stones shall be of following size:

Height of letters for Place Name	...	80 mm
Height of numerals for Kilometrage	...	130 mm
Height of numerals for Route Numbers	...	100 mm

5.2. Shapes and dimensions of standard letters/numerals for the above sizes are given in Plates 4 and 5. For long place names, the thickness of letters and the spacing between them may be reduced suitably, however without making any change in their height.

5.3. When inscribing letters or numerals, all characters having an arc at top or bottom should be extended slightly above or below the line of the other letters/numerals. This is in accordance with the accepted practice for rounded letters and numerals.

5.4. Recommended spacing between adjacent letters/numerals is given in Table 1, and for any combination can be worked out through a two-step process described in the footnote

*This numbering is meant for the use of maintenance staff and will start with zero at the starting station, increasing progressively upto the last kilometre stone at the terminal station.

below the Table. This space is the distance measured horizontally between the extreme right edge of the preceding letter/numeral and the extreme left edge of the following letter/numeral. No part of any letter/numeral may extend into this space. The other edge clearances should be as given below :

Top	...	50 mm
Bottom	...	75 mm
Sides	...	50 mm
Spacing between lines	...	50 mm

5.5. On kilometre stones inscribed in a language other than Roman, the style of lettering shall be the one in general use. The spacing between single or compound letters shall be at least equal to the thickness of the vertical strokes/or the thickness of strokes of letters in case of scripts having no vertical strokes (such as Oriya, Telugu and Kannada).

6. COLOUR OF BACKGROUND AND OF THE INSCRIPTION

6.1. The background colour shall be white with black letters and numerals for names of stations and distances. The semi-circular portion of kilometre stones shall be painted canary yellow (I.S. Shade 309) on National Highways, brilliant green (I.S. Shade 221) on State Highways, and white on Major District Roads. Route numbers written on the semi-circular portion shall be in black colour on canary yellow and white backgrounds, and in white colour on brilliant green background.

7. PLACEMENT

7.1. Normally kilometre stones shall be located on left-hand side of the road as one proceeds from the station from which the kilometre count starts. On divided roads having a central median, kilometre stones should be provided at the left on both sides of the road i.e. independently for each direction of travel.

7.2. Kilometre stones shall be fixed at right angles to the centre line of the carriageway. On embankments, these shall be located on the edge of the roadway beyond the shoulders, if necessary on specially erected platforms. In cut sections, these shall be fixed clear of the shoulders as well as the side drains. (See Plate 6).

TABLE 1. SPACING BETWEEN STANDARD LETTERS OR NUMERALS OF DIFFERENT HEIGHTS

TABLE 1(a) LETTER TO LETTER CODE NUMBER

Preceding Letter	Following Letter: B, D, E, F, H, I, K, L, M, N, P, R, U	C, G, O, Q, S, X, Z	A, J, T, V, W, Y
A	2	2	4
B	1	2	2
C	2	2	3
D	1	2	2
E	2	2	3
F	2	2	3
G	1	2	2
H	1	1	2
I	1	1	2
J	1	1	2
K	2	2	3
L	2	2	4
M	1	1	2
N	1	1	2
O	1	2	2
P	1	2	2
Q	1	2	2
R	1	2	2
S	1	2	2
T	2	2	4
U	1	1	2
V	2	2	4
W	2	2	4
X	2	2	3
Y	2	2	4
Z	2	2	3

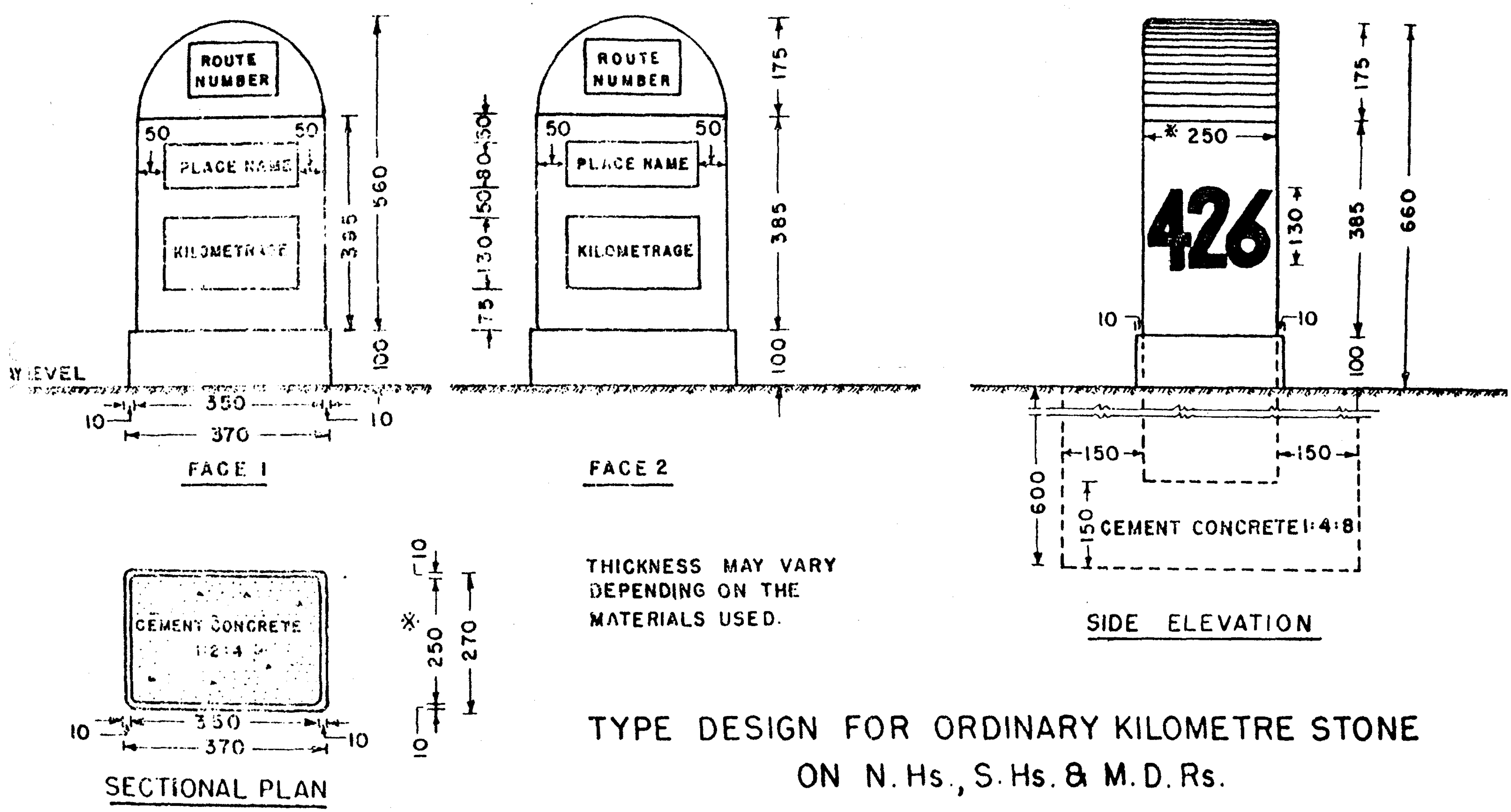
TABLE 1(b) NUMERAL TO NUMERAL CODE NUMBER

Preceding Numeral	Following Numeral: 1, 5	2, 3, 6, 8, 9, 0	4, 7
1	1	1	2
2	1	2	2
3	1	2	2
4	2	2	4
5	1	2	2
6	1	2	2
7	2	2	4
8	1	2	2
9	1	2	2
0	1	2	2

TABLE 1(c) SPACING BETWEEN LETTERS AND NUMERALS IN mm

Code Number	Letter or Numeral Height in mm: 80	100	130
1	19	21	31
2	15	19	24
3	10	13	16
4	5	6	8

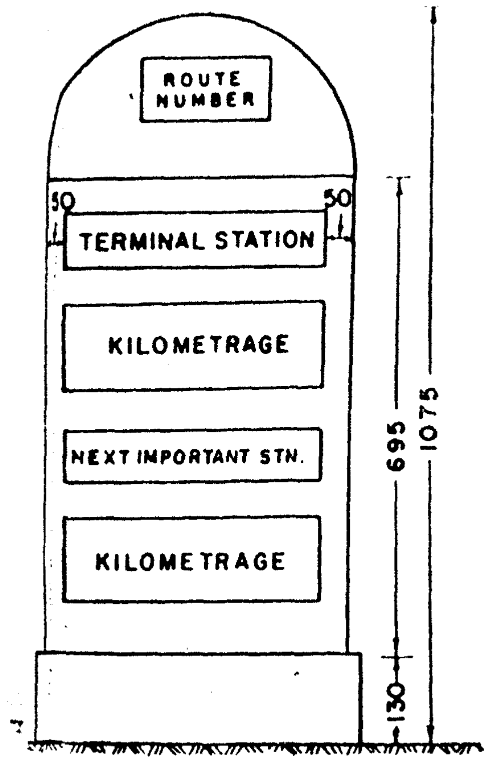
Note : To determine the proper spacing between letters or numerals, obtain the code number from Tables 1(a) and 1(b) and enter Table 1(c) for that code number to desired letter or numeral height. Spacing is measured horizontally from the extreme right edge of the preceding letter/numeral to the extreme left edge of the following letter/numeral.



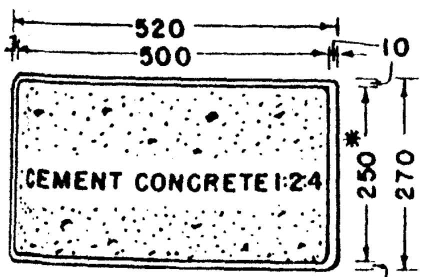
THICKNESS MAY VARY DEPENDING ON THE MATERIALS USED.

TYPE DESIGN FOR ORDINARY KILOMETRE STONE ON N.Hs., S.Hs. & M.D.Rs.

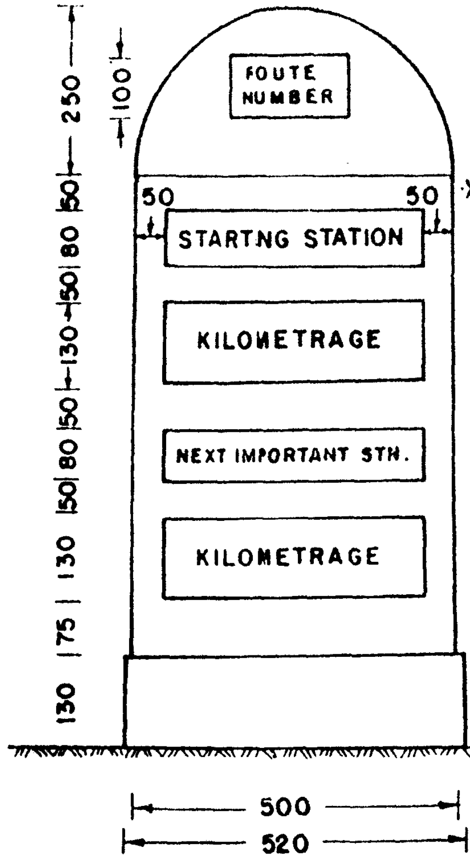
ALL DIMENSIONS IN MILLIMETRES



FACE I

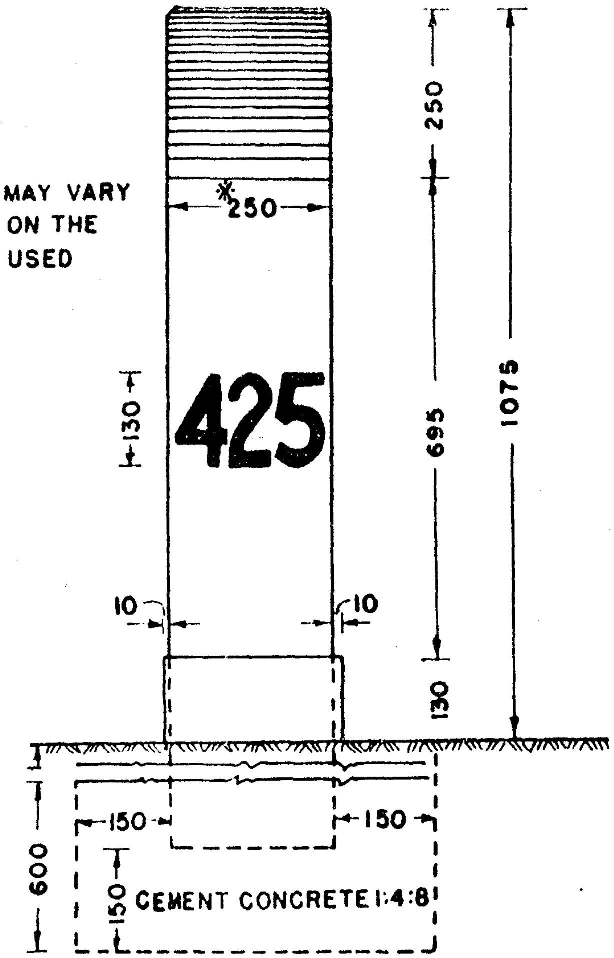


SECTIONAL PLAN



FACE 2

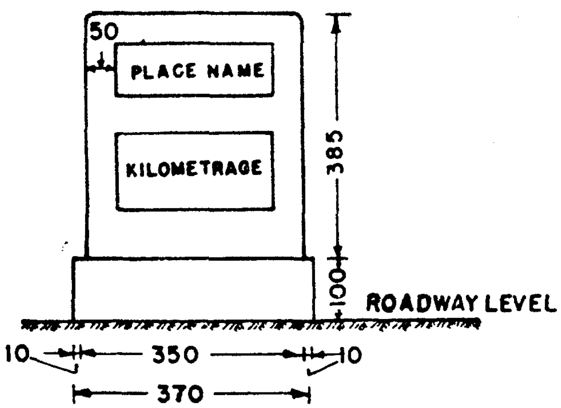
※ THICKNESS MAY VARY DEPENDING ON THE MATERIALS USED



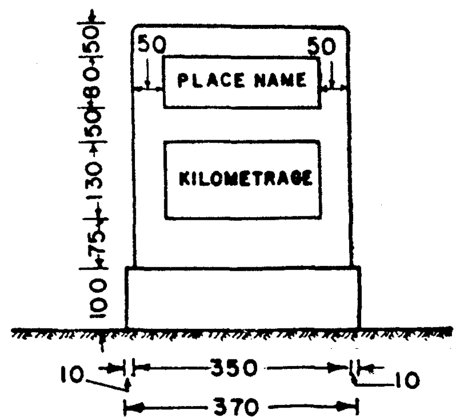
SIDE ELEVATION

TYPE DESIGN FOR THE 5 th KILOMETRE STONE
ON N. Hs., S. Hs. & M. D. Rs.

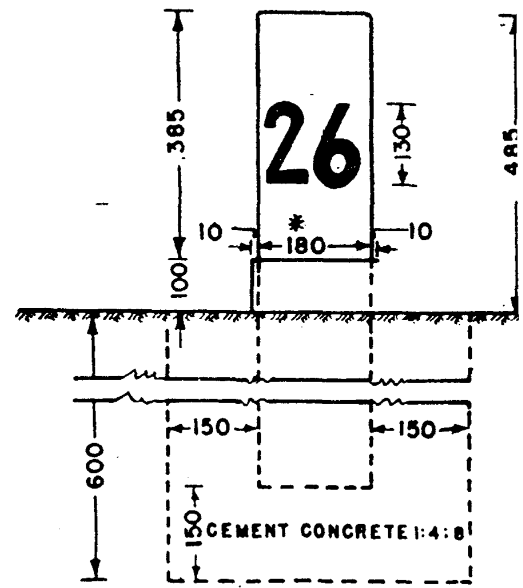
ALL DIMENSIONS IN MILLIMETRES



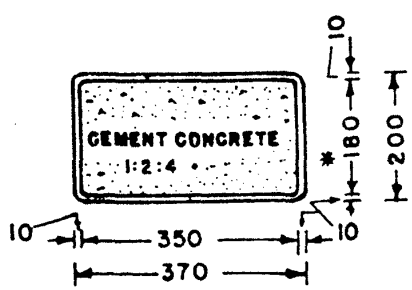
FACE I



FACE 2



SIDE ELEVATION



SECTIONAL PLAN

* THICKNESS MAY VARY DEPENDING ON THE MATERIALS USED

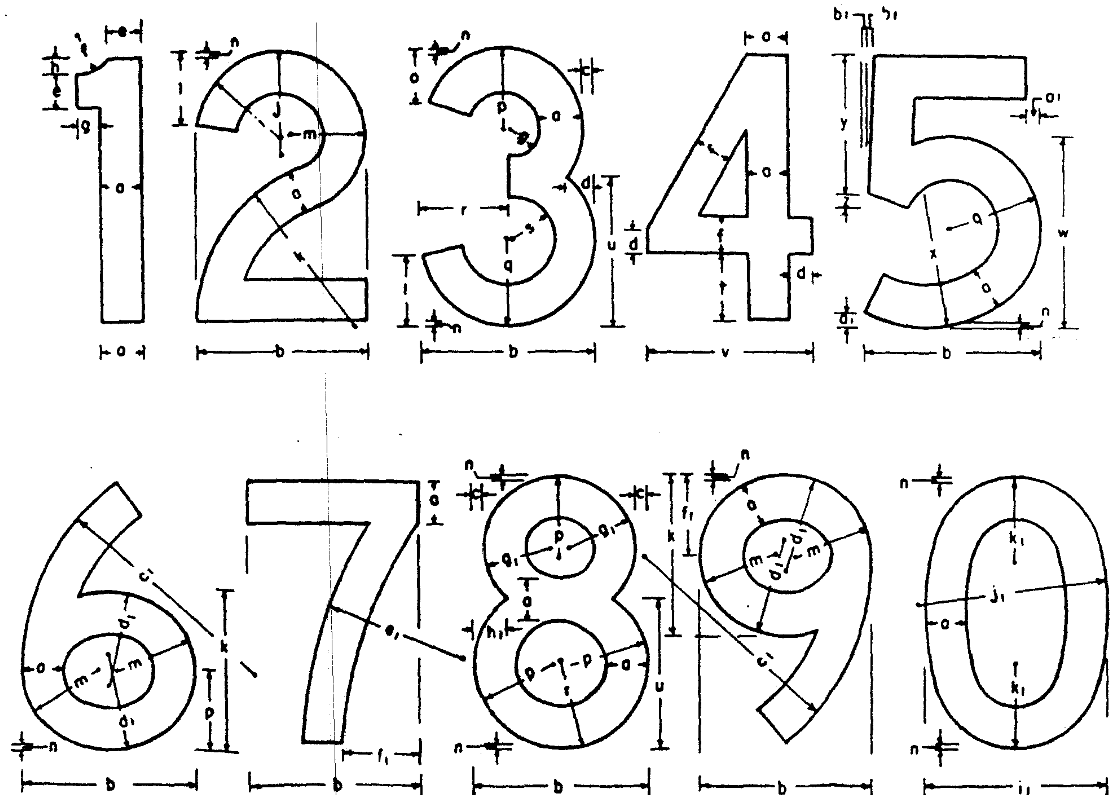
TYPE DESIGN FOR KILOMETRE STONE ON O.D.Rs. & VILLAGE ROADS

ALL DIMENSION IN MILLIMETRI

Height of letters in mm	Dimensions of standard letters in mm											
80	a	a_1	b	b_1	c	c_1	d	d_1	e	e_1	f	
	13	31	67	55	23	62	54	46	35	27	2	
	f_1	g	g_1	h	h_1	i	i_1	j	j_1	k	k_1	
	11	42	6	20	4	22	14	13	24	3	25	
	l	l_1	m	m_1	n	n_1	o	o_1	p	p_1	q	q_1
	26	3	26	60	16	10	18	70	63	51	1	68
	r	r_1	s	s_1	t	u	v	w	x	y	z	
	29	11	56	8	23	49	80	12	14	50	28	

NOTE: FOR SPACING BETWEEN LETTERS, SEE TABLE 1

STANDARD LETTERS



NOTE: FOR SPACING BETWEEN NUMERALS, SEE TABLE 1.

Dimensions of standard numerals in mm

Height of Numerals in mm	a	a_1	b	b_1	c	c_1	d	d_1	e
100	16	5	67	2	4	89	9	36	13
130	20	7	87	3	5	116	12	47	16

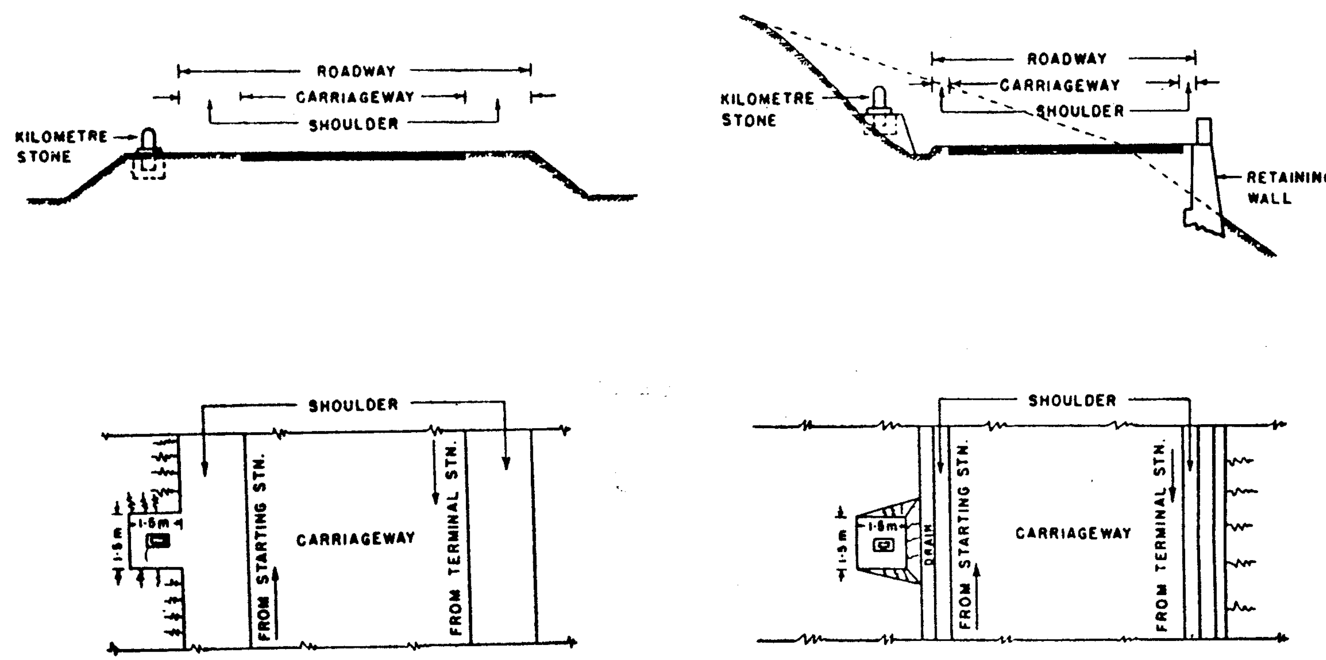
Height of Numerals in mm	e_1	f	f_1	g	g_1	h	h_1	i	i_1
100	150	14	30	9	26	6	12	33	70
130	195	18	39	11	34	8	15	43	91

Height of Numerals in mm	j	j_1	k	k_1	l	m	n	o	p	q
100	39	100	61	32	27	29	2	20	30	34
130	51	130	79	42	35	38	2	26	40	44

Height of Numerals in mm	r	s	t	u	v	w	x	y	z
100	34	15	25	56	64	72	51	52	5
130	45	19	33	73	83	93	66	67	6

STANDARD NUMERALS

Plate 6



(a) IN EMBANKMENT

(b) IN CUTTING

LOCATION OF KILOMETRE STONES

IRC : 9-19

TRAFFIC CENSUS
ON
NON-URBAN ROADS

(First Revision)

THE INDIAN ROADS CONGRESS
1989

IRC : 9-1972

TRAFFIC CENSUS

ON

NON-URBAN ROADS

(*First Revision*)

Published by

THE INDIAN ROADS CONGRESS

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New Delhi-11

1989

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Printed at M/s Dee Kay Printers, New Delhi-110015
(500 copies)

TRAFFIC CENSUS ON NON-URBAN ROADS

1. INTRODUCTION

1.1. Periodic traffic census are a valuable source of basic data for highway planning. As such, these should be a regular feature in all highway departments.

This Standard was originally published in 1960. The revised Standard was considered and approved by the Specifications and Standards Committee in their meeting held on the 18th and 19th November, 1971 and by the Executive Committee in their meeting held on the 26th and 27th April, 1972. Later, it was approved for publication as the finalised Standard by the Council at their 78th meeting held at Nainital on the 10th July, 1972.

2. SCOPE

2.1. It is desirable that traffic census operations be carried out in a uniform manner throughout the country.

2.2 The repetition of census operations, on the scale recommended here, should normally be limited to important trunk routes like the National Highways, State Highways and Major District Roads.

3. SELECTION OF CENSUS POINTS

3.1. Judicious location of traffic count stations is crucial to the success of a census programme. For trunk routes serving inter-city traffic, it is relevant that the census sites should be fixed well away from all urbanised developments and villages. In particular, sites within zone of influence of towns where there may be a regular flow of commuter traffic must be avoided. If need be, additional stations could be set up for these zones.

3.2. Every road should be divided into convenient sections, each carrying approximately similar traffic between points of substantial traffic change. Count stations should be set up for each section. The limits of the sections could generally be the important towns along the road or major roads intersecting or taking off from the highway in question.

3.3. Since division of the highway into sections and fixation of census points for them are decisions of lasting significance, these should be taken at a senior level in each highway department after considering the traffic pattern along the entire route.

3.4 Every subsequent census should be taken at the same locations. New stations could, of course, be added as and when needed.

4. FREQUENCY AND DURATION OF CENSUS

4.1. Traffic should be counted at each point at least twice every year. One count should be taken during the peak season of harvesting and marketing and the other during the lean season. Each time the count should be made for a full week spread over 7 consecutive days and 24 hours of each day.

4.2. Traffic census should not generally encompass abnormal conditions of traffic like a fair or exhibition. In such cases, the count in the area should be postponed by a few days till normalcy returns.

5. RECORDING OF DATA

5.1. For the purpose of counts, a day could be divided into three shifts of 8 hours each and separate enumerators with a supervisor assigned for each shift. Enumerators should be literate persons with preferably middle or matriculation level qualification. It may be worth-while to specially train supervisors to go round from one census point to the next and initiate the other staff new to this kind of work.

5.2. Recording should be done for each direction of travel separately. For this it will be necessary to divide staff for every shift into two parties.

5.3. A field data sheet form for the manual recording of hourly flows is given in Plate I. Before start of enumeration, it should be ensured by the supervisors that the information in the form at the top is duly filled in by the enumerators.

5.4. In each hourly column, the traffic should be recorded by making tally marks in the five-dash system (vertical strokes for the first four vehicles, followed by an oblique stroke for the fifth vehicle so as to depict a total of five). Hourly totals should be made at the end of the shift.

6. COMPILATION OF DATA

6.1. A form for daily traffic summary is shown at Plate II. Information in this sheet should be compiled from the field data sheets. The highest peak hour traffic in the day for fast as well as slow vehicles may be highlighted in the summary sheets by drawing a firm line in red around the figures in the appropriate column.

6.2. The information collected in the daily summary sheets should be transferred to the weekly traffic summary form shown at Plate III. The average daily traffic for the week should then be determined and indicated in the space provided for that in the form.

6.3. The daily and weekly traffic summaries should be prepared in quadruplicate so that one copy could be kept by the Executive Engineer incharge of maintenance of the road and the other copies sent to the planning division at the headquarters office which in turn would remit this information to the other agencies concerned, e.g., the Roads Wing of the Ministry of Shipping and Transport in the case of National Highways. The field data sheets should be preserved as permanent record for at least five years.

6.4. An index map indicating the location of the census site should be attached to the traffic summary sheets.

PLATE 1

TRAFFIC CENSUS
FIELD DATA SHEET

DATE AND DAY OF WEEK :- ______________

DIRECTION OF TRAFFIC UP/DOWN FROM ______________ TO ______________

ROAD CLASSIFICATION :- ______________
KILOMETRAGE/MILEAGE:- ______________
ROUTE NO. (IF ANY) :- ______________
DISTRICT ______________ STATE ______________

TYPE OF VEHICLES / HOUR OF COUNT	CARS, JEEPS, VANS THREE WHEELERS ETC.	BUSES	TRUCKS	MOTOR CYCLES AND SCOOTERS	ANIMAL DRAWN VEHICLES**	CYCLES	OTHERS (Specify)	REMARKS, INCLUDING WEATHER CONDITIONS
1	2	3	4	5	6	7	8	9
FROM.....HRS* TO ......HRS								
HOURLY TOTAL								
FROM.....HRS* TO ....HRS								
HOURLY TOTAL								
FROM.....HRS* TO ....HRS								
HOURLY TOTAL								
FROM.. ..HRS* TO.........HRS								
HOURLY TOTAL								

NOTES:-

1. RECORD TRAFFIC VOLUME IN COLUMNS 2 TO 8 BY MAKING TALLIES IN THE FORM OF VERTICAL STROKES FOR FIRST FOUR VEHICLES AND DRAWING AN OBLIQUE STROKE FOR EVERY 5TH AS SHOWN WITHIN BRACKETS(ꟷ).
2. SOME ROADS CARRY APPRECIABLE VOLUME OF OTHER TRAFFIC LIKE CYCLE RICKSHAWAS. RECORD THE VOLUME OF SUCH VEHICLES IN COLUMN 8 AFTER SPECIFYING THE VEHICLE TYPE.

* THE HOUR OF COUNT SHOULD BE ENTERED BEFORE THE START OF ENUMERATION. PM HOURS SHOULD BE RECORDED AFTER ADDING 12 TO THE ACTUAL HOUR, FOR EXAMPLE 2 PM SHOULD BE RECORDED AS 14·00 HRS

** IF FELT NECESSARY BY HIGHWAY AUTHORITY, THIS COLUMN COULD BE SUB-DIVIDED INTO TWO FOR RECORDING THE VOLUME OF "PNEUMATIC-TYRED" AND "IRON-TYRED" VEHICLES SEPARATELY.

NAME AND SIGNATURE OF ENUMERATOR WITH DATE ______________

NAME AND SIGNATURE OF SUPERVISOR WITH DATE ______________

TRAFFIC CENSUS
DAILY TRAFFIC SUMMARY

FROM .... HRS ON* .... TO .... HRS ON* ....

DIRECTION OF TRAFFIC: FROM .... TO .... (UP)

FROM .... TO .... (DOWN)

ROAD CLASSIFICATION :- ....

KILOMETRAGE/MILEAGE :- ....

ROUTE NO. (IF ANY) :- ....

DISTRICT:- .... STATE ....

COUNT HOUR	FAST VEHICLES											SLOW VEHICLES										REMARKS
	CARS, JEEPS, VANS, THREE WHEELERS ETC.		BUSES		TRUCKS		MOTOR CYCLES AND SCOOTERS		TOTAL FAST			ANIMAL DRAWN VEHICLES		CYCLES		OTHERS (SPECIFY)		TOTAL SLOW			
	UP	DOWN	UP	DOWN	UP	DOWN	UP	DOWN	UP	DOWN	TOTAL COL. 10 & 11	UP	DOWN	UP	DOWN	UP	DOWN	UP	DOWN	TOTAL COL. 19 & 20	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
0000 - 0700																					
0700 - 0800																					
0800 - 0900																					
0900 - 1000																					
1000 - 1100																					
1100 - 1200																					
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2100 - 2200																					
2200 - 2300																					
2300 - 2400																					
0000 - 0100																					
0100 - 0200																					
0200 - 0300																					
0300 - 0400																					
0400 - 0500																					
0500 - 0600																					
TOTAL																					
TOTAL UP & DOWN FOR VEHICLE TYPE																					

* ENTER DATE AND DAY OF WEEK

NAME AND SIGNATURE OF SUPERVISOR WITH DATE

PLATE III

TRAFFIC CENSUS
WEEKLY TRAFFIC SUMMARY

FROM ________ (DATE) TO ________ (DATE)
DETAILS OF CENSUS POINT:- ________

ROAD CLASSIFICATION :- ________
KILOMETRAGE/MILEAGE:- ________
ROUTE NO. (IF ANY):- ________
DISTRICT ________ STATE ________

PERIOD				FAST VEHICLES					SLOW VEHICLES				REMARKS
FROM		TO											
DATE	HOUR	DATE	HOUR	CARS, JEEPS, VANS, THREE WHEELERS	BUSES	TRUCKS	MOTOR CYCLES & SCOOTERS	TOTAL (COL. 5 TO 8)	ANIMAL DRAWN VEHICLES	CYCLES	OTHERS (SPECIFY)	TOTAL (COL. 10 TO 12)	
1	2	3	4	5	6	7	8	9	10	11	12	13	14
TOTAL FOR THE WEEK													
AVERAGE DAILY TRAFFIC FOR THE WEEK													

NAME AND SIGNATURE OF SUPERVISOR WITH DATE ________

RECOMMENDED PRACTICE
FOR
2 CM THICK BITUMEN AND TAR CARPETS

(SECOND REVISION)

THE INDIAN ROADS CONGRESS
1988

IRC : 14-1977

RECOMMENDED PRACTICE
FOR
2 CM THICK BITUMEN AND TAR CARPETS

(SECOND REVISION)

Published by
THE INDIAN ROADS CONGRESS
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RACTICE FOR 2 CM THICK BITUMEN AND TAR CARPETS

INTRODUCTION

as originally printed in 1963. The revised metric units after being approved by the Committee was processed by the Specifi- mmittee and then approved by the Executive as approved by the Council of the Indian meeting held at Darjeeling on the 5th and ing issued as the finalised standard of the

The present revised standard besides containing a few minor lments approved by the Bituminous Pavements Committee ontains revised tolerances for surface evenness as approved Council of the Indian Roads Congress in their 87th meeting t Madras on the 27th August 1976 and incorporated in the pecial Publication 16 "Surface Evenness of Highway Pave-

SCOPE

This is a recommended practice for laying 2 cm thick n and tar carpets. The type and grade of binder is left to the on of the Engineer-in-charge, so as to be in conformity with cumatic, traffic and terrain conditions and based on past successful practices.

A. 2 CM THICK BITUMEN CARPET

A.1. **Materials**

A.1.1. **Binder** : The binder shall be one of the following :

(i) a straight run bitumen of suitable penetration grade complying with IS: 73—1961;

(ii) a cutback bitumen of suitable viscosity complying with IS: 217—1961 or IS: 454—1961 or other approved cutback.

A.1.2. **Coarse aggregates** : The aggregates shall consist of , hard, tough, durable and of y shall be crushed rock, gravel, e free of elongated or flaky pieces, soft and disintegrated material, vegetable or other deleterious matter. The aggregates shall also satisfy the following properties:

Property	Value	Method of test
1. Abrasion value, using Los Angeles machine	max. 35%	IS: 2386 (Part IV)
or		
Aggregate impact value	max. 30%	-do-
2. Flakiness index	max. 30%	IS: 2386 (Part I)
3. Stripping value	max. 25%	IS: 6241
4. Water absorption (except in case of slags)	max. 2%	IS: 2386 (Part III)
5. Soundness: Loss with sodium sulphate—5 cycles (in case of slag only)	max. 12%	IS: 2386 (Part V)
6. Unit weight or bulk density (in case of slag only)	min. 1120 kg per m³	IS: 2386(Part III)

Uncrushed and rounded river gravel or shingle can also be used but the quantity of binder would be different in their case from that given under para A.2. Where such rounded aggregates are used, it may be necessary to add sufficient quantity of coarse sand and an appropriate quantity of hot bitumen to make the mixture suitable, for which purpose such a mix will have to be designed for binder content depending on individual cases.

A.1.3. **Fine aggregates or sand**: The fine aggregates or sand shall consist of clean, hard, durable, uncoated, coarse dry particles, and be free from injurious amounts of dust, soft or flaky particles or organic matter or other deleterious substances.

A.2. **Quantities of Materials Required**

A.2.1. **Aggregates**

A.2.1.1. **For carpet**

	Per 10 m² of road surface
(a) Coarse aggregates—13.2 mm size; passing IS 22.4 mm square mesh, retained on IS 11.2 mm square mesh	0.18 m³
(b) Coarse aggregates—11.2 mm size; passing IS 13.2 mm square mesh, retained on IS 5.6 mm square mesh	0.09 m³
	0.27 m³

A.2.1.2. **For seal coat**

(a) *Low rainfall areas* (under 150 cm per year)	
Medium coarse sand (fineness modulus of more than 2.5) or fine grit passing IS Sieve No. 1.70 mm and retained on IS Sieve No. 180 microns	0.06 m³
(b) **High rainfall areas* (over 150 cm per year)	
Coarse aggregates—6.7 mm size; passing IS 11.2 mm square mesh, retained on IS Sieve 2.8 mm mesh	0.09 m³

A.2.2. **Binder**

	Per 10 m² of road surface
A.2.2.1. **For tack coat**	
(a) On a water-bound macadam surface	7.3 to 9.8 kg
(b) On an existing black top surface	4.9 to 7.3 kg

Note : For emulsions complying with IS: 3117—1965, the same quantities as given above may be used. In case the existing black top surface is extremely rich in binder, or fatty the tack coat can be eliminated in hot climatic regions at the discretion of the Engineer-in-charge if a good bond between the existing surface and the superimposed layer can be ensured.

A.2.2.2. **For premixing**

(a) For 13.2 mm size coarse aggregates	9.5 kg-@ 52 kg per m³
(b) For 11.2 mm size coarse aggregates	5.1 kg-@ 56 kg per m³

*In case emulsions are used, the quantity required will be 50 per cent of what is indicated.

A.2.2.3. **For seal coat**

(a) *Low rainfall areas* (under 150 cm per year)	6.8 kg
(b) **High rainfall areas* (over 150 cm per year)	9.8 kg

A.3. **Construction**

A.3.1. **Preparation of base** : Before the carpet is applied to the existing base, the road must be free from dust or caked mud. Where the existing base is potholed or rutted, these irregularities must be corrected with premixed chippings or coated, macadam, depending upon the depth of the pothole, laid after applying a tack coat of binder and well rammed. thereafter. Where the existing base is extremely porous and absorptive, a suitable primer (vide I.R.C. Tentative Specification for Priming of Base Course with Bituminous Primers) shall be applied. The surface should be cleaned by :

(a) removing caked earth and other foreign matter with wire brushes,

(b) sweeping with brooms, and

(c) finally dusting with sacks.

A.3.2. **Tack coat** : The binder should be heated, wherever required, to the appropriate temperature as indicated by the manufacturers and applied to the base at the rate specified in para A.2.2. It is best to use a sprayer. The binder should be evenly brushed, if need be.

The tack coat should be applied just ahead of the spreading of the premix.

A.3.3. **Preparation of premix** : Mechanical mixer should be preferred. When only improvised hand mixing drums are available for premixing, place 0.028 m^3 of 13.2 mm chippings and 0.014m^3 of 11.2 mm chippings in the mixing drum and mix dry thoroughly. Where straight-run bitumen is used, the aggregates should be suitably heated prior to the adding of bitumen.

Add 2.24 kg of the binder, as per quantities given in para A.2.2.2, heated where required, to a temperature suitable to the

* In case emulsions are used, quantity required will be 50 per cent of what is indicated.

grade of bitumen used, and mix until the chippings are thoroughly coated with the binder.

Empty the premix on to stretchers or wheel barrows and carry to site.

The quantities of chippings and binder per batch as given may be proportionately increased if proper coating is possible in one operation.

A.3.4. **Spreading of premix** : Immediately after applying the tack coat, spread the premix with rakes to the desired thickness and camber, or distribute evenly by means of a drag spreader. Check camber by means of a camber board and even out inequalities.

A.3.5. **Rolling** : As soon as sufficient length, say 15 metres, of the premix has been laid, rolling should commence with smooth wheeled rollers (6 to 9 tonnes) or pneumatic tyred rollers. Rolling should commence at the edges and progress towards the centre longitudinally except in the case of superelevated sections where this should commence at the inner edge and proceed towards the outer edge of the curve.

When the roller has passed once over the whole area, any high spots or depressions which become apparent should be corrected by removing or adding premixed chippings. When this has been done, roll to comapaction. Avoid excessive rolling as this serves no useful purpose and may spoil the carpet.

Moisten the roller wheels to prevent the premix from adhering to the wheels and being picked up.

A.3.6. **Application of seal coat** : In low rainfall areas, i.e. those having rainfall under 150 cm per year, a premixed sand-seal coat mixed preferably in a mechanical mixer after heating the sand, should be applied immediately after laying the carpet and rolled. Materials required for this seal coat are given in paras A.2.1.2. and A.2.2.3.

In high rainfall areas, i.e., those having rainfall over 150 cm per year, a liquid seal coat preferably with chippings (though coarse sand can be used), should be applied after laying the carpet. The binder, heated wherever required to the permitted temperature, should be applied to the cleaned surface, blinded with chippings and rolled.

Materials required are given in paras A.2.1.2. and A.2.2.3.

A.3.7. **Surface finish :** The finished surface shall be uniform and conform to the lines, grades and typical cross-sections specified. When tested with a 3 metre straight edge, the longitudinal profile of the finished surface shall have no undulation greater than 10 mm, and in any 300 metre length the number of undulations of 6 mm size shall not exceed 30. The cross profile, when checked with a camber template, shall not show a variation of more than 6 mm from the specified profile.

The checking of longitudinal and transverse profile shall be carried out in accordance with the methods and procedures described in IRC Special Publication 11 "Handbook of Quality Control for Construction of Roads and Runways".

A.4. **Opening to Traffic**

Traffic may be allowed on the road preferably 24 hours after providing the seal coat. This should be considered the minimum period when the binder used is a cutback or emulsion.

B. 2 CM THICK TAR CARPET

B.1. **Materials**

B.1.1. **Binder :** The binder shall be a road tar of grade R.T.3 or R T.4 (No. IS: 215—1961).

B.1.2. **Coarse aggregates :** The aggregates shall consist of angular fragments and be clean, hard, tough, durable and of uniform quality throughout. They shall be crushed rock, gravel, river shingle or slag and should be free of elongated or flaky pieces, soft and disintegrated material, and vegetable or other deleterious matter. The aggregates shall also satisfy the following properties :

Property	Value	Method of test
1. Abrasion value, using Los Angeles machine	max. 35%	IS : 2386 (Part IV)
or		
Aggregate impact value	max. 30%	IS : 2386 (Part IV)
2. Falkiness index	max. 30%	IS : 2386 (Part I)
3. Stripping value	max. 25%	IS : 6241
4. Water absorption (except in case of slags)	max. 2%	IS : 2386 (Part III)
5. Soundness : Loss with sodium sulphate —5 cycles (in case of slag only)	max. 12%	IS : 2386 (Part V)
6. Unit weight or bulk density (in case of slag only)	min. 1120 kg per m^3	IS: 2386 [Part III]

B.1.3. **Fine aggregate or sand :** As in para A.1.3.

B.2. **Quantities of Materials Required**

B.2.1. **Aggregates**

B.2.1.1. **For carpet**

	Per 10 m^2 of road surface
(a) Coarse aggregates 13.2 mm size passing IS 22.4 mm square mesh and retained on IS 11.2 mm square mesh	0.18 m^3
(b) Coarse aggregates 11.2 mm size passing IS 13.2 mm square mesh and retained on IS 5.6 mm square mesh	0.09 m^3
	0.27 m^3

B.2.1.2. **For seal coat**

	Per 10 m^2 of road surface
Medium coarse sand or fine grit (passing on IS Sieve No. 1.70 mm and retained on IS Sieve No. 180 microns)	0.060 m^3

B.2.2. **Binder**

(a) For priming coat (on water bound macadam surface)	12.2 to 14.7 kg
(b) For tack coat (on an existing black-top surface)	7.3 to 9.8 kg
(c) For seal coat	9.8 kg
(d) For premixing	19.7 kg @72 kg per m^3

B.3. **Construction**

B.3.1. **Preparation of base :** As in para A.3.1.

B.3.2. **Tack coat :** The binder should be heated to 105° to 115°C and should be applied at the rates specified in para B.2.2.

It is best to use a sprayer. The binder should be evenly brushed, if need be.

The tack coat should be applied just ahead of the spreading of the mix.

B.3.3. Preparation of premix : Mechanical mixers should be preferred. When only improvised hand mixing drums are available for premixing, place 0.028 m^3 of 13.2 mm chippings and 0.014 m^3 of 11.2 mm chippings in the mixing drum and mix thoroughly dry.

Add 3 kg of road tar heated to 105° to 115° C and mix until chippings, preheated if necessary, are thoroughly coated with the binder.

Empty the premix on to stretchers or wheel barrows and carry to site.

The quantities of chippings and binder per batch may be proportionately increased if proper coating is possible in one operation.

B.3.4. **Spreading premix :** As in para A.3.4.

B.3.5. **Rolling :** As in para A.3.5.

B.3.6. **Application of seal coat :** Immediately after laying the carpet, the seal coat should be applied in the manner detailed below:

Road tar IS grade R.T.3, heated to 105° C should be sprayed evenly at 9.8 kg per 10 sq. m. and then it should be blinded evenly with medium coarse dry sand as fine grit at the rate of 0.06 m^3 per 10 m^2.

B.3.7. **Surface finish :** As in para A.3.7.

B.4. Opening to Traffic

Traffic may be allowed on the road 24 hours after providing the seal coat.

'ECIFICATION FOR PRIMING OF BASE COURSE WITH BITUMINOUS PRIMERS

(First Revision)

THE INDIAN ROADS CONGRESS
1989

PECIFICATION FOR PRIMING
OF
BASE COURSE
WITH
BITUMINOUS PRIMERS

(*First Revision*)

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SPECIFICATION FOR PRIMING OF BASE COURSE WITH BITUMINOUS PRIMERS

1. INTRODUCTION

1.1. The revision of IRC : 16-1965 "Tentative Specification for Priming of Base Course with Bituminous Primers" was under consideration with the erstwhile Bituminous Pavement Committee and the task to revise the Specification was entrusted to the Director, Highways Research Station, Madras. The draft prepared by the Director, HRS was discussed by the Flexible Pavement Committee in their meeting held at Madras on 30th September, 1988 where it was decided that bitumen emulsions should also be included in the draft for use as a primer. Shri M. B. Jayawant was requested to modify the draft on the basis of the comments of the members. The Committee in their meeting held at Madras on the 10th December, 1988 (Personnel given below) discussed the revised draft in detail and approved it with minor modifications :

Prof. C. G. Swaminathan ...*Convenor*

P. Bhaskaran ...*Member-Secretary*

R. T. Atre	Rep. of I. I. T. Kharagpur (Dr. B. B. Pandey)
G. R. Ambwani	Rep. of Bharat Petroleum Corporation Ltd. (A. D. Nayak)
Dr. M. P. Dhir	The President, IRC (J. M. Malhotra) —Ex-officio
R. A. Goel	The DG (RD) (K. K. Sarin) —Ex-officio
Dr. A.K. Gupta	The Secretary IRC (Ninan Koshi) —Ex-officio
M. B. Jayawant	Lt. Col. R. Bhargava
P. K. Lauria	R. S Kelkar
Sekhar Mukherjee	J. B. Mathur
Anil T. Patel	L. N. Narendra
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R. K. Saxena	
A. Sankaran	
R. S. Shukla	
N. Sen	
Director, HRS, Madras	
Rep. of I. O. C. (S. S. Das Gupta)	
Rep. of Hindustan Petroleum Corporation Ltd., (R. C. Arora)	

1.2. The revised Standard was considered by the Highways Specifications & Standards Committee in their meeting held at New Delhi on the 7th April, 1989. Minor modifications in text and some rearrangement of text was suggested by members and it was decided that S/Shri R. K. Saxena and P. K. Dutta would finalise the text in consultation with the Convenor. The document finalised by them got the approval of Executive Committee through circulation and was placed before the Council in their 126th meeting held at New Delhi on the 29th April, 1989. The Council approved the draft for publication and authorised the Convenor, Highways Specifications & Standards Committee and the Convenor, Flexible Pavement Committee to finalise the draft on the basis of the comments offered by the members of the Council.

2. SCOPE

2.1. This specification relates to the operation of priming an absorbent base course, preparatory to a subsequent bituminous treatment, through application of a low viscosity bituminous material by spraying.

2.2. The specification is intended to indicate what is considered to be a good practice for priming and shall apply unless modified by special provisions to take into account any unusual conditions.

3. OBJECTIVES

3.1. The objectives of the priming briefly are :

- to penetrate the existing base course surface so as to plug capillary voids in it;
- to coat and bond loose mineral particles on the surface of the base course;
- to seal surface pores and make the surface of the base course water resistant;
- to harden and toughen the base course surface ; and
- to assist adhesion between the base and the superimposed bituminous surface course in conjunction with a tack coat.

3.2. Prime coat is not to be regarded as a substitute for tack coat, the objective of which is to ensure a proper bond between the surface being paved and the new bituminous course being placed over it.

4. SELECTION OF PRIMER

4.1. The choice of a bituminous primer shall depend upon the porosity characteristics of the surface to be primed which may be categorised as below :

4.2. Surfaces of Low Porosity

This group comprises of road base courses with tightly

bonded surfaces when consolidated. Materials like well graded crushed rock, gravels, and granular/stabilised materials which, when properly compacted, present a surface of relatively low porosity.

4.3. Surfaces of Medium Porosity

Pavements constructed with materials so graded as to present a relatively less tight surface are termed as having medium porosity, such as mechanically stabilized soil bases and base courses having materials with silty soil binders.

4.4. Surfaces of High Porosity

This group covers those surfaces which will not consolidate to a tight surface, and which when finished, present a weak and open texture of high porosity.

5. MATERIALS

5.1. The bituminous primer to be used should be such that it can penetrate into the base course to perform its intended function. Selection may be made from cut-back bitumens of low viscosity, cationic bitumen emulsions, or road tars. If the required cut-back is not available commercially, the same may be prepared in the field conforming to the job requirements.

5.2. Types of Primer

Table 1 can be used as guidance for choice of primer on different types of surfaces.

TABLE 1. GUIDELINES FOR CHOICE OF PRIMER

Type of Surface	Emulsion	Cut-back	Road tar
Low porosity	Not suitable	MC-0	RT-1 or RT-2
Medium porosity	SS or MS	MC-1 or MC-2 SC-1 or SC-2	RT-2 or RT-3
High porosity	MS	MC-3 or RC-1	RT-3 or RT-4

The primers shall conform to IS : 8887-1978 (for cationic emulsions), IS : 217-1961 (for cut-backs), and IS : 215-1981 (for road tars), as applicable.

5.3. Viscosity

For selecting the appropriate type of primer out of the materials indicated in Table 1, the atmospheric temperature during application should be given consideration. Also, within the range of viscosity specified, the primer for use may be selected keeping in view the level of porosity of the surface to be treated.

5.4. Quantity of Primer

The primer shall be applied at the rate specified in Table-2.

TABLE-2. QUANTITY OF PRIMER (KG PER 10M²)

Type of surface	Emulsion	Cut-back	Road tar
Low porosity	Not suitable	7.5 to 10.0	7.5--10.0
Medium porosity	14.0—20.0	10.0 to 12.5	10.0--12.5
High porosity	17.0—22.0	12.5 to 15.0	12.5—15.0

6. CONSTRUCTION

6.1. Weather and Seasonal Limitations

6.1.1. Cut-back and road tar primers shall not be applied on wet surface or during dust storm or when the weather is foggy or rainy.

6.1.2. Bitumen emulsion can be applied on wet surface. However, emulsions shall not be applied during dust storm or when it is actually raining.

6.1.3. Atmospheric temperature during priming should be above 10°C.

6.2. Equipment

All equipment required for the execution of work shall be in good working condition at site.

6.3. Preparation of Base Course Surface

6.3.1. The base course surface to be primed shall be swept clean and free from dust. All loose materials and other foreign matter on the surface shall be removed completely, if necessary by using power blowers or sweepers.

6.3.2. Large irregularities, pot-holes, depressions, etc. shall be repaired prior to priming. Minor depressions may be ignored until the surface is primed, after which these might be patched with a suitable premixed material prior to the subsequent bituminous treatment.

6.3.3. The underlying surface shall be dry prior to priming. Except that in the case of bitumen emulsions, it may be desirable to dampen the surface slightly in order to obtain better penetration of the primer. Pre-wetting should be done by water spraying, using equipment capable of uniform application of water over the entire surface. The spraying may be taken up 2 to 12 hours before priming, in such quantity that the surface during priming is damp but not saturated with water.

6.3.4. Traffic shall be kept off the prepared areas prior to priming.

6.4. Application of Primer

6.4.1. After the base to be primed has been prepared as described above, the primer shall be uniformly applied over the surface using mechanical sprayers. Rate of application of primer shall correspond to the quantities given in Table-2.

6.4.2. The spraying should preferably be carried out using sprayer mounted on distributor truck or with hand sprayer using mechanical pump. The use of hand-held containers such as watering cans, perforated buckets etc. is unacceptable and should not be permitted under any circumstances. Quantity should be checked periodically using Tray Coating Test or any other suitable means.

6.4.3. Temperature of application of primer should be high enough to permit the primer to be sprayed effectively through the jets of the spray bar and to cover the base course surface effectively.

6.5. Curing

6.5.1. The primed surface shall be allowed to cure fully. No traffic shall be allowed over the primed surface during this period and in any case not before 24 hours if the primer is a cut-back bitumen and 6 hours in the case of bitumen emulsion.

6.5.2. Any pool of excess cut-back primer, which has not been completely absorbed by any part of the base course surface during the curing period, should be carefully swept over the adjacent surface, and then a light sand blotter course applied. The amount applied should be just sufficient to blot up the excess bitumen and prevent it being picked up under traffic.

If an excess of bitumen residue is found on the primed surface after bitumen emulsion has broken, a very light sand dusting may be applied to soak up the surplus material.

All loose sand should be swept from the base course surface prior to any subsequent bituminous treatment.

IRC : 17-1965

TENTATIVE SPECIFICATION
FOR
SINGLE COAT BITUMINOUS SURFACE DRESSING

(*First Reprint*)

THE INDIAN ROADS CONGRESS
1996

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TENTATIVE SPECIFICATION FOR SINGLE COAT BITUMINOUS SURFACE DRESSING

INTRODUCTION

The Tentative Specification as prepared by the Bituminous Pavements Committee (personnel given below) in their meeting held at New Delhi on the 26th March, 1963 was sent to all members of the Council for their comments. The Tentative Specification as adopted by the Bituminous Pavements Committee in their meeting held at Chandigarh in November, 1963 in light of the comments of the members of the Council was approved for publication by the Executive Committee in their meeting held on the 30th September, 1964, and was first published in August, 1965. The present reprint of the same incorporates certain amendments in the light of later decisions of the Council on related specifications.

Sujan Singh ...	*Convenor*
C.G. Swaminathan	*Member-Secretary*

MEMBERS

N.H. Bhagwanani	Narain Singh
R.F. Davies	D.V. Sahni
M.B. Jayawant	Z.S. Shah
Kewal Krishan	Hukam Singh
Mahabir Prasad	Chief Engineer, H&RW, Tamilnadu
S.K. Rajagopalan	Director General (Road Development) & Addl. Secy. to the Govt. of India *(Ex-officio)*

The Tentative specification is intended to indicate what is considered to be good practice for construction of single coat bituminous surface dressing and shall apply unless modified by special provisions to take into account unusual conditions.

1. DESCRIPTION

The work specified consists of a wearing surface composed of a single application of bituminous material covered with one application of cover material of size as specified below, applied on a previously prepared base or pavement.

2. MATERIALS

2.1. Bituminous Materials

The bituminous materials shall conform to the requirements as specified and provided for in the proposal and satisfy the related specification, issued by the Indian Standards Institution (vide I.S.I Standards 73-1961, 215-1961, 217-1961 and 454-1961). The grades of binders to be used would depend upon the climatic conditons.

2.2. Cover Materials

2.2.1. General requirements : The cover material shall consist of crushed stone, crushed slag, crushed gravel (shingle) or other stones, as specified, and shall have clean, strong, durable, and fairly cubical fragments free from disintegrated pieces, salt, alkali, vegetable matter, dust and adherent coatings. In case crushed gravel is not available at reasonable cost, rounded gravel (shingle) may be used.

The aggregate shall preferably be hydrophobic in nature and of low porosity.

2.2.2. Physical requirements : The aggregate shall satisfy the requirements given in Table 1.

TABLE 1

Property	*Value*	*Method of test*
1. Abrasion value, using Los Angeles machine	Max. 35%	I.S. : 2386 (Part IV)
or		
Aggregate impact value	Max. 30%	--do--
2. Flakiness index	Max. 25%	I.S. : 2386 (Part I)
3. Stripping value	Max. 25%	I.S. : 6241
4. Water absorption (except in case of slag)	Max. 1%	I.S. : 2386 (Part III)
5. Soundness : Loss with sodium sulphate—5 cycles (in case of slag only)	Max. 12%	I.S : 2386 (Part V)
6. Unit weight or bulk density (in case of slag only)	Min. 1120 kg per m³	I.S. : 2386 (Part III)

Where all these conditions cannot be satisfied, it is left to the Engineer-in-charge to allow reasonable tolerances.

2.2.3. Size : The size of chippings to be used shall depend on whether the treatment is for the first coat or for the subsequent or renewal coat and shall be as per the size specified below. For single application of the aggregate, it is desirable to keep the grading of the various sizes as specified in Table 2.

TABLE 2

REQUIREMENTS FOR GRADATION OF AGGREGATES

Sieve designation nominal size of aggregate	*Specification*
I. For surfacing water-bound macadam--first coat	100 per cent passing through 22.4 mm square mesh sieve and retained on 11.2 mm square mesh sieve.
II. For subsequent or renewal coats 10 mm	100 per cent passing through 13.2 mm square mesh sieve and retained on 6.7 mm square mesh sieve

NOTE : It is essential to sieve the aggregates through proper size sieves to ensure the size stipulated in the specifications. The sieve sizes indicated above are as per IS : 460-1962.

3. CONSTRUCTION METHODS

3.1. Weather and Seasonal Limitations

Preferably, the surface dressing work shall be carried on only when the atmospheric temperature in shade is 16°C or above. No bituminous material shall normally be applied when the surface or the cover material is damp, when the weather is foggy or rainy or during duststorm, except, in case of emulsions, the surface should be slightly damp.

3.2. Equipment

All equipment necessary for the proper construction of work shall be on the site of the work in good condition.

3.3. Preparation of Road Surface

The underlying course on which surface dressing is to be laid shall be prepared, shaped and conditioned to a uniform grade and section as specified. Any depressions or pot-holes shall be properly made up and thoroughly compacted sufficiently in advance. The defective parts should be clearly cut out and the patches of new material put in, and not put on the existing surface.

Where the existing surface shows signs of "fatting-up", such position should be rectified.

It is important that the surface be dry and thoroughly cleaned immediately before applying the binder. The surface should be swept clean free of caked earth and other foreign matter cleaned first with hard brushes, then with softer brushes and finally blowing off with sacks or gunny bags to remove the fine dust. If the base to be treated consists of stabilised soil or of porous aggregates, a suitable bituminous primer, vide IRC: 16-1965, should be applied uniformly preferably by a mechanical sprayer.

If the base to be covered by the surface treatment is an old bituminous surfacing, it shall be swept clean and free from sand, dirt, dust and other loose, deleterious, foreign matter, by means of mechanical sweepers and blowers, if available, supplemented by hand brooms where necessary or by means of wire brushes, small picks, brass brooms, etc., and shall be dry.

Whenever a prime coat is applied to the surface, no bituminous material shall be applied until the prime coat has thoroughly cured (vide respective IRC: 16-1965). The edges of the surface to be treated shall be defined by rope lines stretched in position.

3.4. Application of Bituminous Material

After the surface to be treated has been prepared as specified above, bituminous material shall be sprayed uniformly over the dry surface preferably using mechanical sprayers. The binder shall be applied at a temperature appropriate to the type of binder and equipment used. Table 3 gives approximate rate of application of bituminous materials and aggregates per 10 m^2 of surfacing.

Bituminous material shall be applied to the surface uniformly in quantities specified. Excessive deposits of bituminous material upon the road surface caused by stopping or starting the sprayer, by leakage or otherwise, shall be immediately removed.

3.5. Application of Cover Material

Process involving single application

Immediately after the application of bituminous material, the cover material in the quantity mentioned in Table 3 shall be spread uniformly by hand or by means of a mechnical gritter so as to cover the surface completely. It is perferable to use mechanical gritters.

The surface shall be broomed with a view to ensure uniform spreading.

3.6. Rolling Cover Materials

Immediately after the application of the cover materials as described in section 3.5, the entire surface shall be rolled with a 6 to 8 tonne road roller. The rolling shall begin at the edge and proceed lengthwise, over the area to be rolled lapping not less than one third of the roller tread and proceed towards the centre. When the centre is reached, the rolling shall then start at the opposite side and again proceed towards the centre. In the superelevated portions, the rolling should proceed from the inner to the outer edge. While the rolling is in progress, additional aggregate shall be spread by hand in whatever quantities may be required to fill irregularities and to prevent picking up of the aggregate by the roller. Rolling shall be continued until the particles are firmly embedded in the bituminous materials and present a uniform closed surface. Excessive rolling which results in the crushing of the aggregate particles, shall be avoided.

3.7. Finishing

The finished surface shall be uniform and conform to the lines, grades and typical cross sections shown in the specifications. When tested with a template and straight edge, the surface shall show no variation greater than 6 mm over a 3 m length.

3.8. Opening to Traffic

When straight run bitumen or road tar is employed as the binder, the finished surface shall be thrown open to traffic on the following day but if in special circumstances, the road is required to be opened to traffic immediately after rolling, speed of the traffic shall be limited to 16 km per hour till the following day.

Where cutback bitumen and emulsion is employed, the finished surface shall be kept closed to the traffic until it has sufficiently cured to hold the cover aggregates in place.

Controlling of traffic shall be done by some suitable device, such as barricading and posting of watchmen, etc.

TABLE 3

APPROXIMATE RATE OF APPLICATION OF BITUMINOUS MATERIALS AND AGGREGATES

AGGREGATES			BINDER			
No.	*Nominal size in*	*Quantity per 10 m²*	*Straight run bitumen per* 10 m²	*Road tar per* 10 m²	*Cutback per* 10 m²	*Emulsion per* 10 m²
	mm	m²	kg	kg	kg	kg
	I. For surfacing water-bound macadam surfaces					
1.	13.2	0.14 to 0.15	17.1 to 19.5	17.1 to 22	19.5 to 22	—
	II. For subsequent or renewal coats					
1.	11.2	0.09 to 0.11	9.8 to 12.2	12.2 to 17.1	9.8 to 12.2	—
2.	11.2 to 2.6	0.08	—	—	—	9.8 to 12.2

IRC : 18-2000

DESIGN CRITERIA FOR PRESTRESSED CONCRETE ROAD BRIDGES (POST-TENSIONED CONCRETE)

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*ADG(B) being not in position. The meeting was presided by Shri Prafulla Kumar, Director General (Road Development) and Addl. Secretary, Ministry of Surface Transport.

NOTATIONS

A_s	:	Area of High Tensile Steel
A	:	Area of longitudinal reinforcement
A_{sv}	:	Cross-sectional area of two legs of a link
A_t	:	Area of connector steel
A_o	:	Area enclosed by the centre line of members forming a box section
A_1	:	Bearing area of the anchorage converted in shape to a square of equivalent area
A_2	:	Maximum area of the square that can be contained within the member without overlapping the corresponding area of the adjacent anchorages, and concentric with the bearing area 'A_1'.
b	:	Width of a rectangular section or rib of a Tee, L or I beam
b_1	:	Side of anchor plate
B_f	:	Width of flange of Tee or L beam
d	:	Overall depth of the girder measured from top of deck slab to the soffit of girder
d_b	:	Depth of the girder from the maximum compression edge to the centre of gravity of the tendons
d_s	:	Diameter of prestressing wire/strand
d_t	:	Depth from extreme compression fibre either to the longitudinal bars or the centroid of the tendons, whichever is greater
E_c	:	Modulus of Elasticity of concrete at 28 days
E_s	:	Modulus of Elasticity of prestressing steel
E_{cj}	:	Modulus of Elasticity of Concrete at j days (j<28 days)
e	:	Base of Naperian Logarithms
F_{bst}	:	Bursting tensile force in end block
f_{cav}	:	Average compressive stress in flexural compressive zone
f_{cj}	:	Actual concrete cube strength at j days subject to a maximum vlaue of f_{ck} (j<28days)
f_{ck}	:	Characteristic compressive strength of 15 cm cubes at 28 days
f_{cp}	:	Compressive stress at centroidal axis due to prestress taken as positive
f_b	:	Permissible compressive contact stress in concrete including any prevailing stress as in the case of intermediate anchorages
f_p	:	Ultimate tensile strength of prestressing steel
f_{pt}	:	Stress due to prestress only at the tensile fibre distance 'y' from the centroid of the concrete section

f_t : Maximum principal tensile stress in concrete

f_y : The characteristic strength of the untensioned steel

f_{yc} : Yield stress of longitudinal steel in compression

f_{yl} : Yield strength of longitudinal reinforcement or 0.2 per cent proof stress which should be taken as not greater than 415 MPa

f_{yv} : Yield strength of links/shear reinforcement or 0.2 per cent proof stress which should be taken as not greater than 415 MPa

G : Dead load

H_{max} : Larger dimension of the section

H_{min} : Smaller dimension of the section

h_{wo} : Wall thickness of members of box section where torsional stress is determined

I : Second moment of area of the section

k : Wobble co-efficient per metre length of prestressing steel

l : Length of specimen

M : Bending moment at the section

MPa : Mega Pascals

Mt : Cracking moment at the concrete section considered

$M_u$lt : Moment of section under ultimate load condition

n : Number of test strength results

P_k : Load in tendon

Q : Design live load including impact

S : Standard deviation (Sample)

SG : Superimposed dead load

S_t : Spacing of connectors

S_v : Link spacing along the length of a member

T : Torsional moment due to ultimate loads

t : Thickness of flange of a Tee beam

V : Shear force at the section considered under ultimate loads, actual volume of water

V_a : Premeasured quantity of water

V_b : Balance quantity of water left

V_c : Ultimate shear resistance of a concrete section

V_{co} : Ultimate shear resistance of a concrete section uncracked in flexure

V_p : Volume of sheathing sample used in water loss study

V_{cr} : Ultimate shear resistance of a concrete section, cracked in flexure

V_{tc} : Torsional shear stress in concrete section upto which no torsional reinforcement is required

V_t : Torsional shear stress at a section

V_{tu} : Ultimate torsional shear stress at a section

x : Distance in metre between points of operation of σpo and σpo (x)

X : Smaller dimension of links measured between centres of legs

y : Tensile fibre distance from the centroid of the concrete section

Y_{po} : $\dfrac{\text{Side of loaded area of end block}}{2}$

Y_o : $\dfrac{\text{Side of end block}}{2}$

Y_1 : Larger dimension of links measured between centres of legs

Δ : Deviation of individual test strength from the average test strength of 'n' test strength results

μ : Co-efficient of friction between cable and duct

θ : Cumulative angle in radian through which the tangent to the cable profile has turned between the points of operation of σpo and $\sigma po(x)$

σ : Standard deviation (population)

σpo : Steel stress at the jacking end

$\sigma po(x)$: Steel stress at a point, distant 'x' from the jacking end

ϕ : Internal nominal diameter of sheathing

1. INTRODUCTION

The Design Criteria for Prestressed Concrete Road Bridges (Post-Tensioned Concrete) was first published in December, 1965. To cater for the technological developments which were taking place in course of time, the Criteria were examined by the Technical Committees of the IRC and revised in 1977 and 1985 in the light of their recommendations.

In the light of further developments in the field of prestressed concrete, the task of reviewing the criteria and carrying out required revisions was entrusted to the Committee for Reinforced, Prestressed and Composite Concrete (B-6) consisting of the following personnel :

Ninan Koshi .. *Convenor*
A.N. Dhodapkar .. *Member-Secretary*

MEMBERS

N.K. Sinha	Jose Kurien
S.G. Joglekar	P.Y. Manjure
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(K.B. Rajoria)	(Prafulla Kumar)

Secretary, IRC
(S.C. Sharma)

CORRESPONDING MEMBER

P.S. Tyagi

The amendments as finalised by this Committee were considered and approved by the Bridge Specifications and Standards Committee in their meeting held at New Delhi on 7th December, 1999 and later approved by the Executive Committee in their meeting held at New Delhi on 14th December, 1999. The draft amendments were discussed and approved by the Council of the Indian Roads Congress at the 157th Council Meeting held at Madurai on 4th January, 2000.

It was also decided that the document would be published as a fully revised Criteria after incorporating all the amendments.

The object of issuing this publication is to establish a common procedure for the design and construction of Prestressed Concrete Road Bridges in India. The publication is meant to serve as a guide to both the design engineers and the construction engineers but compliance with the provisions therein does not relieve them in any way of the responsibility for the stability, soundness and safety of the structures designed and erected by them.

The design and construction of road bridges require an extensive and thorough knowledge of science and technique involved and should be entrusted only to specially qualified engineers with adequate experience of bridge engineering, capable of ensuring careful execution of work.

2. SCOPE

These Criteria cover the design aspects for prestressed concrete (post-tensioned) road bridges (determinate structures only). These are not applicable to the design of members which are subjected to direct compression like piers.

3. MATERIALS

3.1. Cement

Any of the following shall be used with prior approval of the competent authority :

(a) Ordinary Portland Cement conforming to IS: 269

(b) High Strength Portland Cement conforming to IS: 8112

(c) Ordinary Portland Cement conforming to IS:12269 (Grade 53)

(d) Sulphate Resistant Portland Cement conforming to IS:12330.

3.2. Aggregates

All coarse and fine aggregates shall conform to IS:383 and tests for conformity shall be carried out as per IS:2386 Parts I to VIII.

3.2.1. Coarse aggregate

3.2.1.1. Coarse aggregate shall consist of clean, hard; strong, dense non-porous and durable pieces of crushed stone, crushed gravel, natural gravel or a suitable combination thereof or other approved inert material. It shall not contain pieces of disintegrated stones, soft, flaky elongated particles, salt, alkali, vegetable matter or other deleterious materials in such quantities as to reduce the strength or durability of the concrete, or to attack the embedded steel.

The nominal maximum size of aggregates shall usually be restricted to 10 mm less than the minimum clear distance between individual cables or individual untensioned steel reinforcement or 10 mm less than the minimum cover to untensioned steel reinforcement, whichever is smaller. A nominal size of 20 mm coarse aggregates shall generally be considered satisfactory for prestressed concrete work.

3.2.2. **Fine aggregates** : Fine aggregates shall conform to Clause 302.3.3 of IRC:21.

3.3. **Water**

Water used for mixing and curing shall be in conformity with Clause 302.4 of IRC:21.

3.4. Admixtures may be used in conformity with Clause 302.2 of IRC:21.

3.5. **Steel**

3.5.1. The prestressing steel shall be any of the following:

(a) Plain hard drawn steel wire conforming to IS: 1785 (Part I) and IS: 1785 (Part II)

(b) Cold drawn indented wire conforming to IS:6003

(c) High tensile steel bar conforming to IS:2090

(d) Uncoated stress relieved strand conforming to IS:6006, and

(e) Uncoated stress relieved low relaxation steel conforming to IS:14268.

Data in respect of modulus of elasticity, relaxation loss at 1000 hrs., minimum ultimate tensile strength, stress-strain curve etc. shall necessarily be obtained from manufacturers. Prestressing steel shall be subjected to acceptance tests prior to actual use on the works (guidance may be taken from BS:4447). The modulus of elasticity value, as per acceptance tests, shall conform to the design value which shall be within a range not more than 5 per cent between the maximum and minimum.

3.5.2. **Untensioned steel** : Reinforcement used as untensioned steel shall be any of the following :

(a) Mild steel and medium tensile steel bars conforming to IS:432 (Part I)

(b) High strength deformed steel bars conforming to IS:1786 and

(c) Hard drawn steel wire fabric conforming to IS:1566.

The reinforcement bars bent and fixed in position shall be free from rust or scales, chloride contamination and other corrosion products. Where cleaning of corroded portions is required, effective method of cleaning such as sand blasting shall be adopted.

3.5.3. Prestressing accessories like jacks, anchorage, wedges, block plates, etc. being patented items shall be obtained from authorised manufacturers only. The prestressing components and accessories shall be subjected to an acceptance test prior to their actual use on the works (guidance may be taken from BS:4447).

3.6. **Sheathing Ducts**

The sheathing ducts shall be either in mild steel as per the sub-clause 3.6.1 or in HDPE as per sub-clause 3.6.2. They shall be in as long lengths as practical from handling and transportation considerations without getting damaged. They shall conform to the requirements specified in ***Appendix-1A/1B*** and a test certificate shall be furnished by the manufacturer. The tests specified in ***Appendix-1B*** are to be performed as part of additional acceptance tests for prestressing systems employing corrugated HDPE sheathing ducts and are not meant for routine site testing purposes.

3.6.1. **MS sheathing ducts**

3.6.1.1. Unless otherwise specified, the material shall be Cold Rolled Cold Annealed (CRCA) Mild Steel intended for mechanical treatment and surface refining but not for quench hardening or tempering.

3.6.1.2. The material shall be clean and free from rust and normally of bright metal finish. However, in case of use in aggressive environment galvanised or lead coated mild steel strips shall be adopted.

3.6.1.3. The thickness of metal sheathing sh[illegible] not be less than 0.3 mm, 0.4 mm and 0.5 mm for sheat[illegible] ducts having internal diameter upto 50 mm, 75 mm and 90 mm respectively. For bigger diameter of ducts, thickness of sheathing shall be based on recommendations of prestressing system supplier.

3.6.1.4. The sheathing shall conform to the requirements specified in ***Appendix-1A*** and a test certificate shall be furnished by the manufacturer.

3.6.1.5. The joints of all sheathing shall be water tight and conform to the provisions contained in *Appendix-2*.

3.6.2. **Corrugated HDPE sheathing ducts**

3.6.2.1. Unless otherwise specified, the material for the ducts shall be high-density polyethylene with more than 2 per cent carbon black to provide resistance to ultraviolet degradation and shall have the following properties :

Specific Density	:	0.954 g/cm^3 at 23°C
Yield Stress	:	18.0 N/mm^2
Tensile Strength	:	21.0 N/mm^2
Shore Hardness D-3 sec. value	:	60
-15 sec. value	:	58
Notch impact strength at 23°C	:	10 kJ/m^2
-40°C	:	4 kJ/m^2
Coefficient of Thermal Expansion for 20°C - 80°C	:	1.50 x 10^{-4} kJ/m^2

3.6.2.2. The thickness of the wall shall be 2.3 ± 0.3 mm as manufactured and 1.5 mm after loss in the compression test, for duct size upto 160 mm OD.

3.6.2.3. The ducts shall be corrugated on both sides. The ducts shall transmit full tendon strength from the tendon to the surrounding concrete over a length not greater than 40 duct diameters.

3.6.2.4. These ducts shall be joined by adopting any one or more of the following methods, as convenient to suit the individual requirements of the location, subject to the satisfactory pressure tests, before adoption.

* Screwed together with male and female threads.
* Joining with thick walled HDPE shrink couplers with glue. This can also be used for connection with trumpet, etc.
* Welding with electrofusion couplers.

The joints shall be able to withstand an internal pressure of 0.5 bar for 5 minutes as per test procedure given in ***Appendix-1A***.

4. CONCRETE

Concrete shall be in accordance with Clause **302.6** of IRC:21.

5. LOADS AND FORCES

5.1. The loads and forces and load combinations as per IRC:6-1966 and as applicable for the given structure shall be duly accounted for.

5.2. All critical loading stages shall be investigated. The stages stated below may normally be investigated:

(i) Stage prestressing;

(ii) Construction stages including temporary loading, transport, handling and erection or any occasional loads that may occur during launching of girders, etc. including impact, if any;

(iii) The design loads as per load combination of 5.1 above including the following discrete stages:

(a) Service Dead Load+Prestress with full losses.

(b) Service Dead Load+Live Load+Prestress with full losses.

(iv) For the combination of loads with differential temperature gradient effects, maximum 50 per cent live load shall be considered and any tensile stresses shall be taken care of by providing adequately designed untensioned steel subject to the crack width limitations stipulated in IRC:21. This shall apply notwithstanding the provision contained in Clause 7.2.2. However, in the case of precast segmental construction no tension shall be permitted under this load combination.

(v) Ultimate load, as per Clause 12.

6. STAGE PRESTRESSING

Stage prestressing is permissible. The number of stages of prestressing and grouting shall be reduced to the minimum, preferably not more than 2. However, concrete shall have attained a strength of not less than 20 *MPa* before any prestressing is applied.

7. PERMISSIBLE STRESSES IN CONCRETE

7.1. Permissible Temporary Stresses in Concrete

7.1.1. These stresses are calculated after accounting for all losses except those due to relaxation of steel, residual shrinkage and creep of concrete.

7.1.2. The compressive stress produced due to loading mentioned in Clause 5.2 (ii) shall not exceed 0.5 f_{cj} which shall not be more than 20 *MPa*, where f_{cj} is the concrete strength at that time subject to a maximum value of f_{ck}.

7.1.3. At full transfer the cube strength of concrete shall not be less than 0.8 f_{ck}. Temporary compressive stress in the extreme fibre of concrete (including stage prestressing) shall not exceed 0.50 f_{cj} subject to a maximum of 20 *MPa*.

7.1.4. The temporary tensile stresses in the extreme fibres of concrete shall not exceed 1/10th of the permissible temporary compressive stress in the concrete.

7.2. Permissible Stress in Concrete during Service

7.2.1. The compressive stress in concrete under service loads shall not exceed 0.33 f_{ck}.

7.2.2. No tensile stress shall be permitted in the concrete during service.

7.2.3. If pre-cast segmental elements are joined by prestressing, the stresses in the extreme fibres of concrete during service shall always be compressive and the minimum compressive stress in an extreme fibre shall not be less than five per cent of maximum permanent compressive stress that may be developed in the same section. This provision shall not, however, apply to cross prestressed deck slabs.

7.2.4. The structure shall also be checked for 20 per cent higher time dependent losses like creep, shrinkage, relaxation, etc. Under this condition, no tensile stress shall be permitted.

7.3. Permissible Bearing Stress Behind Anchorages

The maximum allowable stress, immediately behind the anchorages in adequately reinforced end blocks may be calculated ıc equation :

$$f_b = 0.48\, f_{cj} \sqrt{\frac{A_2}{A_1}} \text{ or } 0.8\, f_{cj} \text{ whichever is smaller}$$

Where f_b = the permissible compressive contact stress in concrete including any prevailing stress as in the case of intermediate anchorages

A_1 = the bearing area of the anchorage converted in shape to a square of equivalent area

A_2 = the maximum area of the square that can be contained within the member without overlapping the corresponding area of adjacent anchorages, and concentric with the bearing area A_1.

Notes : (i) The above value of bearing stress is permissible only if there is a projection of concrete of at least 50 mm or $b_1/4$, whichever is more all round the anchorage, where b_1 is as shown in **Fig. 1.**

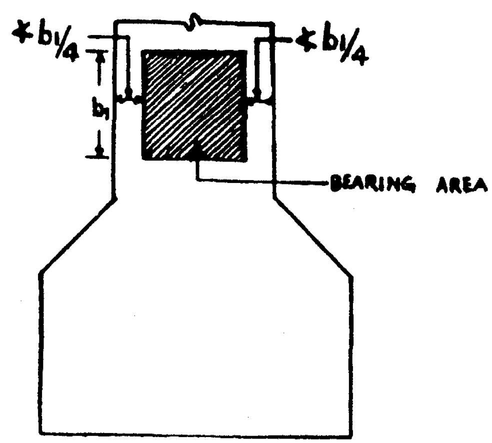


Fig. 1.

(ii) Where embedded anchorages are provided, the reinforcement details, concrete strength, cover and other dimensions shall conform to the manufacturer's specifications/specialist literature.

(iii) The pressure operating on the anchorage shall be taken before allowing for losses due to creep and shrinkage of concrete and relaxation of steel, but after allowing for losses due to elastic shortening, relaxation of steel and seating of anchorage.

8. PERMISSIBLE STRESSES IN PRESTRESSING STEEL

Maximum jack pressure shall not exceed 90 per cent of 0.1% proof stress. For the purpose of this Clause 0.1% proof stress shall be taken as equal to 85% of minimum Ultimate Tensile Strength (UTS).

9. SECTION PROPERTIES

9.1. For members consisting of precast as well as cast-in-situ units, due consideration shall be given to the different moduli of elasticity of concrete in the precast and in-situ portions.

9.2. Openings in Concrete Section

For the purpose of determining the flexural stresses both prior to and after grouting of the cables or tendons, the properties of the section such as area, position of centroid and moment of inertia may be based upon the full section of the concrete without deducting for the area of longitudinal openings left in the concrete for prestressing tendons, cable ducts or sheaths. No allowance for the transformed area of the prestressing tendons shall, however, be made.

Deduction shall be made for the holes of transverse prestressing tendons at sections where they occur, for determining the stresses before grouting of these holes.

inner corners of the box section. For all other corners fillets of suitable size may be provided.

9.3.2.7. The minimum clear height inside the box girders shall be 1.5 m to facilitate inspection.

9.4. Diaphragms/Cross Girders

Diaphragms shall be provided depending upon design requirements. The thickness of diaphragms shall not be less than the minimum web thickness.

10. MODULI OF ELASTICITY

10.1. Modulus of Elasticity of Steel (E_s)

10.1.1. For the purpose of design the following nominal values of modulus of elasticity can be assumed except where the manufacturers certified values or test results are available:

Table 1

Type of steel	Modulus of elasticity *MPa*
Plain hard-drawn wires (conforming to IS:1785 and IS:6003)	2.1×10^5
High tensile steel bars rolled or heat treated (conforming to IS:2090)	2.0×10^5
Strands (conforming to IS:6006)	1.95×10^5

10.1.2. Representative values of modulus of elasticity as supplied by the manufacturers or as per test results based on one test of 3 samples for every lot of 10 tonnes or part thereof shall be used for verification of the elongation calculations.

10.2. Modulus of Elasticity of Concrete (E_c)

Unless otherwise determined by tests, the modulus of elasticity, E_c of concrete shall be assumed to have a value

$$E_c = 5700\sqrt{f_{ck}} \quad MPa$$

The value of the modulus of elastcity E_{cj} of the concrete at j days may be taken to be.

$$E_{cj} = 5700\sqrt{f_{cj}} \quad MPa$$

11. LOSSES IN PRESTRESS

Decrease in prestress in steel due to elastic shortening, creep and shrinkage of concrete, relaxation of steel, friction and seating of anchorages shall be calculated on the following basis:

11.1. Elastic Shortening

The loss due to elastic shortening of concrete shall be computed based on the sequence of tensioning. However, for design purposes, the resultant loss of prestress in tendons tensioned one by one may be calculated on the basis of half the product of modular ratio and the stress in concrete adjacent to the tendons averaged along the length. Alternatively the loss of prestress may be computed exactly based on sequence of stressing.

11.2. Creep of Concrete

The strain due to creep of concrete shall be taken as specified in Table 2.

Table 2

Maturity of concrete at the time of stressing as a percentage of f_{ck}	Creep strain per 10 *MPa*
40	9.4×10^{-4}
50	8.3×10^{-4}
60	7.2×10^{-4}
70	6.1×10^{-4}
75	5.6×10^{-4}
80	5.1×10^{-4}
90	4.4×10^{-4}
100	4.0×10^{-4}
110	3.6×10^{-4}

Notes: (i) The creep strain during any interval may be taken as the strain due to a sustained stress equal to the arithmetic mean of the initial and the final stress occurring during that interval.

(ii) The stress for the calculation of the loss due to creep shall be taken as the stress in concrete at the centroid of the prestressing steel. Variation in stress, if any, along the centroid of the prestressing steel, may be accounted for.

(iii) Values of creep strain for intermediate figures for the maturity of concrete at the time of stressing may be interpolated taking a linear variation between the values given above.

(iv) The above values are for Ordinary Portland Cement.

11.3. Shrinkage of Concrete

The loss in prestress in steel, due to shrinkage of concrete shall be estimated from the values of strain due to residual shrinkage given in Table 3.

11.4. Relaxation of Steel

Relaxation of steel shall invariably be verified by testing to ascertain conformance to the respective codes for prestressing

Table 3

Age of concrete at the time of stressing, in days	Strain due to residual shrinkage
3	4.3×10^{-4}
7	3.5×10^{-4}
10	3.0×10^{-4}
14	2.5×10^{-4}
21	2.0×10^{-4}
28	1.9×10^{-4}
90	1.5×10^{-4}

Notes: (i) Values for intermediate figures for any age of concrete may be interpolated taking a linear variation between the values given.

(ii) The above are for Ordinary Portland Cement.

steel as per Clause 3.5.1. For calculation of permissible temporary stress in concrete as per Clause 7.1 losses due to relaxation of steel shall be taken on the basis of 1000 hour value. For calculation of stress in service condition, long term relaxation loss values occurring at about 0.5×10^6 hours shall be considered, which shall be taken as 3 times the 1000 hour value given in Table 4A.

Table 4A

Relaxation loss at 1000 hours at 20°C ± 2°C (as % of initial stress)

Initial stress	Relaxation loss for Normal relaxation steel (%)	Relaxation loss for low relaxation steel (%)
0.5 fp	0	0
0.6 fp	2.5	1.25
0.7 fp	5.0	2.5
0.8 fp	9.0	4.5

Notes: (i) For intermediate values linear interpolation may be done

(ii) fp = Minimum Ultimate Tensile Stress (UTS) of steel

Table 4B

Relaxation between **relaxation** loss and time upto 1000 hours

Time in Hours	1	5	20	100	200	500	1000
Relaxation loss as % of loss at 1000 hrs	15	25	35	55	65	85	100

11.5. Losses Due to Seating of Anchorages

Depending upon the type of post tensioning, losses in prestress occur due to slip of wires, draw-in of male cones, strains in anchorage, the value of which shall be as per tests or manufacturer's recommendations and duly accounted, for considering reverse friction near the anchorage ends. For this purpose the values of co-efficient of friction and wobble co-efficient shall be taken same as those stipulated for positive friction.

11.6. Friction Losses

Steel stress in prestressing tendons $\sigma_{po}(x)$ at any distance x from the jacking end can be calculated from the formula

$$\sigma_{po} = \sigma_{po}(x)\, e^{(\mu\theta + kx)}$$

Where
- σ_{po} = the steel stress at the jacking end
- e = the base of Naperian Logarithms
- μ = the co-efficient of friction
- θ = the cumulative angle in radians through which the tangent to the cable profile has turned between the points of operation of σ_{po} and $\sigma_{po}(x)$.
- $\sigma_{po}(x)$ = the steel stress at a point, distant 'x' from the jacking end
- k = the wobble co-efficient per metre length of steel
- x = the distance between points of operation of σ_{po} and $\sigma_{po}(x)$ in metres.

The value of μ and k given in Table 5 may be adopted for calculating the friction losses.

Table 5

Type of high tensile steel	Type of duct or sheath	Values recommended to be used in design	
		k per metre	μ
Wire cables	Bright metal	0.0091	0.25
	Galvanized	0.0046	0.2
	Lead coated	0.0046	0.18
	Unlined duct in concrete	0.0046	0.45
Uncoated stress relieved strands	Bright metal	0.0046	0.25
	Galvanized	0.0030	0.20
	Lead coated	0.0030	0.18
	Unlined duct in concrete	0.0046	0.50
	Corrugated HDPE	0.0020	0.17

Notes:
(i) Values to be used in design may be altered to the values observed, on satisfactory evidence in support of such values.
(ii) For multi-layer wire cables with spacer plates providing lateral separation, the value of μ may be adopted on the basis of actual test results.
(iii) When the direction of friction is reversed, the index of 'e' in the above formula shall be negative.
(iv) The above formula is of general application and can be used for estimation of friction between any two points along the tendon distant 'x' from each other.

The values of μ and k used in design shall be indicated on the drawings for guidance in selection of the material and the methods that will produce results approaching the assumed values.

11.7. For structures constructed by segmental construction or other complex construction methods which require accurate determination and detailed information of time dependent effects, specialist literature may be referred to.

12. ULTIMATE STRENGTH

A prestressed concrete structure and its constituent members shall be checked for failure conditions at an ultimate load of 1.25 G + 2 SG + 2.5 Q under moderate condition and 1.5 G + 2 SG + 2.5 Q under severe exposure conditions where G, SG and Q denote permanent load, superimposed dead load and live load including impact respectively. The superimposed dead load shall include dead load of precast footpath, hand rails, wearing course, utility services, kerbs etc. For sections, where the dead load causes effects opposite to those of live load, the sections shall also be checked for adequacy for a load of G + SG + 2.5 Q.

13. CALCULATION OF ULTIMATE STRENGTH

Under ultimate load conditions, the failure may either occur by yielding of the steel (under-reinforced section) or by direct crushing of the concrete (over-reinforced section). Ultimate moment of resistance of sections, under these two alternative conditions of failure shall be calculated by the following formulae and the smaller of the two values shall be taken as the ultimate moment of resistance for design :

(i) Failure by yield of steel (under-reinforced section)

$$M_{ult} = 0.9\, d_b\, A_s\, f_p$$

Where A_s = the area of high tensile steel

f_p = the ultimate tensile strength for steel without definite yield point or yield stress or stress at 4 per cent elongation whichever is higher for steel with a definite yield point.

d_b = the depth of the beam from the maximum compression edge to the centre of gravity of the steel tendons.

Non-prestressed reinforcement may be considered as contributing to the available tension for calculation of the ultimate moment of resistance in an amount equal to its area times its yield stress, provided such reinforcement is welded or has sufficient bond under conditions of ultimate load.

(ii) Failure by crushing of concrete

$$M_{ult} = 0.176\, bd_b^2\, f_{ck}$$

for a rectangular section

$$M_{ult} = 0.176\, bd_b^2\, f_{ck} + \frac{2}{3}\, 0.8\, (B_f - b) \left(d_b - \frac{t}{2}\right) x\, t\, f_{ck}$$

for a Tee beam

Where b = the width of rectangular section or web of a Tee beam

B_f = the width of flange of Tee beam

t = the thickness of flange of a Tee beam

14. SHEAR AND TORSION

14.1. Shear

14.1.1. The calculations for shear are only required for the Ultimate Load.

At any section the ultimate shear resistance of the concrete alone, V_c shall be considered for the section both uncracked (see Clause 14.1.2) and cracked (see Clause 14.1.3) in flexure irrespective of the magnitude of M_t, and the lesser value taken and, if necessary, shear reinforcement (see Clause 14.1.4) provided.

For a cracked section, the conditions of maximum shear with co-existent bending moment and maximum bending moment with co-existent shear shall both be considered.

The effect of the vertical component of the bottom flange force in members of variable depth may also be considered where applicable. While calculating this component the design moment to be considered shall be concomitant with the design shear force being considered.

14.1.2. **Sections uncracked in flexure**

14.1.2.1. The ultimate shear resistance of a section uncracked in flexure, V_{co}, corresponds to the occurrence of a maximum principal tensile stress, at the centroidal axis of the section, of $f_t = 0.24\, f_{ck}$.

In the calculation of V_{co}, the value of prestress at the centroidal axis has been taken as 0.8 f_{cp}. The value of V_{co} is given by :

$$V_{co} = 0.67\, bd \sqrt{f_t^2 + 0.8 f_{cp} f_t}$$

Where $*b$ = width in the case of rectangular member and width of the rib in the case of T, I and L beams

d = overall depth of the member

f_t = maximum principal tensile stress given by $0.24\sqrt{f_{ck}}$

f_{cp} = compressive stress at centroidal axis due to prestress taken as positive.

*Where the position of a duct coincides with the position of maximum principal tensile stress, e.g., at or near the junction of flange and web near a support, the value of b should be reduced by the full diameter of the duct if ungrouted and by two-thirds of the diameter if grouted.

In flanged members where the centroidal axis occurs in the flange, the principal tensile stress should be limited to $0.24\sqrt{f_{ck}}$ at the intersection of the flange and web; in this calculation, 0.8 of the stress due to prestress at this intersection may be used in calculating V_{co}.

For a section uncracked in flexure and with inclined

tendons or vertical prestress, the component of prestressing force normal to the longitudinal axis of the member may be added to V_{co}.

14.1.2.2. For bridge decks with precast prestressed beams and cast-in-situ deck slab, V_{co} can be evaluated as given below:

M_{pc} = ultimate design moment acting on the precast section alone

V_{c1} = ultimate design shear force acting on the precast section alone

Ip = second moment of area of precast section alone

I_c = second moment of area of composite section

$(Ay)_p$ = first moment of area above composite centroid for precast section alone

$(Ay)_c$ = first moment of area above composite centroid for composite section

f_{cm} = stress at composite centroid due to M_{pc}

f_{cp} = stress at composite centroid due to prestress

f_{cp} = $0.8\, f_{cp} + f_{cm}$.

f_s = stress at composite centroid due to V_{c1}

= $\dfrac{V_{c1}\,(Ay)_p}{(Ipb)}$, if $f_s \geq f_t$ then section revision is required

V_{c2} = additional ultimate shear force which can be carried by the composite section before the principal tensile stress at composite centroid reaches $f_t = 0.24\, (fck)^{1/2}$

= $\dfrac{(I_c b)}{(Ay)c}\,[(f_t^2 + f_{cp} f_t)^{1/2} - f_s]$

Vco = $V_{c1} + V_{c2}$

14.1.3. **Sections cracked in flexure** : The ultimate shear resistance of a section cracked in flexure, Vcr may be calculated using the equation given below :

$$\mathrm{Vcr} = 0.037\ \mathrm{bd}_b \sqrt{f_{ck}} + \frac{M_t}{M}\, V$$

Where d_b = is the distance from the extreme compression fibre to the centroid of the tendons at the section considered;

M_t = is the cracking moment at the section considered, $Mt = (0.37\sqrt{f_{ck}} + 0.8 f_{pt})I/y$ in which f_{pt} is the stress due to prestress only at the tensile fibre distance y from the centroid of the concrete section which has a second moment of area I;

V and M = are the shear force and corresponding bending moment at the section considered due to ultimate loads;

V_{cr} = should be taken as not less than 0.1 bd $\sqrt{f_{ck}}$. The value of V_{cr} calculated at a particular section may be assumed to be constant for a distance equal to $d_b/2$, measured in the direction of increasing moment from that particular section.

For a section cracked in flexure and with inclined tendons, the component of prestressing forces normal to the longitudinal axis of the member should be ignored.

14.1.4. **Shear reinforcement** : When V, the shear force due to the ultimate load is less than $V_c/2$ then no shear reinforcement need be provided. A minimum shear reinforcement shall be provided when V is greater than $V_c/2$ in the form of links such that

$$\frac{A_{sv}}{S_v} \times \frac{0.87 F_{yv}}{b} = 0.4\,MPa \qquad \text{..... (1)}$$

When the shear force V, due to the ultimate loads exceeds V_c, the shear reinforcement provided shall be such that

$$\frac{A_{sv}}{S_v} = \frac{V - V_c}{0.87 f y_v d_t} \qquad \text{.....(2)}$$

Where V_c = is the shear force that can be carried by the concrete

f_{yv} = is the yield strength of the links/shear reinforcement or 0.2 per cent proof stress which should be taken as not greater than 415 MPa

A_{sv} = is the corss-sectional area of the two legs of a link

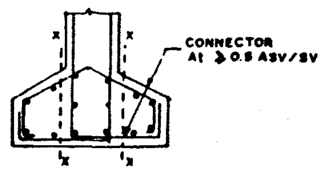
S_v = is the link spacing along the length of member

d_t = is the depth from the extreme compression fibre either to the longitudinal bars having diameter not less than the link bar over which the link will pass or to the centroid of the tendons, whichever is greater.

In beams, at both corners in the tensile zone, a link shall pass round a longitudinal bar, a tendon or a group of tendons having a diameter not less than the link diameter. A link shall extend as close to the tension and compression faces as possible, with due regard to cover. The links provided at a cross section shall between them enclose all the tendons and additional reinforcement provided at the cross section and shall be adequately anchored, Fig. 2 (i-iii). In no case shear reinforcement provided shall be less than that required as per equation (1) above when V is greater than $V_c/2$.

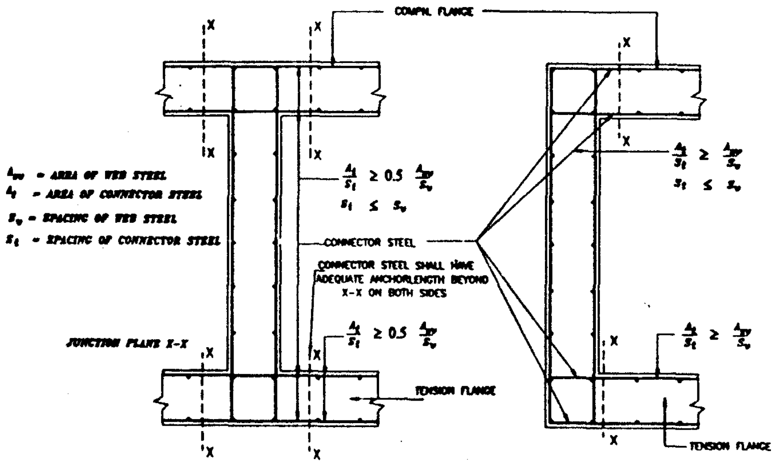
14.1.5. **Maximum shear force** : In no circumstances shall the shear force 'V', due to ultimate loads, exceed the appropriate value given in Table 6 multiplied by bd_b, where 'b' is as defined in sub-clause 14.1.2, less either the diameter of the duct for ungrouted or two-thirds the diameter of the duct for grouted ducts and 'd_b' is the distance from the compression face to the centroid of the actual steel area in tensile zone but not less than 0.80 times the overall depth of the member.

The shear force V should include an allowance for prestressing only for sections uncracked in flexure (see Clause 14.1.2).



Connector details tension flange (bulb) of T-beam

(i)



Connector details at junction of intermediate web and flanges of box girder

(ii)

Connector details at junction of end web and flanges of box girder

(iii)

Fig. 2. Connector reinforcement for compression flange and tension flange

Table 6. Maximum Shear Stress

	Concrete Grade						
	30	35	40	45	50	55	60
	MPa	MPa	MPa	MPa	MPa	MPa	MPa
Maximum Shear Stress	4.1	4.4	4.7	5.0	5.3	5.5	5.8

Note : For intermediate values linear interpolation may be done.

14.2. Torsional Resistance of Beams

14.2.1. **General** : Torsion does not usually decide the dimensions of members, therefore, torsional design should be carried out as a check after the flexural design. In general, where the torsional resistance or stiffness of members has not been taken into account in the analysis of the structure no specific calculations for torsion will be necessary, adequate control of any torsional cracking being provided by the required nominal shear reinforcement. Therefore, provisions made in the clause are to be followed when the effect of torsion is appreciable. Alternative methods of designing members subjected to combined bending, shear and torsion could also be used provided the rationality of the method adopted is justified.

14.2.2. **Stresses and reinforcement** : Calculations for torsion are required only for ultimate loads and the torsional shear stresses should be calculated assuming a plastic stress distribution. Where the torsional shear stress V_t exceeds the value V_{tc} from Table 7, reinforcement shall be provided. In no case, shall the sum of shear stresses resulting from shear force and torsion $(V + V_t)$ exceed the value of V_{tu} from Table 7 nor in the case of small sections $(Y_l < 550$ mm) should the torsional shear stress, V_t, exceed $V_{tu} \times Y_{1/550}$ where Y_l is the larger dimension of a link.

Torsion reinforcement shall consist of rectangular effectively closed links together with longitudinal reinforcement. This reinforcement is addition to that required for shear or bending.

Table 7. Ultimate Torsion Shear Stress

	Concrete Grade						
	30	35	40	45	50	55	60
	MPa	MPa	MPa	MPa	MPa	MPa	MPa
V_{tc}	0.37	0.40	0.42	0.42	0.42	0.42	0.42
V_{tu}	4.10	4.45	4.75	5.03	5.30	5.56	5.81

14.2.3. Computation of torsional stresses for various cross sections

14.2.3.1. Rectangular section

$$V_t = \frac{2T}{\left(h^2_{min}\right) \times \left(h_{max} - \frac{h_{min}}{3}\right)}$$

Where T = is the torsional moment due to ultimate loads
h_{min} = is the smaller dimension of the section
h_{max} = is the larger dimension of the section

Torsional reinforcement should be provided such that

$$\frac{A_{sv}}{S_v} \geq \frac{T}{0.8 X_1 Y_1 (0.87 f_{yv})}$$

$$A_{SL} \geq \frac{A_{sv}}{S_v}(X_1 + Y_1)\left(\frac{f_{yv}}{f_{yL}}\right)$$

Where A_{sv} = is the total area of legs of closed links at a section
A_{SL} = is the area of longitudinal reinforcement

f_{yL} = is the yield strength of longitudinal reinforcement which should not be taken greater than 415 *MPa*
f_{yv} = is the yield strength of links
S_v = is the spacing of the links
X_l = is the smaller dimension of the link measured between centres of legs
Y_l = is the larger dimension of the link measured between centres of legs.

To prevent a detailing failure the closed links shall be detailed to have minimum cover and a pitch less than the smallest of $(X_l + Y_l)/4$, 16 times longitudinal corner bar diameters and 200 mm. The longitudinal reinforcement shall be positioned uniformly such that there is a bar at each corner of the links. The diameter of the corner bars shall be not less than the diameter of the links.

14.2.3.2. **T, L and I sections** : Such sections shall be divided into component rectangles for purpose of torsional design. This shall be done in such a way as to maximize the function $\Sigma(h_{max} \times h^3_{min})$ where h_{max} and h_{min} are the larger and smaller dimensions of each component rectangle. Each rectangle shall then be considered subject to a torque :

$$\frac{T\left(h_{max} \times h^3_{min}\right)}{\sum\left(h_{max} \times h^3_{min}\right)}$$

Reinforcement shall be so detailed as to tie the individual rectangles together. Where the torsional shear stress in a rectangle is less than V_{tc} no torsional reinforcement need to be provided in that rectangle.

14.2.3.3. Box section

$$V_t = T/2h_{wo} A_o$$

Where h_{wo} = is the wall thickness of members where the stress is determined;

A_o = is the area enclosed by the centre line of members forming the box.

Torsional reinforcement is to be provided such that

$$\frac{A_{sv}}{S_v} \geq \frac{T}{A_o(0.87 f_{yv})}$$

$$A_{SL} \geq \frac{A_{sv}}{S_v}\left(\frac{Perimeter\ of\ Ao}{2}\right) \times \frac{f_{yv}}{f_{yl}}$$

The detailing requirements of Clause 14.2.3.1, should still be observed. In detailing the longitudinal reinforcement to cater for torsional stresses account may be taken of those areas of the cross section subjected to simultaneous flexural compressive stresses and a lesser amount of reinforcement in the compressive zone may be taken as :

$$\text{Reduction in steel area} = f_{cav} \times \frac{\text{(Area of section subject to flexural compression)}}{0.87 f_{yc}}$$

Where f_{cav} = is the average compressive stress in the flexural compressive zone, and

f_{yc} = is the yield stress of longitudinal steel in compression.

15. MINIMUM REINFORCEMENT

15.1. General

The quantity of untensioned steel required for design or constructional purposes shall not be less than the minimum stipulated in Clauses 15.2 to 15.4. Various types of minimum steel requirements need not be added together. Bars in such reinforcement shall, however, not be placed more than 200 mm apart. The minimum diameter shall not be less than 10 mm for

severe condition of exposure and 8 mm for moderate condition of exposure.

In case of in-situ segmental construction for bridges located in marine environment continuity of untensioned reinforcement from one segment to the next shall be ensured.

15.2. In the vertical direction, a minimum reinforcement shall be provided in the bulb/web of the beams/rib of box girders, such reinforcement being not less than 0.3 per cent of the cross sectional area of the bulb/web in plan for mild steel and 0.18 per cent for HYSD bars respectively. Such reinforcement shall be as far as possible uniformly spaced along the length of the web. In the bulb portion, the cross sectional area of bulb in plan shall be taken.

In all the corners of the section, these reinforcements should pass round a longitudinal bar having a diameter not less than that of the vertical bar or round a group of tendons. For tee-beams, the arrangement in the bulb portion shall be as shown in Fig. 2.

15.3. Longitudinal reinforcements provided shall not be less than 0.25 per cent and 0.15 per cent of the gross cross sectional area of the section for mild steel and HYSD bars respectively, where the specified grade of concrete is less than M 45. In case the grade of concrete is M 45 or more, the provision shall be increased to 0.3 per cent and 0.18 per cent respectively. Such reinforcement shall as far as possible be evenly spaced on the periphery. Non-prestressed high tensile reinforcement can also be reckoned for the purpose of fulfilling the requirement of this clause.

15.4. For solid slabs and top and bottom slabs of box girders, the top and underside of the slabs shall be provided with reinforcement consisting of a grid formed by layers of bars. The minimum steel provided shall be as follows :

(i) **For solid slabs and top slab of box girders**: 0.3 per cent and 0.18 per cent of the gross cross sectional area of the slab for MS and HYSD bars respectively, which shall be equally distributed at top and bottom.

(ii) ***For soffit slab of box girders***: The longitudinal steel shall be at least 0.18 per cent and 0.3 per cent of sectional area for HYSD and MS bars respectively. The minimum transverse reinforcement shall be 0.3 per cent and 0.5 per cent of the sectional area for HYSD and MS bars respectively. The minimum reinforcement shall be equally distributed at top and bottom.

15.5. For cantilever slab minimum reinforcement of 4 nos. of 16 mm dia HYSD bars or 6 nos. of 16 mm dia MS bars should be provided with minimum spacing at the tip divided equally between the top and bottom surface parallel to support.

N.B. Notwithstanding the nomenclature "untensioned steel", this provision of reinforcement may be utilised for withstanding all action affects, if necessary.

16. COVER AND SPACING OF PRESTRESSING STEEL

16.1. Wherever prestressing cable is nearest to concrete surface, the minimum clear cover measured from outside of sheathing, shall be 75 mm.

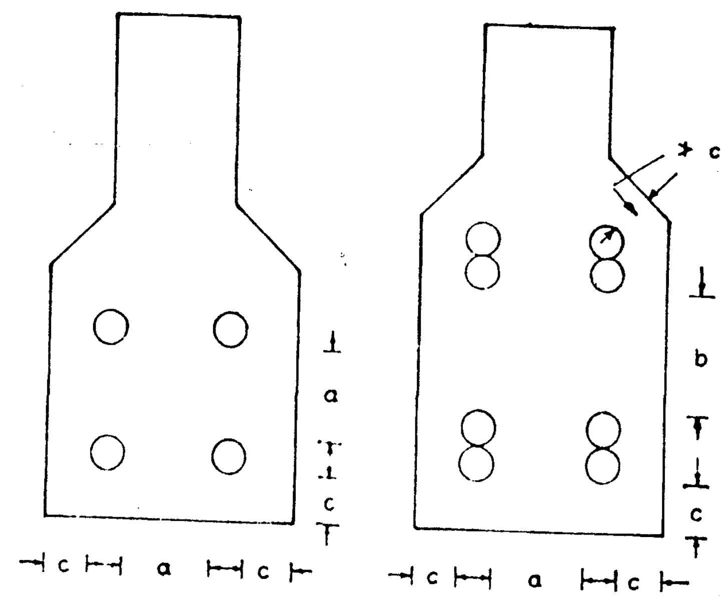
16.2. The minimum clear cover to untensioned reinforcement including links and stirrups shall be as per Clause 304.3 of IRC:21.

16.3. A minimum clear distance of 50 mm or diameter of the duct, whichever is greater, shall be maintained between individual cables when grouping of cables is not involved.

16.4. Grouped Cables

16.4.1. Grouping of cables shall be avoided to the extent possible. If unavoidable, only vertical grouping of cables, upto 2 cables may be permitted as shown in **Fig. 3.** The minimum clear spacing between groups shall be diameter of the duct or 50 mm, whichever is greater.

Note : In case of severe conditions of exposure, grouping of cables should be altogether avoided. This may be achieved by the use of high capacity strands.



a, b $\nless$ 50 mm or diameter of duct whichever is greater, C $\nless$ 75 mm

Fig. 3.

16.4.2. Individual cables or ducts of grouped cables shall be deflected or draped in the end portions of members. The clear spacing between cables or ducts in the end one metre of the members as specified in Clause 16.3 shall be maintained.

16.5. The placement of cables or ducts and the order of stressing and grouting shall be so arranged that the prestressing steel, when tensioned and grouted, does not adversely affect the adjoining ducts.

16.6. All cables shall be threaded by threading machine or any contrivance into preformed ducts. Wherever two stage prestressing is contemplated, a dummy core shall be provided in the preformed ducts of the second stage cables, which shall be pulled out after the first stage prestressing and grouting is over. Thereafter, the cables for the second stage shall be threaded into the preformed ducts. Where prestressing in more than two stages is contemplated, the above procedure shall be followed for subsequent stage cables also.

Stressing of cable/part of cable to avoid shrinkage cracks shall not be treated as a stage.

17. END BLOCKS

17.1. End block shall be designed to distribute the concentrated prestressing force at the anchorage. It shall have sufficient area to accommodate anchorages at the jacking end and shall preferably be as wide as the narrowest flange of the beam. Length of end block in no case shall be less than 600 mm nor less than its width. The portion housing the anchorages shall as far as possible be precast.

17.2. The bursting forces in the end blocks, should be assessed on the basis of the ultimate tensile strength. The bursting tensile force, F_{bst}, existing in an individual square end

block loaded by a symmetrically placed square anchorage or bearing plate, may be derived from Table 8.

Where $2\,Y_o$ = is the side of end block
$2\,Y_{po}$ = is the side of loaded area
P_k = is the load in the tendon assessed as above
F_{bst} = is the bursting tensile force.

Table 8. Design Bursting Tensile Forces in End Blocks

Y_{po}/Y_o	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_k	0.23	0.20	0.17	0.14	0.11

Notes :(i) For intermediate values linear interpolation may be made.
(ii) The values in the table above generally hold good for internal anchorages. For external anchorages the design force may be increased by 10 per cent.

This force, F_{bst}, will be distributed in a region extending from 0.2 Y_o to 2 Y_o from the loaded face of the end block as shown in **Fig. 4**.

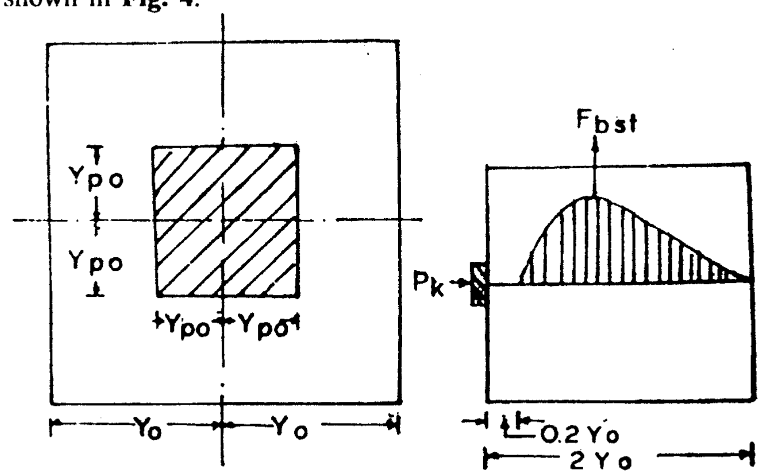


Fig. 4.

Reinforcement provided in this region to sustain the bursting tensile force may be calculated based on a tensile strength of 0.87 f_y except that the stress should be limited to a value corresponding to a strain of 0.001 when the concrete cover to the reinforcement is less than 50 mm.

In the rectangular end blocks, the bursting tensile forces in the two principal directions should be assessed on the similar basis as in Table 8.

When circular anchorages or bearing plates are used, the side of the equivalent square area should be derived.

Where groups of anchorages or bearing plates occur, the end block should be divided into a series of symmetrically loaded prisms and each prism treated in the above manner. In detailing the reinforcement for the end block as a whole, it is necessary to ensure that the groups of anchorages are appropriately tied together. Special attention should be paid to end blocks having a cross section different in shape from that of the general cross section of the beam and reference should be made to specialist literature. Compliance with the above requirements will generally ensure that bursting tensile forces along the loaded axis are provided for. In case where large concentrated tendon forces are involved alternative methods of design based on specialist literature and manufacturer's data may be more appropriate.

17.3. Consideration should also be given to the spalling tensile stresses that occur in end blocks. Where the anchorage or bearing plates are highly eccentric, these stresses reach a maximum at the loaded face. The end face of anchorage zone shall be continuously reinforced to prevent edge spalling.

Reinforcement shall be placed as close to the end face as possible.

18. THICKENING OF WEBS OF GIRDERS

The thickening of webs of girders towards the end blocks shall be achieved gradually with a splay in plan of not steeper than 1 in 6. Suitable thickening for isolated anchorages away from the end blocks shall be made whenever necessary to reduce stress concentration.

19. ANCHORAGE OF CABLES AND STRESSING

Anchorage of cables in the top deck surface shall not be permitted. All anchorages shall be properly sealed after prestressing and grouting operations. All wires/strands in one cable should be stressed simultaneously by using multi-stressing jack.

20. SPLAY OF CABLES IN PLAN AND MINIMUM RADIUS OF CABLES IN ELEVATION

The splay of cables in plan, for bringing them from their position in the bottom flange at mid-span into the web towards the supports shall not be more than 1 in 6. The points of splay shall be suitably staggered on both sides of the longitudinal centre line of the web of the girder. The minimum radius of curvature, spacing and cover for curved cables shall be specified to ensure that bursting of the side cover both perpendicular to the plane of curvature and in the plane of curvature of the ducts does not take place. Guidance in this regard may be taken from BS:5400: Part 4: (Appendix-D) subject to spacing and cover stipulations given in Clause 16.

21. SLENDER BEAMS

Slender beams are those in which :

(a) ratio of span to width of top flange is more than 60, and

(b) ratio of width of top flange to the depth of beam is less than 1/4.

For such beams permissible stress shall be reduced suitably and they shall also be provided with adequate temporary restraints during handling and erection which should be investigated.

22. EMERGENCY CABLES/STRANDS

Besides the design requirements, additional cables/strands shall be symmetrically placed in the structure so as to be capable of generating prestressing force of about 4 per cent of the total design prestressing force in the structure. Only those cables which are required to make up the deficiency shall be stressed and the remainder pulled out and the duct hole shall be grouted.

23. STORAGE AND HANDLING OF PRESTRESSING MATERIALS

A recommended practice for storage and handling of prestressing material is given in *Appendix-3*.

24. PRESTRESSING OPERATION

A recommended practice for prestressing operations is given at *Appendix-4*.

25. GROUTING OF CABLES

A recommended practice for grouting of cable is given at *Appendix-5*.

TESTS ON SHEATHING DUCTS

(1) All tests specified below shall be carried out on the same sample in the order given below.

(2) At least 3 samples for one lot of supply (not exceeding 7000 metre length) shall be tested.

3. WORKABILITY TEST

A test sample 1100 mm long is soldered to a fixed base plate with a soft solder **(Fig. 1A-1)**. The sample is then bent to a radius of 1800 mm alternately on either side to complete 3 cycles.

Thereafter, the sealing joints will be visually inspected to verify that no failure or opening has taken place.

4. TRANSVERSE LOAD RATING TEST

The test ensures that stiffness of the sheathing is sufficient to prevent permanent distortion during site handling.

The sample is placed on a horizontal support 500 mm long so that the sample is supported at all points of outward corrugations.

A load as specified in Table below is applied gradually at the centre of the supported portion through a circular contact surface of 12 mm dia. Couplers shall be placed so that the load is applied approximately at the centre of two corrugations, **Fig. 1A-2**. The load as specified below is applied in increments.

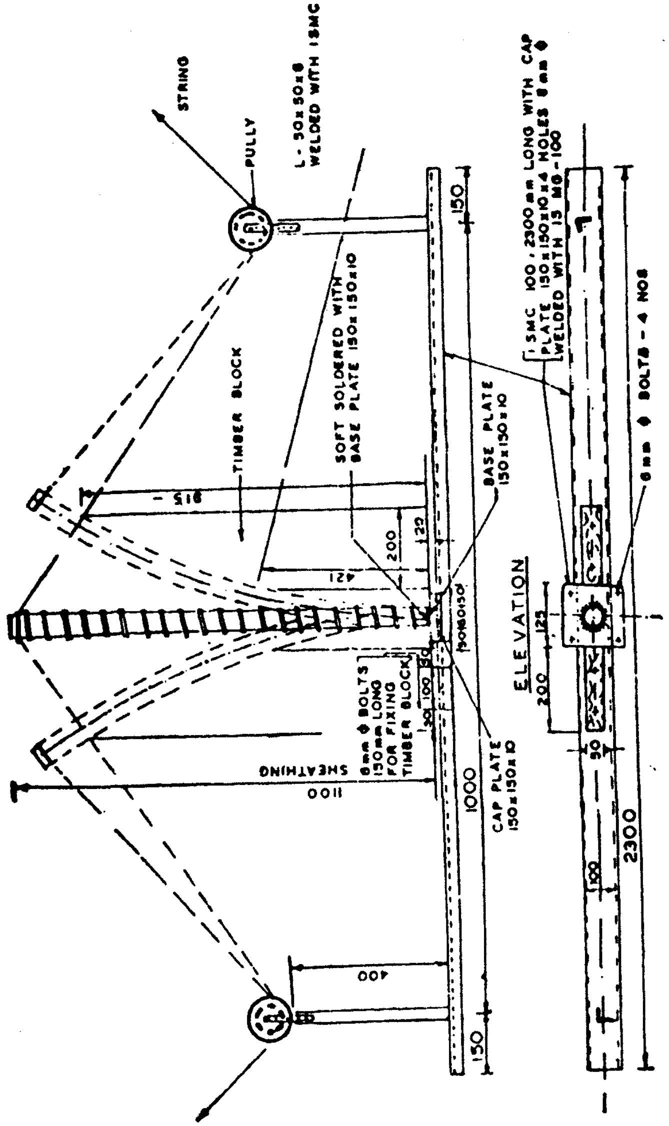


Fig. 1A-1. Workability test

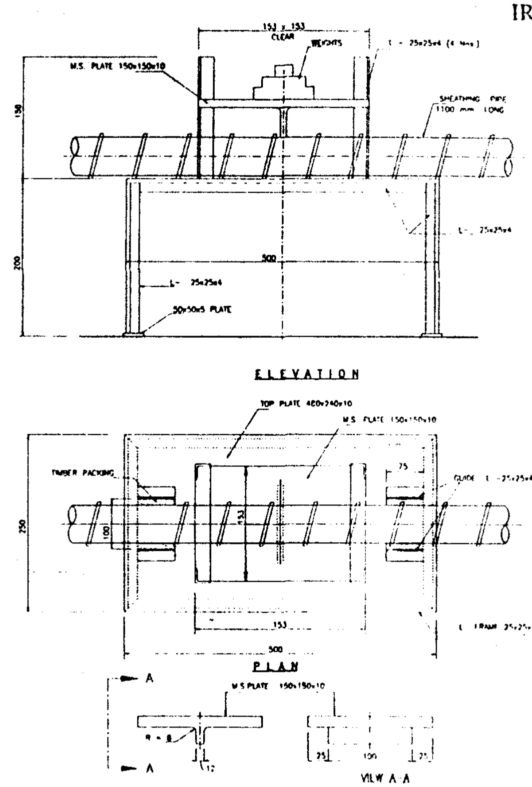


Fig. 1A-2. Transverse load rating test

Table

		more than	more than	more than	more than	more than	more than
Dia :	25 mm	35 mm	45 mm	55 mm	65 mm	75 mm	85 mm
	to	up to	up to	up to	up to	up to	up to
	35 mm	45 mm	55 mm	65 mm	75 mm	85 mm	90 mm
Load :	250 N	400 N	500 N	600 N	700 N	800 N	1000 N

The sample is considered acceptable if the permanent deformation is less than 5 per cent.

5. TENSION LOAD TEST

The test specimen is subjected to a tensile load. The hollow core is filled with a wooden circular piece having a diameter of 95 per cent of the inner dia of the sample to ensure circular profile during test loading, **Fig. 1A-3**.

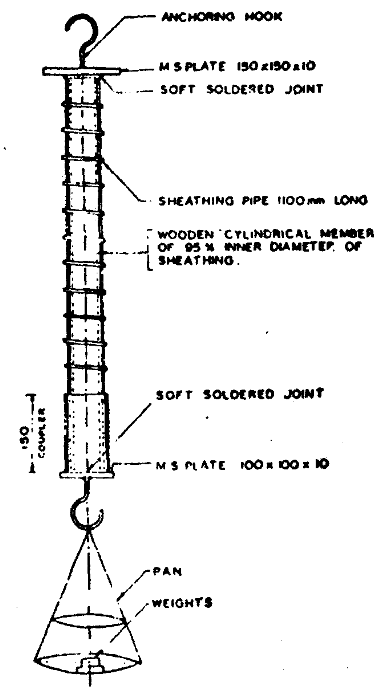


Fig. 1A-3. Tension load test

A coupler is screwed on and the sample loaded in increments, till specified load. If no deformation of the joints nor slippage of couplers is

noticed, the test shall be considered satisfactory :

Dia in mm	*Load*
25 to 35	300 *N*
35 to 45	500 *N*
45 to 55	800 *N*
55 to 65	1100 *N*
65 to 75	1400 *N*
75 to 85	1600 *N*
85 to 90	1800 *N*

6. WATER LOSS TEST

The sample is sealed at one end. The sample is filled with water and after sealing, the end is connected to a system capable of applying a pressure of 0.05 MPa, **Fig. 1A-4** and kept constant for 5 minutes, hand pump and pressure gauge or stand pipe system can be used.

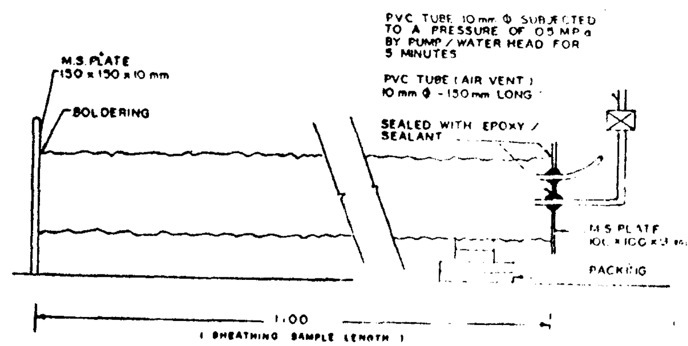


Fig. 1A-4. Test for water loss study

The sample is acceptable if the water loss does not exceed 1.5 per cent of the volume. The volume is worked out as follows:

Another sample 500 mm long is sealed at one end and the volume of hollow space arrived at by pouring water from a measuring cylinder.

The computation of relative profile volume is worked out as follows:

V_a - Premeasured quantity of water in a measuring cylinder

V_b - Balance quantity of water left in the cylinder after completely filling of the test sample

Actual Volume 'V_p' = $V_a - V_b$

$$\text{Relative Profile Volume} = \frac{V_p - \frac{\pi \phi^2 l}{4}}{\pi \phi l} \ \text{cm}^3/\text{cm}^2$$

Where l is length of specimen and ϕ internal nominal dia. of sheathing.

Appendix-1B

TESTS ON CORRUGATED HDPE SHEATHING DUCTS

The additional acceptance tests for the prestressing systems employing corrugated HDPE ducts shall cover the following two tests:

1. BOND TEST

Object: To establish satisfactory bond characteristics between the tendon and concrete, in the ultimate condition.

Equipment

* 3 nos. similar reinforced concrete beams with a HDPE duct of length equal to 40 times the duct diameter,
* Prestressing tendon of adequate length for stressing and for embedding in the beam,
* Tendon anchorage system,
* Load cells and meters,
* Grout constituents.

Method: Cast an adequately reinforced beam to withstand the prestressing operation and of length to embed 40 times the dia. of duct to suit the tendon to be adopted. Introduce the strands of the tendon by spacing them parallel by means of ply-spacers as shown in **Fig. 1B-1** and fill the duct with grout of strength not less than 27 N/sq.mm. When the grout has attained the necessary strength, stress the tendon slowly increasing the load to the failure capacity. The failure capacity of the bond shall be at least equal to the anchorage efficiency or 0.95 of failure capacity of the tendon. At least 3 nos. of tests shall be carried out to ascertain the adequacy of the duct.

2. COMPRESSION TEST FOR THE LOSS OF WALL THICKNESS

Object: To establish the wear and tear of the sheathing material and the rigidity of the duct surface against indentation and abrasion under concentrated line loading from the tendon constitutents.

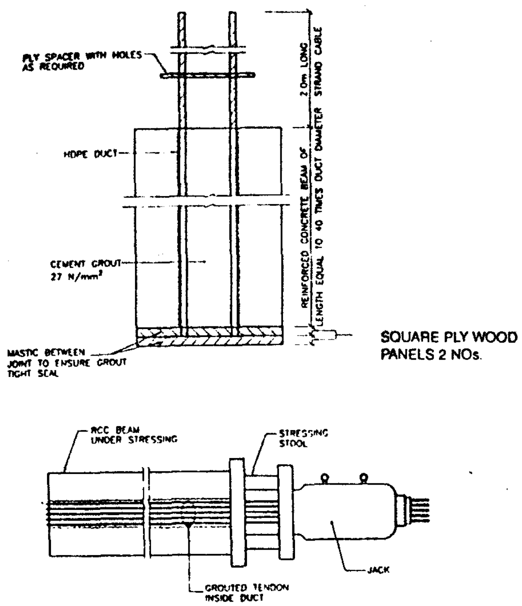


Fig. 1B-1. Bond test arrangement

Equipment:

* 3 nos. of concrete blocks
* 1 no. of 1000 mm long strand forming the tendon
* A 3 MN press
* A loading beam of 300 mm length to transmit 5 kN load
* A rubber pad for placing between the press and the beam for uniform and constant load transfer
* A bearing plate with a mono strand jack to pull the strand under loaded condition
* A digital calliper

Method: Cast 3 nos. of concrete cubes of 300 mm size, of the same strengt as of main structure, with half cut HDPE sheathing ducts embedded in it a the top as shown in **Fig. 1B-2.** Care shall be exercised to ensure that the duc surface has uniform contact with concrete all around. Place the concret block over the press with a 1000 mm length of strand forming the tendo placed in the duct and apply the 5 kN uniform load gradually as shown. Pul the strand under the stressed condition by 200 mm across the duct. Repea the test on all the 3 nos. of ducts so embedded. Measure the indentation formed in all the 3 nos. of ducts along the length of the strand, by means o digital calliper. The residual thickness of the duct shall not be less than 1. mm.

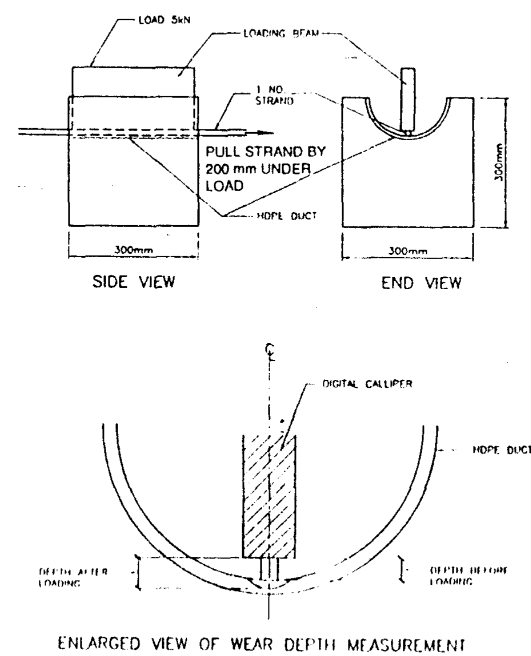


Fig. 1B-2. Compression test arrangement

Appendix-2

SPECIFICATION FOR SHEATHING DUCT JOINTS

The sheathing ducts shall be of the spiral corrugated type. For major projects, the sheathing ducts should preferably be manufactured at the project site utilising appropriate machines. With such an arrangement, long lengths of sheathing ducts may be used with consequent reduction in the number of joints and couplers.

Where sheathing duct joints are unavoidable, such joints shall be made cement slurry tight by the use of corrugated threaded sleeve couplers which can be tightly screwed on to the outer side of the sheathing ducts. A heat-shrink coupler could also be used if suitable.

Typical details of a sleeve coupler is shown in **Fig. 2.1**. The length of the coupler should not be less than 150 mm but should be increased upto 200 mm wherever practicable. The joints between the ends of the coupler and the duct shall be sealed with adhesive sealing tape to prevent penetration of cement slurry during concreting. The couplers of adjacent ducts should be staggered wherever practicable. As far as possible, couplers should not be located in curved zones. The corrugated sleeve couplers are being conveniently manufactured using the sheath making machine with the next higher size of die set.

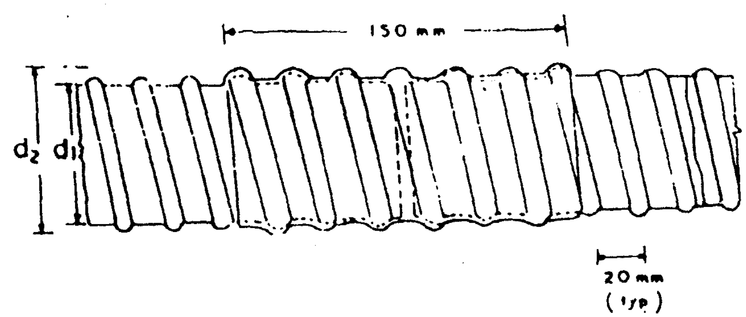


Fig. 2.1

The heat-shrink coupler Fig. 2.2 is supplied in the form of bandage rolls which can be used for all diameters of sheathing ducts. The bandage is

coated on the underside with a heat sensitive adhesive so that after heating the bandage material shrinks on to the sheathing duct and ensures formation of a leak proof joint, without the need for extra taping or support in the form of corrugated sleeve couplers. The heating is effected by means of a soft gas flame.

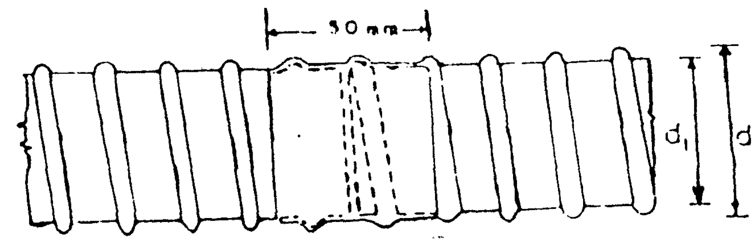


Fig. 2.2

Appendix-3

RECOMMENDED PRACTICE FOR STORAGES AND HANDLING OF PRESTRESSING MATERIAL

1. All prestressing steel, sheathing, anchorages and sleeves or couplings shall be protected during transportation, handling and storage. For wires upto 5 mm dia, coils of about 1.5 m dia, and for wires above 5 mm dia, coils of about 2 m dia without breaks and joints shall be obtained from the manufacturer.

2. Materials shall be stored in accordance with the provisions contained in relevant specifications. All efforts shall be made to store the materials in proper places so as to prevent their deterioration or intrusion by foreign matter and to ensure their satisfactory quality and fitness for the work. The storage space shall also permit easy inspection, removal and re-storage of the materials.

3. The prestressing steel, sheathing and other accessories shall be stored under cover and protected from rain or damp ground. These shall also be protected from the ambient atmosphere if it is likely to be aggressive. All prestressing steel shall be provided with temporary protection during storage such as coating of soluble oils, silica gel or vapour phase inhibiting materials of proven specifications.

4. Storage at site shall be kept to the absolute minimum. All materials even though stored in approved godowns shall be subjected to acceptance test prior to their immediate use.

Appendix-4

RECOMMENDED PRACTICE FOR PRESTRESSING OPERATIONS

Prestressing operation and grouting shall be entrusted to only specially trained and qualified personnel. All prestressing accessories shall be procured from authorised manufacturers with in-house testing facilities. Contractors shall also be required to engage specialised agencies who should also be entrusted with the total service contract for fabrication of cables, protection of cables during concreting, prestressing and grouting. Necessary certificates shall also be accorded by such specialised agencies that the work has been carried out in accordance with prescribed specifications. In exceptional cases where the client is convinced that the contractor of the bridge itself is well experienced and has qualified personnel and sufficient track record to substantiate his performance in the particular system of prestressing being adopted, the prestressing and grouting operations could be entrusted to the contractor.

Appendix-5

RECOMMENDED PRACTICE FOR GROUTING OF POST-TENSIONED CABLES IN PRESTRESSED CONCRETE BRIDGES

1. GENERAL

1.1. The recommendations cover the cement grouting of post-tensioned tendons of prestressed concrete members of bridges. This also covers some of the essential protective measures to be adopted for minimising corrosion in PSC bridges.

1.2. The purpose of grouting is to provide permanent protection to the post-tensioned steel against corrosion and to develop bond between the prestressing steel and the surrounding structural concrete. The grout ensures encasement of steel in an alkaline environment for corrosion protection and by filling the duct space, it prevents water collection and freezing.

2. MATERIALS

2.1. **Water**

Only clean potable water free from impurities conforming to Clause 3.3 of this criteria shall be permitted. No sea or creek water is to be used at all.

2.2. **Cement**

Ordinary Portland Cement should be used for preparation of the grout. It should be as fresh as possible and free from any lumps. Pozzolana cement shall not be used.

2.3. **Sand**

It is not recommended to use sand for grouting of prestressing tendons. In case the internal diameter of the ducts exceed 150 mm, use of sand may be considered. Sand, if used, shall conform to IS: 383 and shall pass through IS Sieve No. 150. The weight of sand in the grout shall not be more than 10 per cent of the weight of cement, unless proper workability can be ensured by addition of suitable plasticizers.

2.4. **Admixtures**

Acceptable admixtures conforming to IS: 9103 may be used if tests have shown that their use improves the properties of grout, i.e. increasing fluidity, reducing bleeding, entraining air or expanding the grout. Admixtures must not contain chlorides, nitrates, sulphides, sulphites or any other products which are likely to damage the steel or grout. When an expanding agent is used, the total unrestrained expansion should not exceed 10 per cent. Aluminium powder as an expanding agent is not recommended for grouting because its long term effects are not free from doubt.

2.5. **Sheathing**

2.5.1. For specifications sheathing, Clause 3.6 of this criteria may be referred to.

2.5.2. **Grout openings or vents**

(a) All ducts should have grout opening at both ends. For this purpose special openings should be provided where such openings are not available at end anchorages. For draped (curved) cables crown points should have a grout vent. For draped cables longer than 50 m grout vents or drain holes may be provided at or near the lowest points. It is a good practice to provide additional air vents at suitable intervals. All grout openings or vents should include provisions for preventing grout leakage.

(b) Standard details of fixing couplers, inlets, outlets and air vents to the duct/anchorage shall be followed as recommended by the supplier of the system of prestressing.

2.5.3. Ducts should be securely fastened at close intervals. All unintended holes or openings in the duct must be repaired prior to concrete placing. The joints of the couplers and the sheathing should be made water proof by use of tape or similar suitable system capable of giving leak proof joints. Grout openings and vents must be securely anchored to the duct and to either the forms or to reinforcing steel to prevent displacement during concreting operations due to weight, buoyancy and vibrations.

2.5.4. Ducts require very careful handling as, being of thin metal, they are susceptible to leakage due to corrosion in transit or storage, by tearing ripping in handling particularly when placed adjoining to reinforcing steel, by pulling apart of joints while inserting tendons prior to concreting, or by accidental puncturing while drilling for form ties/inserts. Ducts are also liable to damage by rough use of internal vibrator and sparks from welding being done close by.

3. EQUIPMENT

3.1. Grout Colloidal Mixer

It is essential that the grout is maintained in a homogenous state and of uniform consistency so that there is no separation of cement during entire grouting process. It is, therefore, necessary that the grout be continuously mixed in a colloidal mixer with a minimum speed of 1000 RPM and travel of discharge not exceeding 15 m per second.

3.2. Grout Pump

The pump should be a positive displacement type and should be capable of injecting the grout in a continuous operation and not by way of pulses. The grout pump must be fitted with a pressure gauge to enable pressure of injection to be controlled. The minimum pressure at which grout should be pumped shall be 0.3 MPa and the grout pump must have a relief arrangement for bypass of the grout in case of build up of pressure beyond 1 MPa. The capacity of the grout pump should be such as to achieve a forward speed of grout of around 5 to 10 metres per minute. The slower rates are preferable as they reduce the possibility of occurrence of voids. If the capacity of the pump is large, it is usual to grout two or more cables simultaneously through a common manifold.

Use of hand pumps for grouting is not recommended. Use of compressed air operated equipment for injection is prohibited as it is likely that there will be some air entrapped in grout.

3.3. Water Pump

Before commencement of grouting, a stand by direct feed high pressure water pump should be available at site for an emergency.

In case of any problem in grouting the ducts, such pump shall immediately be connected to the duct and all grout flushed by use of high pressure water flushing. It is, therefore, necessary to have adequate storage of clean potable water for operation of the water pump for such emergencies.

3.4. Grout Screen

The grouting equipment should contain a screen having a mesh size of IS: 106 (IS:150 if sand is used). Prior to introduction into the grout pump, the grout should be passed through such screen. This screen should be easily accessible for inspection and cleaning.

3.5. Connections and Air Vents

Standard details of fixing inlets, outlets, and air vents to the sheathing and/or anchorage should be followed as recommended by specialist supplier of the system of prestressing. In general, all connections are to be of the "Quick couple" type and at change of diameters suitable reducers are to be provided.

4. PROPERTIES OF THE GROUT

4.1. Water/cement ratio should be as low as possible, consistent with workability. This ratio should not normally exceed 0.45.

4.2. The temperature of the grout after accounting for the ambient temperature of the structure shall not exceed 25°C.

4.3. Before grouting, the properties of the grout mix should be tested in a laboratory depending on the facilities available. Tests should be conducted for each job periodically. The recommended test is described below.

4.3.1. Compressive strength: The compressive strength of 100 mm cubes of the grout shall not be less than 17 MPa at 7 days. Cubes shall be cured in a moist atmosphere for the first 24 hours and subsequently in water. These tests shall be conducted in advance to ascertain the suitability of the grout mix.

5. MIXING OF GROUT

5.1. Proportions of materials should be based on field trials made on the grout before commencement of grouting, but subject to the limits specified above. The materials should be measured by weight.

5.2. Water should be added to the mixer first, followed by portland cement and sand, if used. Admixture, if any, may be added as recommended by the manufacturer.

5.3. Mixing time depends upon the type of the mixer but will normally be between 2 and 3 minutes. However, mixing should be for such a duration as to obtain uniform and thoroughly blended grout, without excessive temperature increase or loss of expansive properties of the admixtures. The grout should be continuously agitated until it is injected.

5.4. Once mixed, no water shall be added to the grout to increase its fluidity.

5.5. Hand mixing is not permitted.

6. GROUTING OPERATIONS

6.1. General

(a) Grouting shall be carried out as early as possible but not later than 2 weeks of stressing a tendon. Whenever this stipulation cannot be complied with for unavoidable reasons, adequate temporary protection of the steel against corrosion by methods or products which will not impair the ultimate adherence of the injected grout should be ensured till grouting. The sealing of the anchorage ends after concreting is considered to be a good practice to prevent ingress of water. For structures in aggressive environment, sealing of the anchorage ends is mandatory.

Notes : 1. Application of some patented water soluble oils for coating of steel/VPI powder injection/sending in of hot, dry, oil-free compressed air through the vents at frequent intervals have shown some good results.

2. Some of the methods recommended for sealing of anchorages are to seal the openings with bitumen

impregnated gunny bag or water proof paper or by building a brick pedestal plastered on all faces enclosing the exposed wires outside the anchorages.

(b) Any traces of oil if applied to steel for preventing corrosion should be removed before grouting operation.

(c) Ducts shall be flushed with water for cleaning as well as for wetting the surfaces of the duct walls. Water used for flushing should be of same quality as used for grouting. It may, however, contain about 1 per cent of slaked lime or quick lime. All water should be drained through the lowest drain pipe or by blowing compressed air through the duct.

(d) The water in the duct should be blown out with oil free compressed air.

Blowing out water from duct for cables longer than 50 m draped up at both ends by compressed air is not effective, outlet/vent provided at or near the lowest point shall be used to drain out water from duct.

(e) The connection between the nozzle of the injection pipe and duct should be such that air cannot be sucked in.

(f) All outlet points including vent openings should be kept open prior to commencement of injection grout.

(g) Before grouting, all air in the pump and hose should be expelled. The suction circuit of the pump should be air-tight.

6.2. Injection of grout

(a) After mixing, the grout should be kept in continuous movement.

(b) Injection of grout must be continuous and should not be interrupted.

(c) For vertical cable or cables inclined more than 60° to the horizontal injection should be effected from the lowest anchorage or vent of the duct.

(d) The method of injection should ensure complete filling of the ducts. To verify this, it is advisable to compare the volume of the space to be filled by the injected grout with the quantity of grout actually injected.

(e) Grouting should be commenced initially with a low pressure of injection of upto 0.3 MPa increasing it until the grout comes out at the other end. The grout should be allowed to flow freely from the other end until the consistency of the grout at this end is the same as that of the grout at the injection end. When the grout flows at the other end, it should be closed off and build up of pressure commenced. Full injection pressure at about 0.5 MPa shall be maintained for at least one minute before closing the injection pipe. It is a recommended practice to provide a stand pipe at the highest point of the tendon profile to hold all water displaced by sedimentation or bleeding. If there is a build up of pressure much in excess of 1 MPa without flow of grout coming at the other end, the grouting operation should be discontinued and the entire duct flushed with high pressure water. Also, the bypass system indicated in para 3.2 above is essential for further safety.

(f) In the case of cables draped downwards e.g. in cantilever construction simultaneous injection from both ends may be adopted **Fig. 5.1**.

(g) Grout not used within 30 minutes of mixing should be rejected.

(h) Disconnection is facilitated if a short length of flexible tube connects the duct and injection pipe. ~~This~~ can be squeezed and cut off after the grout has hardened.

7. PRECAUTIONS AND RECOMMENDATIONS FOR EFFECTIVE GROUTING

(a) In cold and frosty weather, injection should be postponed unless special precautions are taken. If frost is likely to occur within 48 hours after injection, heat must be applied to the member and maintained for at least 48 hours after injection so that the temperature of the grout does not fall below 5°C. Prior to commencement of grout, care must be taken to ensure that the duct is completely free of frost/ice by flushing with warm water, but not with steam.

(b) When the ambient temperature during the day is likely to exceed 40°C, grouting should be done in the early morning or late evening hours.

(c) When the cables are threaded after concreting, the duct must be temporarily protected during concreting by inserting a stiff rod or a rigid PVC pipe or any other suitable method.

(d) During concreting, care shall be taken to ensure that the sheathing is not damaged. Needle vibrators shall be used with extreme care by well experienced staff only, to ensure the above requirements.

(e) It is a good practice to move the cables in both directions during the concreting operations. This can easily be done by light hammering the ends of the wires/strands during concreting. It is also advisable that 3 to 4 hours after concreting the cable should be moved both ways through a distance of about 20 cms. With such movement, any leakage of mortar which has taken place in spite of all precautions, loses bond with the cables, thus reducing the chance of blockages. This operation can also be done by fixing prestressing jacks at one end pulling the entire cable and then repeating the operation by fixing the jack at the other end.

(f) The cables to be grouted should be separated by as much distance as possible.

(g) In case of stage prestressing, cables tensioned in the first stage should not remain ungrouted till all cables are stressed. It is a good practice, while grouting any duct in stage prestressing, to keep all the remaining ducts filled up with water containing 1 per cent lime or by running water through such ducts till the grout has set. After grouting the particular cable, the water in the other cables should be drained and removed with compressed air to prevent corrosion.

(h) Care should be taken to avoid leaks from one duct to another at joints of precast members.

(i) End faces where anchorages are located are vulnerable points of entry of water. They have to be necessarily protected with an

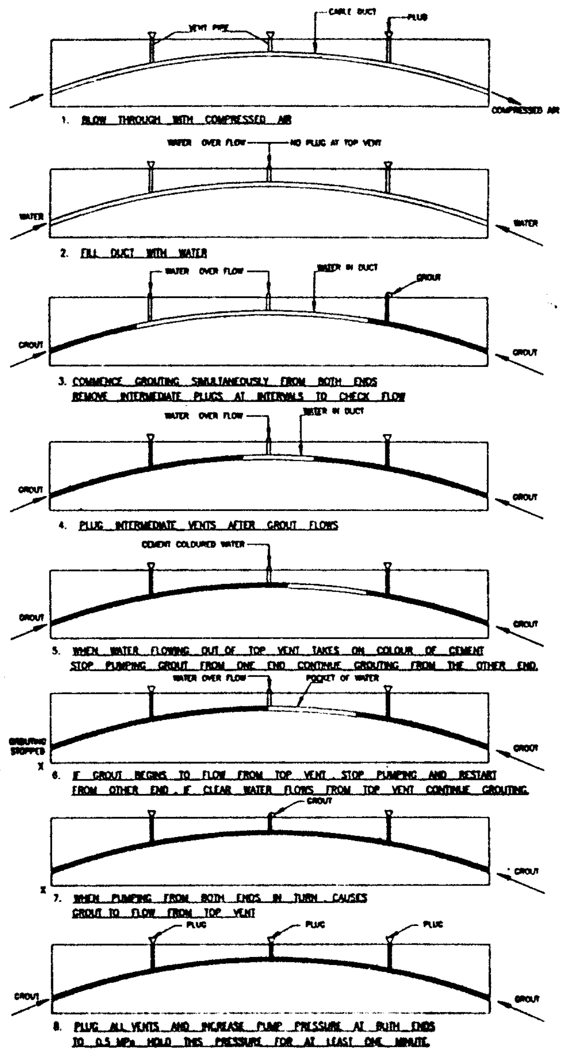


Fig. 5.1. Procedure for grouting of cables draped downwords

effective barrier. Recesses should be packed with mortar concrete and should preferably be painted with water proof paint.

(j) After grouting is completed, the projecting portion of the vents should be cut off and the face protected to prevent corrosion.

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SECTION VI
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CONTENTS

SECTION VI
Composite Construction

COMPOSITE CONSTRUCTION

INTRODUCTION

The object of the Standard Specifications and Codes of Practice published by the Indian Roads Congeress 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 for engineers, engaged in the design and/or construction of road bridges. The provisions herein shall be used with discretion and care shall be taken to ensure that the stability and soundness of the structures designed and/or constructed as per these provisions are satisfactory.

The design and construction of road bridges require an extensive and thorough 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.

The Bridges Committee in their meeting held at New Delhi on the 16th & 17th September, 1975 constituted a Subcommittee (personnel given below) to revise IRC : 22-1966 'Standard Specifications and Code of Practice for Road Bridges—Section VI (Composite Construction)'.

Amitava Banerjee	... Convenor	
K.S. Rakshit	... Member-Secretary	
Dr. N.S. Bahl	G. Venkatesulu	
P.C. Bhasin	A Rep. of Gammon India Ltd.	
Dr. P. Ray Chaudhuri	Rep. of M/s Stup (Consultants) (M.C. Tandon)	
Gouranga Ganguly		
A. Ghosal	Rep. of R.D.S.O. (Satya Bhushan)	
S.K. Ghosh	Rep. of I.S.I. (G. Raman)	
P.V. Naik	Rep. of D.G.B.R. (R.V. Ramamurthy)	
U.T. Khemani		
Dr. K. Sreenivasa Rao	Rep. of SERC, Roorkee (D.S. Prakash Rao)	
Dr. V.K. Raina		
K.B. Sarkar		
The President, Indian Roads Congress (K. Tong Pang Ao)		*-Ex-officio*
The Director General (Road Development) and Addl. Secretary to the Govt. of India (K.K. Sarin)		*-Ex-officio*
The Secretary, Indian Roads Congress (Ninan Koshi)		*-Ex-officio*

The Subcommittee held fourteen meetings on 16th and 17th July, 8th December, 1976, 12th and 13th May, 29th and 30th September, 1977, 13th and 14th March, 10th and 11th July, 26th and 27th December, 1978, 6th and 7th July 1979, 9th and 10th April, 24th and 25th November, 1980, 20th and 21st August 1981, 26th August, 6th to 8th December, 1982 and 2nd and 3rd March, 1983.

The Bridge Code Section VI as revised by the Subcommittee was considered and finalised by the Bridges Committee in their meeting held at New Delhi on the 20th August, 1985. This Standard Specifications and Code of Practice for Road Bridges, Section VI (Composite Construction) has been thoroughly revised to cope with the modern technological developments which have taken place in this field of engineering in various countries of the world.

Later on the draft document was approved by the Executive Committee through circulation in January, 1986 and was placed before the Council in their 115th meeting held at Bhopal on the 1st February, 1986. The Council while approving the revised draft (First Revision) for being published by the Indian Roads Congress, authorised the Convenor of the Bridges Committee to make any changes which may be considered necessary in light of the comments offered by the members.

As directed by the Council, the draft was reviewed by the Convenor of the Bridges Committee, and editorial corrections where considered necessary were incorporated in the text.

600. GENERAL

600.1. Scope

This code will apply to simply supported bridges of composite construction. Where appropriate, the requirements of this specification may be applied to other types of bridges with amendments, as necessary. This code is applicable to Box-Girders only when special considerations based on available information have to be made.

600.2. Types

Bridges of composite section covered by this code include the following types :

(i) Reinforced concrete or prestressed concrete slab with structural steel girders.

(ii) Reinforced concrete or prestressed concrete slab with pre-cast reinforced concrete or prestressed concrete girders.

600.3. Notations

A = Area

A_b = Area of steel at bottom

A_c = Area of composite girder

A_h = Area of steel at haunch

A_p = Area of prestressing steel

A_s = Area of in-situ slab, area of steel

A_{sl} = Area of steel in longitudinal direction

A_t = Area of steel at top

b = Width

b_f = Width of flange

b_h = Width of haunch

c_1, c_2 = Covers

d = Diameter/Effective depth

d_h = Depth of haunch

E_c = Modulus of elasticity of concrete at 28 days

$E_{ci}\ (s)$ = Modulus of elasticity of concrete of in-situ slab at i days

$E_{cj}\ (p)$ = Modulus of elasticity of pre-cast girder at j days

$E_c\ (p)$ = Modulus of elasticity of concrete of prefabricated girder at 28 days

$E_c\ (s)$ = Modulus of elasticity of concrete for in-situ slab at 28 days

E_s = Modulus of elasticity of steel

f_{ck} = Characteristic compressive strength of 150 mm cubes at 28 days

f_u = Ultimate strength of steel

f_y = Yield strength of steel

I Moment of inertia

H, H_1, H_2. - Horizontal forces

h, h_1 = Depth; thickness

h_f = Depth of flange

i, j = Age of concrete in days

k = Coefficient

L = Length

L_s = Length of shear plane

M_u = Ultimate moment of resistance of composite section

m = Modular ratio

P = Pitch/spacing

Q_r = Allowable range of shear resistance per connector

Q_u = Ultimate shear resistance of each connector

V = Vertical shear

V_R = Range of vertical shear

V_L = Longitudinal or horizontal shear per unit length

V_r = Range of horizontal shear per unit length

$\bar{Y}$ = Distance of the centroid of the area under consideration from the neutral axis of the composite section

λ = Load reduction factor.

600.4. **Units**

The units and their symbols shall be adopted as given in Appendix II of IRC : 71.

600.5. **Reference to Other Related Codes**

Apart from design requirements specified herein, for other relevant design requirements in respect of reinforced concrete, structural steel or prestressed concrete, IRC : 18, IRC : 21 and IRC : 24, respectively shall be followed.

601. TERMINOLOGY

601.1. **Composite Action**

The acting together of the girder and slab as a unit ensured by the use of mechanical device known as shear connectors.

601.2. **Shear Connectors**

There are three main types of shear connectors, viz., Rigid Shear Connector, Flexible Shear Connector and Anchorage Shear Connector.

601.2.1. **Rigid shear connector :** Rigid shear connector consists of short length bars, stiffened angles, channels or tees welded on to the flange of the steel girders and derives resistance to horizontal shear by bearing against concrete (some of this type are shown in Fig. 1). Such connectors should be provided with anchorage devices as shown in Fig. 2.

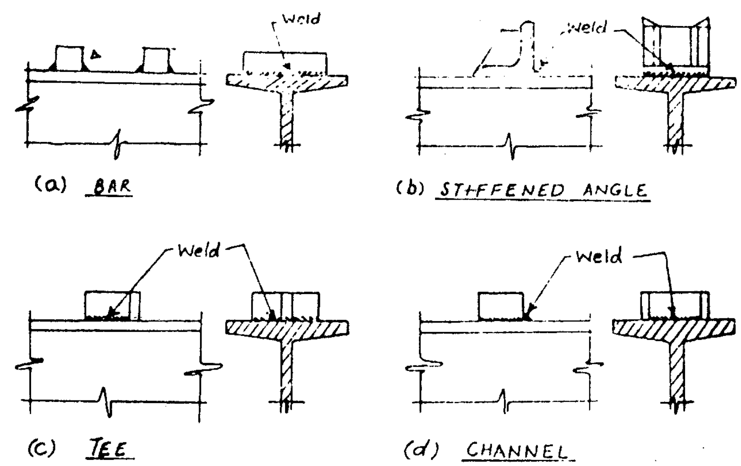


Fig. 1. Rigid connectors

601.2.2. **Flexible shear connector :** Flexible shear connector consists of studs, channels, angles or tees welded to steel girders and derives resistance to horizontal shear through bending of the connectors (some of this type are shown in Fig. 3). Where necessary, such shear connectors shall be provided with anchorages device (see Clause 611.3.2.).

601.2.3. **Anchorage shear connector :** This connector is used to resist horizontal shear and to prevent separation of the girder from the concrete slab at the interface through bond. Fig. 4.

601.3. **Differential Shrinkage**

It is the difference between the shrinkage of the two elements from the time composite action comes into play.

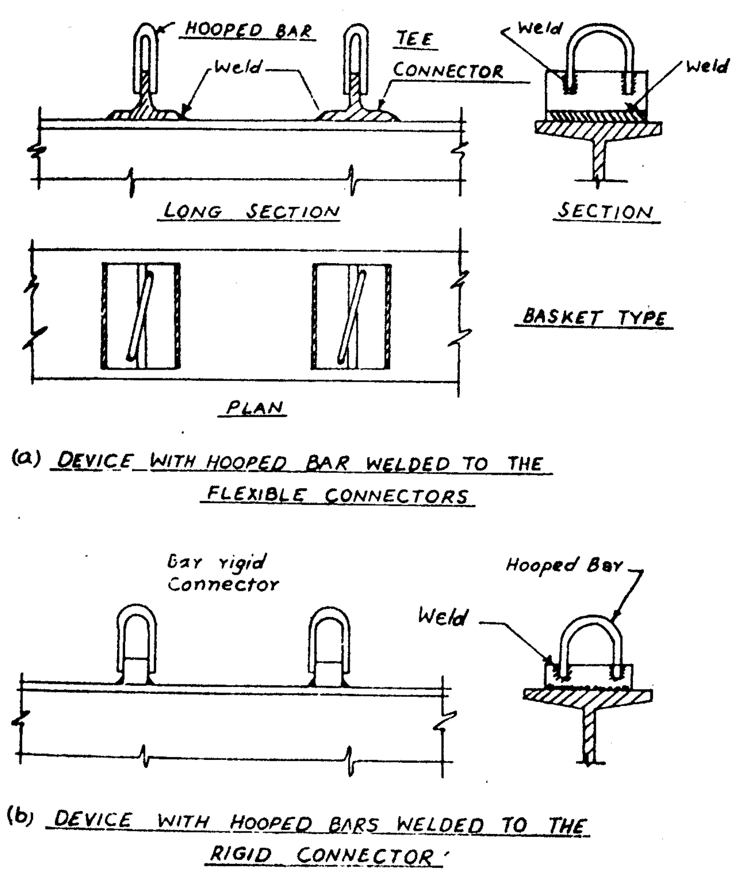


Fig. 2 Mechanical device to prevent vertical separation

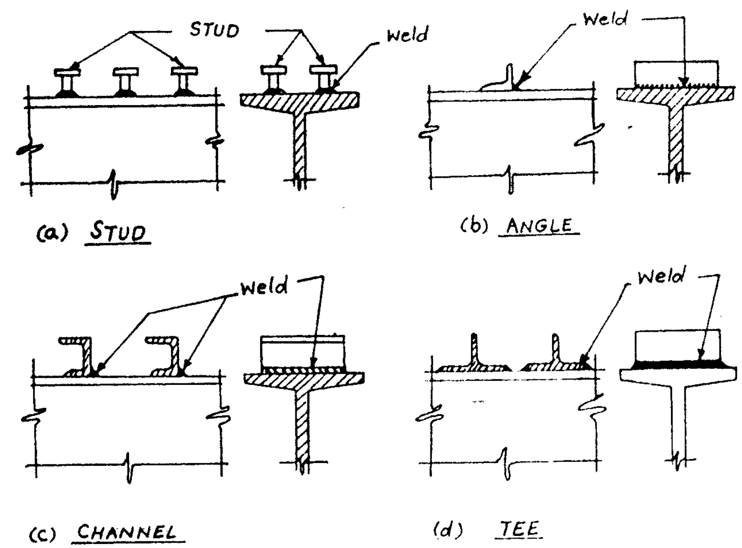


Fig. 3. Flexible connectors

Note : Hoop anchorage to resist vertical separation not shown in Fig. 1 and Fig. 3 (b) and (d)

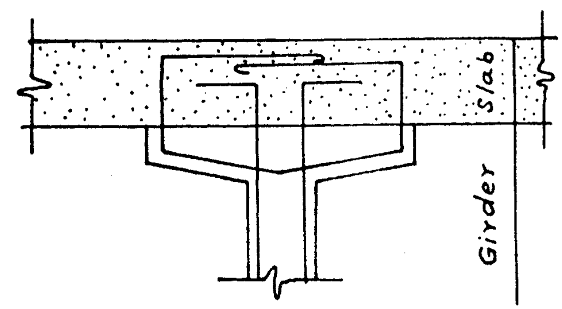


Fig. 4. Anchorage connectors

602. **PROPERTIES OF MATERIALS**

For properties of materials, viz., modulus of elasticity of concrete and steel, creep and shrinkage of concrete, relaxation of steel, IRC : 18, IRC : 21 and IRC : 24 as applicable, shall be referred to.

603. GENERAL STRUCTURAL REQUIREMENT

Relevant clauses regarding durability of structures as provided in appropriate codes shall be followed.

Members which are subjected to cyclic loading shall have to be examined for fatigue effects. Relevant clauses concerning fatigue effect as incorporated in appropriate codes shall be referred to in this regard.

604. DETERMINATION OF COMPOSITE SECTION

604.1. **Effective Flange Width**

604.1.1. **For the purpose of design, the effective width of compression flange of "T-Beams" (or interior beams) and "Edge Beams" (or exterior beams) for different type of construction may** be calculated as per Clause 305.12.2. of IRC : 21. For calculation of deflection, however, the full width of slab may be considered as effective.

604.2. **Equivalent Section**

604.2.1. **For prefabricated units in reinforced concrete or prestressed concrete, consideration shall be given to the different moduli of elasticity of the concrete of pre-cast unit and the cast-in-situ unit. The cast-in-situ slab shall be transformed into the corresponding equivalent area of the pre-cast girder.**

604.2.2. To determine the equivalent area in service condition, the effective flange width as determined under Clause 604.1. shall be divided by the modular ratio.

$$m = \frac{E_c(p)}{E_c(s)}$$

Where, $E_c(p)$ = Modulus of elasticity of pre-cast girder at 28 days

$E_c(s)$ = Modulus of elasticity of in-situ concrete slab at 28 days

604.2.3. In determining the sectional properties of the composite section such as equivalent area, equivalent moment of inertia, etc., at the age under consideration, the effective gross area of the in-situ slab shall be transformed into the corresponding equivalent area of the prefabricated unit.

This shall be done by dividing the effective width of the in-situ slab by the modular ratio m_i which is given by

$$m_i = \frac{E_{cj}(p)}{E_{ci}(s)}$$

Where $E_{cj}(p)$ = Modulus of elasticity of pre-cast concrete at j days ($j \ngtr 28$), and

$E_{ci}(s)$ = Modulus of elasticity of cast-in-situ concrete at i days ($i \ngtr 28$).

604.2.4. For composite section with prefabricated units in steel, the equivalent area of concrete slab shall be determined by dividing the effective width of concrete slab by the modular ratios, m as follows :

(i) For permanent loads, $m = \dfrac{E_s}{K_c\, E_c\,(s)}$

(ii) For transient loads, $m = \dfrac{E_s}{E_c\,(s)}$

Where, E_s = Modulus of elasticity of steel of girder

$E_c(s)$ = Modulus of elasticity of cast-in-situ concrete at 28 days

K_c = Creep factor which may be taken as 0.5

The equivalent area of the concrete slab at any age, however, shall be determined by dividing the effective width of the concrete slab by the modular ratio.

$$m = \frac{E_s}{E_{ci}\,(s)}$$

Where, E_s = Modulus of elasticity of steel, and

$E_{ci}(s)$ = Modulus of elasticity of cast-in-situ concrete at i days ($i \ngtr 28$ days)

605. DIFFERENTIAL SHRINKAGE

605.1. **Differential Shrinkage in Composite Section with in-situ Slab Over Pre-cast Girder**

The effect of differential shrinkage between the prefabricated girder and cast-in-situ slab shall be duly considered in the design. However, for common cases, specific assessment of the stress resultants may not be deemed necessary. Clause 605.2. may be referred to in this connection for provision of reinforcement.

For cases where stress resultants due to differential shrinkage are likely to be significant, specific assessment of such stress resultants may be made and accounted for in detailing and provision of reinforcement required. For method of assessment, special literature on the subject may be referred to.

605.2. To cater for differential shrinkage stresses minimum tensile reinforcement in the longitudinal direction of the cast-in-situ slab shall not be less than 0.2 per cent of the cross sectional area (for all grades of steel).

606. DESIGN ASSUMPTIONS FOR COMPOSITE STRUCTURES

606.1. **For the purpose of design, prefabricated unit shall be considered as sustaining its self-load. Where the load of the form work and the in-situ concrete is carried directly by the prefabricated unit without adequate props, it shall also be accounted for.**

606.2. Composite action shall be considered to be effective only after the in-situ concrete has attained at least 75 per cent of its cube strength (characteristic strength). In case the composite girder is required to carry imposed loads in addition to self weight of girder prior to 28 days, the sectional properties of the girder may be determined as per Clause 604.2.3. or 604.2.4.

606.3. The composite sections shall be so proportioned that the neutral axis of the composite section is generally located below the in-situ concrete slab.

In case of steel or concrete prefabricated unit, if the neutral axis is located inside the in-situ concrete slab, the portion of the slab below the neutral axis shall not be considered effective for computing moment of inertia or resisting moment except for deflection calculations.

607. METHOD OF DESIGN

607.1. Design Load

The design loads shall be in accordance with that stipulated in the IRC : 6 Bridge Code Section II. The structures and all elements shall be designed to sustain safely the various loads and forces.

607.2. Analysis of Structures

The structures shall be analysed by following analytical method in accordance with laws of mechanics, recognised method of design and sound engineering practice. Structures shall be analysed to find out moment, shear, axial force, etc., by using elastic method with design load. The stiffness of members shall be determined on the basis of an equivalent section as defined in Clause 604.2.

607.3. Permissible Stresses of Materials

Permissible stresses of different grades of concrete and steel

shall be in accordance with the stipulations made to this effect in IRC:21 and those of structural steel shall be in accordance with the stipulations made to this effect in IRC:24.

Increase in permissible stresses for various load combinations shall be governed by the corresponding provisions made in IRC:21 and IRC:24.

607.4. Deflection

The deflection shall be limited to the relevant provisions of IRC:21 and IRC:24.

608. DESIGN OF COMPOSITE SECTION WITH STEEL BEAM AND R.C. SLAB

608.1. Design of Section for Flexure

608.1.1. Steel section

608.1.1.1. Where the unpropped steel section carries the weight of the concrete slab or any other load during construction, it shall be designed in accordance with IRC:24 and the resulting stresses added to those induced later in composite section.

608.1.1.2. For computation of relief in stresses due to removal of such temporary loads added during construction, appropriate sectional properties of the composite section shall be considered depending upon the time of removal.

608.1.1.3. The total calculated stresses in the steel section shall not exceed the appropriate allowable stresses.

608.1.2. Composite section

608.1.2.1. The stresses in the composite section (steel beam, concrete slab and longitudinal reinforcement) shall be calculated in accordance with elastic theory, ignoring concrete in tension and assuming full interaction between the steel beam and concrete slab through properly designed shear connectors.

608.1.2.2. The design of composite section shall be done in the same manner as that of Tee-beam. For this purpose, the properties of the composite section may be computed in accordance with Clause 604.

608.1.3. **Deck slab**

608.1.3.1. The deck slab shall be designed in accordance with elastic theory to resist :

(a) The effects of local bending due to loads acting directly on it with due consideration of the dispersion of load in accordance with provisions of IRC:21, and

(b) The effects of loads acting on the composite member or members of which it forms a part, in accordance with the requirements of Clause 608.1.2.1.

608.2. **Design of Section for Shear**

608.2.1. **Vertical shear**

608.2.1.1. The vertical shear shall be assumed to be resisted by the steel section alone.

608.2.2. **Longitudinal shear**

608.2.2.1. Longitudinal shear per unit length of the composite beam shall be calculated on the basis of the elastic theory assuming an uncracked slab of effective width as in Clause 604.1.

608.2.2.2. The longitudinal shear V_L at the interface of the prefabricated unit and in-situ unit shall be determined from the following equation :

$$V_L = \frac{V.A_c.Y}{I}$$

Where

- V_L = The longitudinal shear per unit length at the interface in the section under consideration.
- V = The vertical shear due to dead load and live load including impact acting on the composite section.
- A_c = The transformed compressive area of concrete above the neutral axis of the composite section.
- Y = The distance from the neutral axis of the composite section to the centroid of the area A under consideration.
- I = The moment of inertia of the whole transformed composite section

Note: 1 When the deck slab is cast with the girders supported by adequate temporary props, the shear V is the total external shear due to dead load of the deck including the girder and the live load

with impact. When the deck slab is cast with girders unpropped, the shear V will be the total external shear due to dead load added after the concrete has attained a strength compatible to the composite action assumed and the live load with impact. In the latter case, when the slab is supported independent of the girder system, the shear V will be the total external shear including the self weight of the slab.

Note : 2 The effective flange width of the concrete slab shall be taken as constant over the entire span.

608.2.2.3. Adequate shear connectors and adequate transverse shear reinforcement as per Clause 611 shall be provided to resist the longitudinal shear. Proper detailing precautions as mentioned in Clause 612 shall also be taken to supplement the design checks.

609. DESIGN OF COMPOSITE SECTION WITH R.C. GIRDER AND R.C. SLAB

609.1. **Design of Section for Flexure**

609.1.1. **Precast section**

609.1.1.1. When the unpropped precast section carries the weight of the concrete slab or any other load during construction, it shall be designed in accordance with IRC:21 and the resulting stresses added to those induced later in composite section.

609.1.1.2. For computation of relief in stresses due to removal of such temporary loads added during construction, appropriate sectional properties of the composite section shall be considered depending upon the time of removal.

609.1.1.3. The total calculated stresses in the precast section shall not exceed the appropriate allowable stresses.

609.1.2. **Composite section**

609.1.2.1. The stresses in the composite section (precast beam, concrete slab and longitudinal reinforcement) shall be calculated in accordance with elastic theory, ignoring concrete in tension and assuming full interaction between precast beam and concrete slab through properly designed shear connectors.

609.1.2.2. The design of a precast R.C. Tee-beam and cast-in-situ concrete slab, when acting together as a composite section, shall be done in the same manner as that of a monolithic R.C. Tee-beam. For this purpose, the properties of the composite section may be computed in accordance with Clause 604.

609.1.3. **Deck slab**

609.1.3.1. The deck slab shall be designed in accordance with elastic theory to resist :

(a) The effects of local bending due to loads acting directly on it with due consideration of the dispersion of load in accordance with the provisions of IRC : 21, and

(b) The effects of loads acting on the composite member or members of which it forms a part, in accordance with the requirement of Clause 609.1.2.1.

609.2. **Design of Section for Shear**

609.2.1. **Vertical shear**

609.2.1.1. The vertical shear shall be assumed to be taken up entirely by the rib of the transformed flanged beam. The effects of the vertical shear are then calculated by the conventional methods for reinforced concrete Tee-beams as envisaged in IRC : 21 and IRC : 18.

609.2.2. **Longitudinal shear**

609.2.2.1. Longitudinal shear per unit length of the composite beam shall be calculated on the basis of elastic theory assuming an uncracked slab of effective width as in Clause 604.1.

609.2.2.2. The longitudinal shear, V_L at the interface of the prefabricated unit and the in-situ unit shall be determined as stated in Clause 608.2.2.2.

609.2.2.3. Adequate reinforcement in the form of shear connectors as well as adequate transverse shear reinforcement as per Clause 611 shall be provided to resist the longitudinal shear. Proper detailing precautions as mentioned in Clause 612 shall also be taken to supplement the design checks.

610. DESIGN OF COMPOSITE SECTION WITH P.S.C. GIRDER AND R.C. OR P.S.C. SLAB

610.1. **Design of Section for Flexure**

Design of section shall be as indicated in IRC : 18.

610.2. **Design of Section for Shear**

For design of section for shear the principles and procedures as laid down in the IRC : 18 shall be followed.

610.3. **Design of Section for Torsion**

To safeguard against failure of concrete by compression, necessary check as envisaged in the IRC : 18 shall be exercised.

Reinforcement for resisting shear resulting from torsional moments shall be provided in addition to reinforcement required to resist vertical shear in the manner specified in the IRC : 18.

611. SPECIAL CONSIDERATIONS FOR STRUCTURAL ELEMENTS

611.1. **General**

Special considerations for structural elements in the design of composite section include the following :

(i) The longitudinal shear at the interface of the prefabricated unit and the in-situ unit shall be transferred from one unit to the other by means of special elements known as "Shear Connectors" to ensure interaction between the two units.

(ii) The longitudinal shear so transferred from the prefabricated unit to the in-situ unit shall be resisted by the provision of adequate transverse shear reinforcement in the in-situ unit.

611.2. **Design Requirements of Shear Connectors**

611.2.1. The shear connectors shall be designed to ensure integral action between the prefabricated unit and the in-situ concrete unit so that the two units act as composite structure. The shear connectors shall satisfy the following two basic requirements:

(i) Shear along the contact surface is transferred without slip, and

(ii) Separation of the prefabricated unit and the in-situ slab in the perpendicular direction is prevented.

611.2.2. The shear connectors shall be of the type which permit a thorough compaction of concrete in order to ensure perfect contact of their entire surfaces with concrete.

611.2.3. The shear connectors which are to be welded to the steel girder shall be of weldable steel. The capacity of the welds at permissible stress shall not be less than the shear resistance of the connectors. Welding shall be in accordance with the requirements of the relevant Indian Standards.

611.3. **Types of Shear Connectors**

611.3.1. Shear connectors shall consist of any or a combination of the following types.

611.3.2. **Shear connectors for deck with R.C. slab over prefabricated steel girder**

(i) Rigid connectors consist of short length bars, stiffened angles of Tees or channels welded on the flange of the steel fabricated units (see Fig. 1.). With a view to preventing the separation of the in-situ slab from the prefabricated unit in the direction perpendicular to the contact surface, some mechanical device must be provided along with these connectors (see Fig. 2).

(ii) Flexible connectors consist of studs or angles or channels or tees welded on the prefabricated steel unit (see Fig. 3). As the head of the stud or the horizontal leg of the channel prevents the vertical separation of the two units, no special device is required for these connectors for preventing the vertical separation. But in case of angle or tee connectors, some special device as shown in, Fig. 2 is required for preventing the vertical separation.

611.3.3. **Shear connectors for deck with R.C. or P.S.C. slab :** To resist the longitudinal shear at the interface of in-situ slab and prefabricated girder, bond or anchorage type shear connectors consisting of mild steel or HYSD bars shall be provided (see Fig. 4). The top of the flange shall be made rough for effective bonding. Sufficient number of ties shall be provided to prevent separation of the two units in the direction perpendicular to the contact surface.

611.4. **Design of Shear Connectors**

611.4.1. **Shear connectors for deck with R.C. slab and prefabricated steel girder :** Shear connectors may be either of M.S. or H.T.S. according to the material specifications of steel girders.

611.4.1.1. **High tensile shear connector from fatigue strength consideration**

611.4.1.1.1. To cater for the effect of repeated loading, the shear connectors shall be designed for fatigue. The design method of shear connectors from fatigue strength considerations shall be as given in Clauses 611.4.1.1.2. and 611.4.1.1.3.

611.4.1.1.2. The range of longitudinal shear per unit length at the interface is given by the following expression :

$$V_r = \frac{V_R \cdot A_c \cdot \bar{Y}}{I}$$

Where, V_R = Range of vertical shear i.e. the difference between the maximum and minimum shear envelope due to live load and impact.

V_r = Range of horizontal shear per unit length

A_c = Area of transformed section on one side of the interface

$\bar{Y}$ = The distance of the centroid of the area under consideration from the neutral axis of the composite section

I = Moment of inertia of the composite section

611.4.1.1.3. Based on the fatigue strengths of connector and the range of horizontal shear per unit length, the spacing of shear connectors is given by :

$$P = \frac{\Sigma Q_r}{V_r}$$

Where,

P = Spacing of connectors

Q_r = The allowable range of horizontal shear for each connector evaluated from equations given hereinafter (ΣQ_r is the resistance of all connectors at one transverse cross-section of the girder).

V_r = The range of horizontal shear per unit length of beam at the interface and may be determined from equation given in 611.4.1.1.2.

The fatigue strength of stud connectors, is given by the following expression :

$$Q_r = \alpha A \cdot 10^{-3} \text{ (for ratio of } \frac{h}{d} > 4)$$

Where,

Q_r = Allowable range of horizontal shear per stud connector (kN)

A = Area of stud (mm²)

d = Diameter of stud connector (mm)

h = Height of stud (mm)

α = 55 MPa for 20 × 10^5 cycles

For channel angle or tee shear connectors, **the fatigue strength** shall be calculated from the following formula :

$$Q_r = \beta L \cdot 10^{-3}$$

Where,

Q_r = Allowable range of horizontal shear (kN) per connector

L = Length of connector (mm), measured transverse to the beam axis

β = 370 N/mm for 20 × 10^5 cycles

611.4.1.2. **High tensile shear connector from the consideration of ultimate flexural strength of the composite section :** In addition to providing adequate fatigue strength, sufficient number of connectors should be provided so that the flexural strength of the composite member can be reached. Usually the requirements will be satisfied in most composite beams because fatigue considerations are usually critical except in cases of shored construction. The flexural strength of composite members can be achieved if sufficient connectors are provided to resist the maximum horizontal force in the slab and the connectors could be spaced uniformly.

611.4.1.2.1. The maximum horizontal force is given by :

$H_1 = A_{st} . f_y . 10^{-3}$

$H_2 = 0.85 f_c . b_f . h_f . 10^{-3}$

Where,

H_1, H_2 = Horizontal force (kN)

A_{st} = Area of tensile steel (mm^2) in longitudinal direction

f_y = Yield stress of steel (MPa)

f_c = Cube (characteristic) strength of concrete (MPa)

b_f = Effective width of flange of in-situ slab

h_f = Thickness of in-situ slab

The ultimate flexural strength of any composite section will be governed by either of the aforesaid equations depending upon whether the steel section or the concrete section is large. Therefore, the maximum possible compressive force in the slab would be the smaller of H_1 or H_2 and sufficient connectors should be provided to resist the horizontal force H_1 or H_2 whichever is the lesser after calculating the ultimate strength of shear connectors from Clause 611.4.1.2.2.

611.4.1.2.2. The ultimate strengths of shear connectors are given by the following expressions :

(i) H.T. Stud connectors

$$Qu = 0.5A\sqrt{f_{ck} E_c} \times 10^{-3}$$

(ii) H.T. Channel/Angle Tee connectors

$$Qu = 45 (h + 0.5 h_1) L\sqrt{f_{ck}} \times 10^{-3}$$

Where,

Qu = Ultimate shear esisting capacity of one shear connector (kN)

A = Area of stud connector (mm^2)

h = Average thickness of channel/Angle/Tee flange (mm)

h_1 = Thickness of channel/Angle/Tee web (mm)

f_{ck} = Cube (characteristic) strength of concrete (MPa)

E_c = Modulus of Elasticity of concrete (MPa)

To ensure the development of the ultimate flexural strength of composite beams, a larger margin of safety against connector failure should be provided than is provided for the beam. This margin should be achieved by providing a load reduction factor of $\lambda = 0.85$ to the ultimate shear strength of connectors. The number of connectors required from fatigue considerations will usually exceed the requirements for flexural strength. The minimum number of shear connectors required between the points of maximum moment and end supports shall be determined by :

$$n = \frac{H}{\lambda Qu}$$

Where,

n = Number of shear connectors between points of maximum moment and end supports

H = Smaller value of H_1 and H_2 (as per Cl. 611.4.1.2.4.)

λ = Load reduction factor = 0.85

Qu = Ultimate shear resisting capacity of one connector

If the number of shear connectors given by above equation exceeds the number provided by the spacing from the formula mentioned in 611.4.1.1.3., additional connectors should be added to ensure that the ultimate strength of the composite section is achieved.

611.4.1.3. **Mild steel shear connector**

611.4.1.3.1. For mild steel shear connectors, the safe shear for each shear connector shall be calculated as below :

(a) For welded stud connector of steel with minimum ultimate strength of 460 MPa, and yield point of 350 MPa and elongation of 20 per cent

(i) For a ratio of h/d less than 4.2

$Q = 1.49\, h\, d\, \sqrt{fck}$

(ii) For a ratio of h/d equal to or greater than 4.2

$Q = 6.08\, d^2\, \sqrt{fck}$

Where,

Q = The safe shear resistance in Newton of one shear connector

h = Height of stud in mm

d = Dia of stud in mm

(b) For channnel/ Angle Tee connector made of mild steel with minimum ultimate strength of 420 to 500 MPa yield point of 230 MPa and elongation 21 per cent

$Q = 3.32 (h + 0.5t) L \sqrt{f_{ck}}$

Where,

Q = The safe shear resistance in Newton of one shear connector

h = The maximum thickness of flange measured at the face of the web in mm

t = Thickness of the web of shear connector in mm

L = Length of the shear connector in mm

611.4.1.3.2. The spacing of shear connectors shall be determined from the formula

$$P = \frac{\Sigma Q}{V_L}$$

Where,

V_L = The longitudinal shear per unit length as stated in clause 611.4.1.3.3.

Q = Safe shear resistance of each Shear Connector as stated in clause 611.4.1.3.1. above (ΣQ is the total shear resistance of all connectors at one transverse cross-section of the girder)

611.4.1.3.3. The longitudinal shear per unit length at the interface of the prefabricated unit and in-situ unit shall be evaluated from the expression given below:

$$V_L = \frac{V.A_e.\bar{Y}}{I}$$

Where

V = Vertical shear due to dead load placed after composite section is effective and working live load with impact

V_L = Longitudinal shear per unit length

A_e = Area of transformed section on one side of interface

$\bar{Y}$ = Distance of the centroid of the area under consideration from the neutral axis of the composite section

M = Moment of inertia of the composite section

611.4.2. **Shear connectors for deck with R.C. or P.S.C. slab and R.C. or P.S.C. prefabricated girder**

611.4.2.1. The load factor for design of shear connectors under ultimate load shall be 1.5 for dead load and 2.5 for live load. The dead load to be taken for calculating the ultimate horizontal shear shall be the dead load operating after composite action is effective.

611.4.2.2. The ultimate longitudinal shear V_L per unit length at the interface shall be evaluated from the expression as given in

Clause 611.4.1.3.3. by calculating ultimate vertical shear with the above mentioned load factors.

611.4.2.3. The ultimate shear resistance of an anchorage connector is given by the following equation :

$$Q_u = A_s f_u \times 10^{-3}$$

Where

Q_u = The ultimate shear resisting capacity of the anchorage connector in kN

A_s = The cross-sectional area of the anchorage connector mm²

f_u = The ultimate tensile strength of steel of the anchor connector in MPa. The anchor bars shall be placed either vertical or at an angle of 45°

611.4.2.4. The spacing of the shear connector shall be calculated from the formula

$$P = \frac{\Sigma Q_u}{V_L}$$

Where,

P = Spacing of shear connectors

Q_u = Ultimate shear resistance of each connector

V_L = Ultimate longitudinal shear per unit length

611.4.2.5. Notwithstanding the fact that full shear resistance is provided by anchor bars, the ultimate bond stress at the interface shall not exceed 2.1 MPa. The interface shall always be made rough for effective bonding.

611.5. **Design of Transverse Reinforcement**

611.5.1. The longitudinal shear force V_L per unit length transferred from prefabricated girder to in-situ slab through any shear plane shall not exceed either of the following and the reinforcement shall be calculated accordingly.

(i) $0.4 L_s \sqrt{f_{ck}}$ or

(ii) $0.7\, A_s\, \sigma_y + 0.08\, L_s \sqrt{f_{ck}}$

Where,

L_s = The length of shear plane under consideration in mm. Typical shear planes are shown in, Fig. 5

f_{ck} = Cube (characteristic) strength of concrete (MPa) but not more than 45 MPa,

A_s = The sum of the cross-sectional areas per unit length of beam of all reinforcing bars intersected by the shear plane (mm²/mm)

σ_y = The yield stress (MPa) of the reinforcing bars inter-sected by the shear plane but not more than 450 MPa.

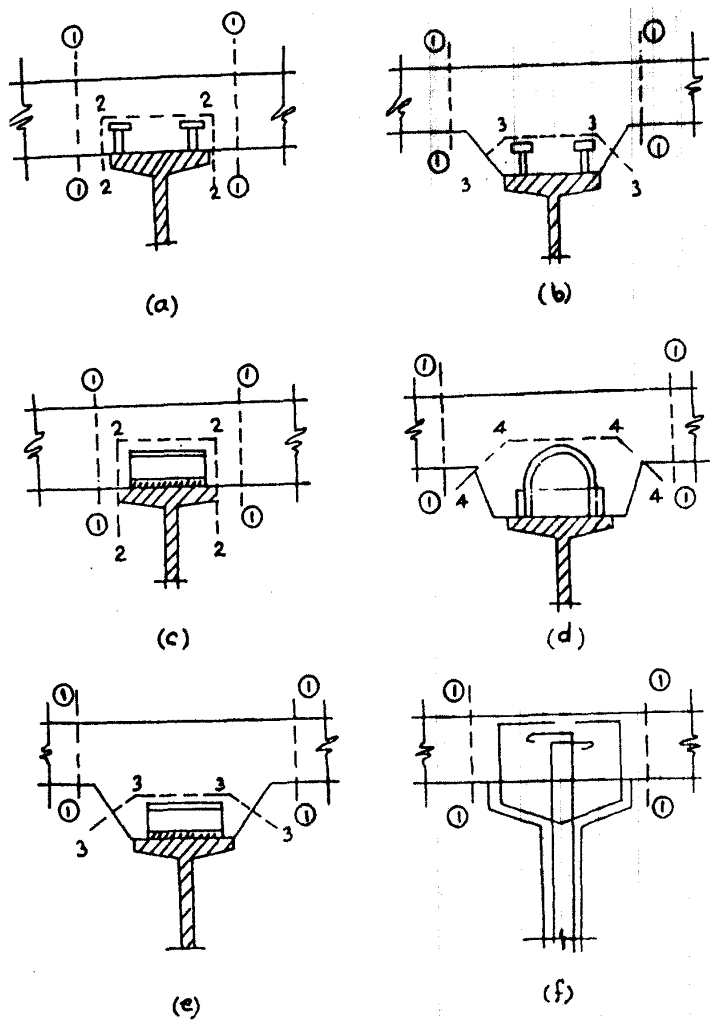


Fig. 5. Shear planes (typical)

611.5.2. **Transverse reinforcement**

611.5.2.1. Shear planes i.e. the surfaces on which longitudinal shear failure can take place in in-situ slab of composite beam in the process of transfer of longitudinal shear from the girder to the slab, are of four main types as shown in, Fig. 5. Sufficient transverse reinforcement shall be provided with a view to transfer longitudinal shear from the girder to the effective width of the slab. The area of transverse reinforcement per unit length of beam will be the sum total of all the reinforcement (A_t, A_h or A_b as shown in, Fig. 6) which are intersected by the shear plane and are fully anchored on both sides of the shear plane considered.

611.5.2.2. The total transverse reinforcement, A_s per unit length of beam in case of shear plane 1—1 which crosses the whole thickness of the slab will be, the sum of (A_t+A_b) (see Fig. 6). Area of reinforcement A_t and A_b include those provided for flexure. The total transverse reinforcement across plane 2—2 in Fig. 6 (a) is $A_s = 2A_b$ and that across plane 3—3 in Fig. 6 (b) is $A_s = 2A_h$ as these planes do not cross the full thickness of the slab. In case of shear plane 4—4, the total transverse reinforcement, $A_s = 2\ (A_b+A_h)$ Fig.6 (c). Some of the above transverse reinforcements may be curtailed provided they satisfy the provisions as laid down in Clause 611.5.2.4.

611.5.2.3. **Minimum transverse reinforcement**

(a) The cross-sectional area of minimum transverse reinforcement across any shear plane per unit length of beam calculated as per 611.5.1. shall not be less than that determined from the following equation :

$$A_s = \frac{0.8\ L_s}{\sigma_y}$$

Where,

A_s, L_s and σ_y are as defined in Clause 611.5.1.

(b) Across shear plane 1 – 1 where L_s h_c, not less than 50 per cent of A_t should be provided near the bottom of the slab as A_b. Where the length of shear plane around the connector [plane 2 – 2 in Fig. 6 (a)] is less than twice the thickness of the slab, transverse reinforcement in addition to that required for flexure should be provided in the bottom of the slab to prevent longitudinal splitting around the connectors.

The cross-sectional area of this additional reinforcement per unit length of beam must not be less than $0.8\ Sh_e\ /\ \sigma_y$.

This additional reinforcement need not be provided if the minimum compressive force per unit length of beam acting normal to and over the surface of shear plane is greater than $1.4\ Sh_e$.

(c) The cross-sectional area of transverse reinforcement, A_h in a haunch

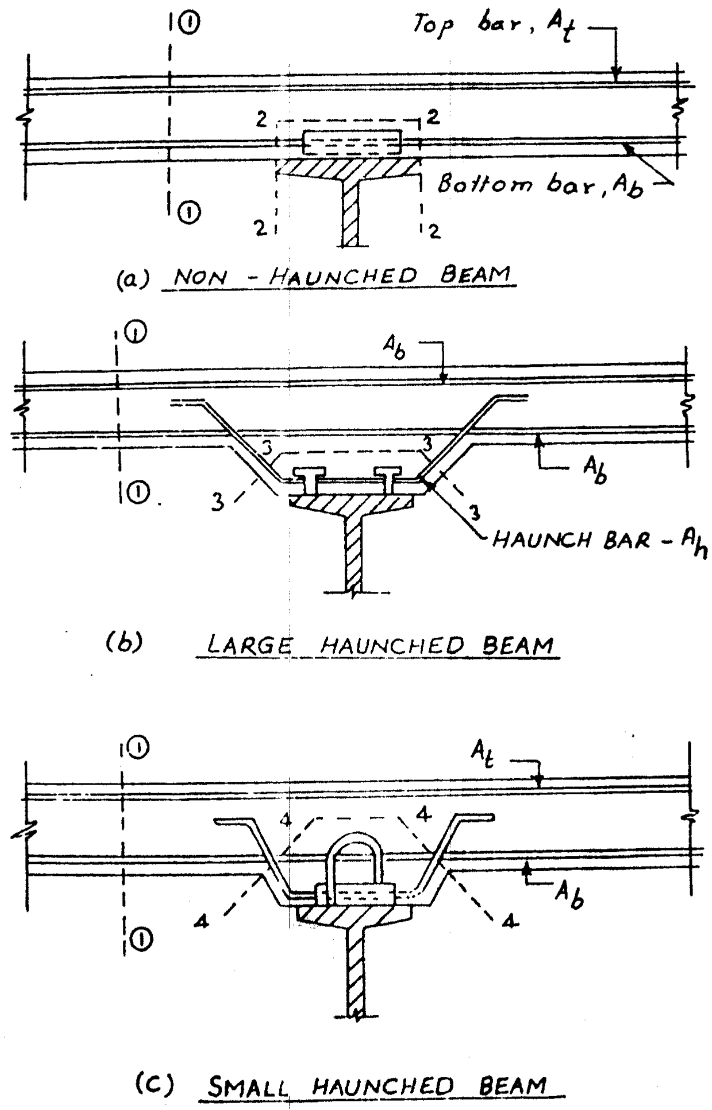


Fig. 6. Transverse reinforcement across shear planes

per unit length of the beam must not be less than $0.4\ SL_s / \sigma_y$ where L_s is the length of shear plane 3-3 or 4-4 in, Fig. 6.

611.5.2.4. **Curtailment of transverse reinforcement**

The transverse reinforcement provided to transfer longitudinal shear safely from the girder to the slab across shear plane 1-1 (Fig. 6) may be curtailed provided the requirements of 611.5.1. to 611.5.2.3. in respect of all shear planes within the effective width are satisfied. For this purpose, the shear force V_L for such plane may be assumed to vary linearly from the calculated maximum force on a plane adjacent to the shear connector to zero midway between the centre line of the beam and that of an adjacent beam or to zero at an adjacent free-edge.

612. DETAILING

612.1. Details of Haunches

612.1.1. **For concrete slab over steel girder**

612.1.1.1. The dimensions for haunches to be provided between top of steel girder and soffit of slab shall be as indicated in Fig. 7, the sides of haunches being located outside a line drawn at 45 degree from the outside edge of the base of the connectors.

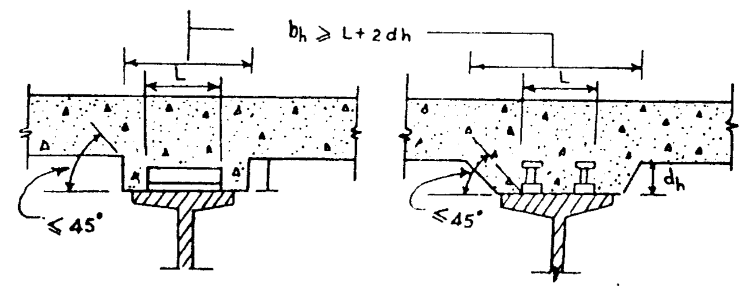


Fig. 7. Dimensions of haunches

612.1.2. **For concrete slab over concrete girder**

612.1.2.1. Where haunches are provided in the cast in-situ slab over precast concrete girder, the angle between the haunch line of in-situ slab and the face of the flange of the precast girder shall be 90 degrees and the top of the precast girder shall not be higher than the soffit line of the in-situ slab as shown in Fig. 8.

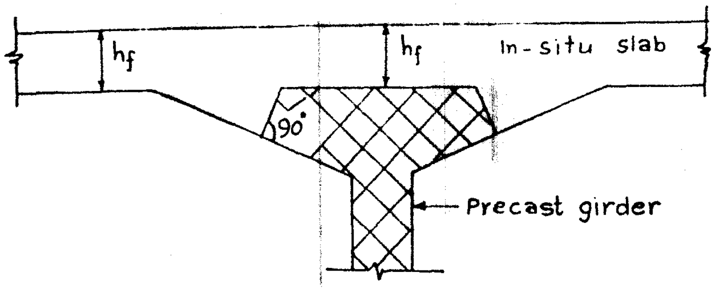


Fig. 8

612.2. **Dimensions and Other Details of Shear Connectors**

612.2.1. **For steel girder**

612.2.1.1. The diameter of stud connectors welded to the flange plate shall not exceed twice the plate thickness. The height of the stud connectors shall not be less than four times their diameter or 100 mm. The diameter of the head of the stud connectors shall not be less than one and half times the diameter of the stud. The leg length of the weld joining other types of connectors to the flange plate shall not exceed half the thickness of the flange plate. Channel and angle connectors shall have at least 6 mm fillet welds placed along the heel and toe of the channels/angles. The clear distance between the edge of a girder flange and the edge of the shear connectors shall not be less than 25 mm (see Fig. 9).

612.2.1.2. The surface of a shear connector which resists the separation between the two units (i.e. the underside of the stud or the inner face of the top flange of a channel or the inside of a hoop) shall extend not less than 40 mm clear above the bottom transverse reinforcement, nor less than 40 mm into the compression zone of the concrete flange. Where a concrete haunch is used between the steel flange and the soffit of the slab, the surface of the connector that resists separation between the two units shall be placed not less than 40 mm above the transverse reinforcement in the haunches (vide Fig. 10) provided this reinforcement is adequate to satisfy the requirement for transfer of longitudinal shear as per Clause 611.5. Where the shear connectors are placed adjacent to longitudinal edge of concrete slab, transverse reinforcement provided in accordance with Clause 611.5. shall be fully

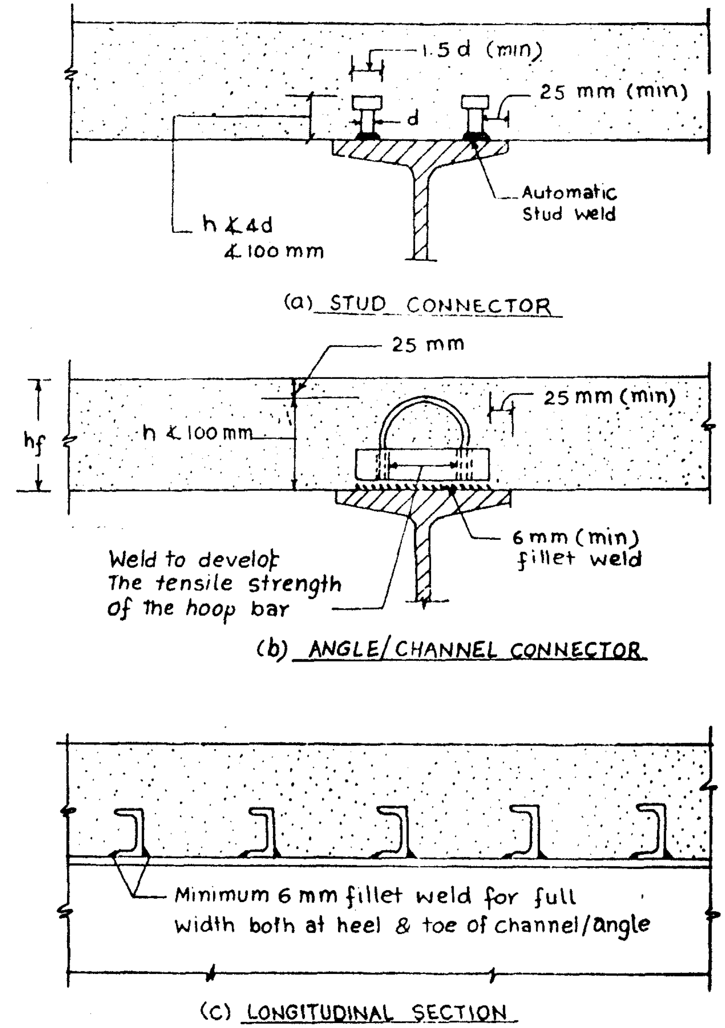


Fig. 9. Details of connectors on steel girders

anchored in the concrete between the edge of the slab and the adjacent row of connectors.

612.2.1.3. **The overall height of a connector including any hoop which is an integral part of the connector shall be at least 100 mm** with a clear cover of 25 mm.

612.2.2. **For concrete girder**

612.2.2.1. In order to resist the longitudinal shear and the separation force all the vertical reinforcement from web and flange of the precast element shall be extended into the cast-in-situ concrete slab and adequately anchored into it. Such reinforcement shall not be less than 0.15 per cent of the contact area or 130 sq. mm per metre of the span and shall be evenly distributed as far as possible over the contact area [see Fig. 11 (e) and 11 (f)].

612.3. **Cover to Shear Connectors**

612.3.1. **For steel girder**

612.3.1.1. The clear depth of concrete cover over the top of the shear connectors shall not be less than 25 mm. The horizontal clear concrete cover to any shear connector shall not be less than 50 mm (see Fig. 10).

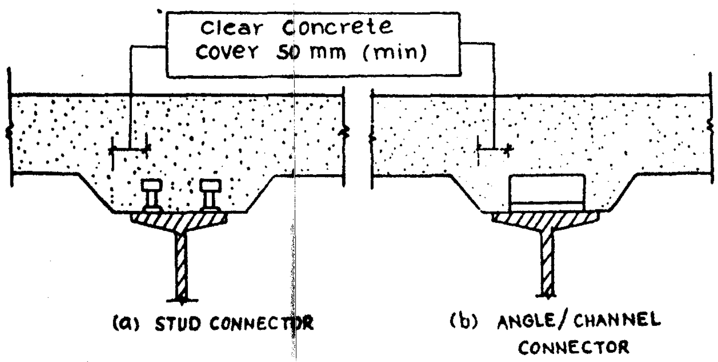


Fig. 10. Cover to connectors

612.3.2. **For concrete girder**

612.3.2.1. The cover to be provided for shear connectors shall conform to the relevant provisions regarding cover to reinforcements as per IRC: 18 and IRC: 21.

612.4. **Spacing of Shear Connectors**

612.4.1. **General**

612.4.1.1. The shear connectors shall be provided throughout the length of the beam and may be uniformly spaced between critical cross-sections.

612.4.2. **For steel girder**

612.4.2.1. The maximum spacing of shear connectors in the longitudinal direction shall be limited to 600 mm or three times the thickness of the slab or four times the height of the connector (including any hoop which is an integral part of the connector) whichever is the least.

612.4.2.2. The spacing of the stud connectors in any direction shall not be less than 75 mm.

612.4.3. **For concrete girder**

612.4.3.1. The spacing of the anchor or bond shear connectors shall not be less than 0.7 times the depth of the slab and shall not be greater than two times the depth of the slab.

612.5. **Detailing of Transverse Reinforcement**

612.5.1. The transverse reinforcement shall be placed at locations as shown in, Fig. 11.

612.5.2. The haunch bars shall be extended beyond the junction of bottom bars by a length equal to the anchorage length. Similarly, the anchorage connectors from the prefabricated concrete girders shall be extended beyond the junction of bottom bars by a length equal to the anchorage length.

613. **MATERIALS AND WORKMANSHIP**

613.1. **Material Specifications for Steel Members**

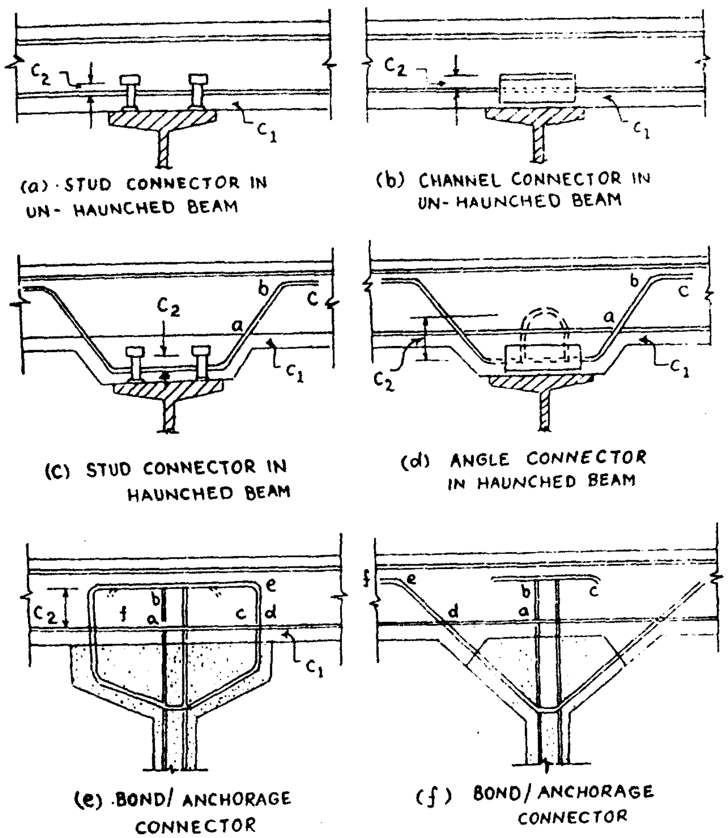
613.1.1. **Steel for structural members**

All structural steel shall comply with the Indian Standards with latest amendments as appropriate.

IS : 226—Structural Steel (Standard Quality)

IS : 2062—Structural Steel (Fusion Welding Quality)

IS : 961—Structural Steel (High Tensile)



NOTE
$C_1 \ngtr 50$ mm $\nless 25$ mm or dia of bar
$C_2 \nless 40$ mm
abc / def = ANCHORAGE LENGTH

Fig. 11. Arrangement of transvere reinforcement

613.1.2. **Steel for shear connectors**

613.1.2.1. Steel for shear connector of rigid and flexible type as detailed in clause 611.3.2. shall comply with the Indian Standards IS : 2062 (fusion welding of quality) or IS : 961 conforming to Fe 540 W-117 (with latest amendments).

613.1.2.2. Steel for anchorage type shear connectors as defined in Clause 611.3.3. shall comply with the following Indian Standards with latest amendments.

IS : 432—Mild Steel grade I or Medium Tensile Steel Bars

IS : 1786—High Strength Deformed Steel Bars and Wires

613.1.3. **Welding electrodes** : Welding electrodes shall comply with the requirements of the following Indian Standards with latest amendments.

IS : 814—Covered Electrodes for Metal Arc Welding of Mild Steel

IS : 1442—Covered Electrodes for Metal Arc welding of High Tensile Steel

613.2. **Material Specifications for Reinforced Concrete and Prestressed Concrete Members**

613.2.1. For reinforced concrete members, specifications for constituent materials, strength of concrete and reinforcement shall comply with the relevant provisions of IRC: 21.

613.2.2. For prestressed concrete members, specifications for constituent materials, strength of concrete, reinforcement and prestressing steel, etc., shall comply with the relevant provisions of IRC : 18.

613.3. **Workmanship for Steel Structures**

613.3.1. All structural steel members and components shall be fabricated, welded, erected and protected against corrosion as specified in IRC : 24.

613.3.2. The dimensions and other details of the rigid and flexible type shear connectors shall be as specified in Clause 612.2. The welding shall be done properly so that shear connectors do not fail due to defective welding. Adequate cover to the shear connectors as specified in Clause 612.3. shall be provided.

613.4. **Workmanship for Reinforced Concrete and Prestressed Concrete Members**

613.4.1. Workmanship in respect of concrete formwork, reinforcement, etc., for reinforced concrete and prestressed concrete members shall be in accordance with the relevant provisions of IRC : 18 and IRC : 21 respectively along with IRC : 87.

613.4.2. Prestressing work shall be done as specified in IRC : 18.

613.4.3. The anchorage type shear connectors shall be placed with adequate cover and requisite bond length. The bars to be used as shear connectors shall be free from kinks and undesirable bends

614. MAINTENANCE

614.1. General

With a view to ensure the desired life span of a bridge structure for which it is designed and to get the maximum service from the bridge, routine inspection and periodic maintenance shall be made as a rule. Any defect or damage in the structure noticed during routine inspection shall be rectified forthwith before it can cause any harmful effect on the bridge structure and thereby aggravate the situation.

614.2. Inspection of the Bridge Superstructure

614.2.1. The inspection of various components of the superstructure shall be made as recommended in Chapter 4 of the I.R.C. Special Publication:18—Manual for Highway Bridge Maintenance Inspection.

614.2.2. The inspection report shall consist of the "check items" as listed in the "Proforma for Inspection Report" of the IRC Special Publication 18.

614.3. Maintenance of Structural Members

614.3. The structural members shall be checked thoroughly as indicated in Chapter 4 of IRC Special Publication 18 and such corrective and remedial measures as found necessary for proper maintenance of the bridge superstructure shall be taken.

STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD BRIDGES

SECTION V

STEEL ROAD BRIDGES

(Second Revision)

THE INDIAN ROADS CONGRESS
2001

STANDARD SPECIFICATIONS
AND
CODE OF PRACTICE
FOR
ROAD BRIDGES

Section V –
Steel Road Bridges

(Second Revision)

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Section-V

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INTRODUCTION

The Standard Specifications and Code of Practice for Road Bridges, Section V-Steel Road Bridges, IRC:24-1967 was published by the Indian Roads Congress in 1967. Since this code was brought out more than three decades ago, its revision and updation, commensurate with the current data and incorporation of new concepts and materials has been a long felt need. The work of revision of this code was accordingly taken up by the Steel Bridges Committee (B-7) during its tenure from 1994. The draft was discussed at length during various meetings and finalised. After detailed discussion, the Committee constituted in 1997, consisting the following personnel considered and approved the draft in its meeting held on 21.7.98 for being placed before the Bridges Specifications & Standards Committee.

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R.P. Indoria	..	Member-Secretary

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Mahesh Tandon	A.K. Bhattacharya
	A.K. Basa

The draft approved by Steel Bridges Committee was discussed by the Bridges Specifications and Standards (BSS) Committee in their meeting on 24.10.98 and approved the same subject to certain modifications. The modified copy approved by Convenor, BSS Committee on 21.6.99 and later it was approved by the Executive Committee on 16.7.99. Finally, the draft was approved by the Council of the Indian Roads Congress at the 156th Council Meeting held at Jaipur on the 6th August, 1999 subject to modifications in light of the comments of Council members. The Convenor, Steel Bridges Committee sent the modified document on 2.6.2000 for forwarding the same to Convenor, BSS Committee for its approval. The Convenor, BSS Committee approved the document for printing on 31.10.2000.

The object of issuing the Standard Specifications and Code of Practice for Steel Road Bridges is to establish a common procedure for the design and construction of road bridges in India.

This revised publication is meant to serve as a guide to both the Design 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 structures designed and erected by them.

The design and construction of road bridges require extensive and thorough 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.

501. SCOPE

501.1. This code deals mainly with the design of superstructure of structural steelwork in road bridges. Wherever the provisions of this code do not cover the design requirements in certain particular cases, special literature may be referred to.

501.2. Provisions of this code generally apply to riveted, bolted and welded constructions using hot rolled steel sections only. Cold formed sections are not covered in this code.

501.3. IRC:22-1986 (Section VI) may be referred, wherever applicable in case of concrete work composite with steel.

502. LIMITATIONS

This code generally applies to normal steel bridges. For the following types of bridges for which there are special requirements for design, the provision of the present code will not form adequate basis. This code applies to such bridges to the extent where the special code covering these areas refers to the provisions of this present code. Further reference may be made to ***Appendix-A***.

(a) curved bridges
(b) cable-stayed bridges
(c) suspension bridges
(d) temporary bridges
(e) pedestrian bridges
(f) swing bridges
(g) bascule bridges
(h) box girder bridges
(i) prestressed steel bridges

503. REFERENCES

While preparing this code, practices prevailing in this country in the design and construction of steel bridges have been primarily kept in view. However, recommendations offered in the following publications have also been considered:

(a) IRS Code of Practice for the design of steel or wrought iron bridges carrying rail, road or pedestrian traffic incorporating latest addendum/corrigendum.

(b) BS:5400-Part 3:1982: Code of Practice for Design of Steel Bridges.

504. DEFINITIONS AND SYMBOLS

504.1. **Definitions:** For the purpose of this code, the following definitions shall apply:

504.1.1. **Buckling load:** The load at which a member or a structure as a whole collapses in service or buckles in a load test.

504.1.2. **Dead loads:** The self weights of all permanent construction and installations.

504.1.3. **Effective lateral restraint:** Restraint which produces sufficient resistance in a plane perpendicular to the plane of bending to restrain the compression flange of a loaded strut, beam or girder from buckling to either side at the point of application of the restraint.

504.1.4. **Elastic critical moment:** The elastic moment which will initiate yielding or cause buckling.

504.1.5. **Factor of safety:** The factor by which the yield stress of the material of a member is divided to arrive at the permissible stress in the material.

504.1.6. **Gauge:** The transverse spacing between parallel adjacent lines of fasteners.

504.1.7. **Imposed (live) load:** The load assumed to be produced by the intended use of occupancy including distributed, concentrated, impact, vibration and snow loads but excluding wind and earthquake loads.

504.1.8. **Load factor:** The numerical factor by which the working load is to be multiplied to obtain an appropriate design ultimate load.

504.1.9. **Main member:** A structural member which is primarily responsible for carrying and distributing the applied load.

504.1.10. **Pitch:** The centre to centre distance between individual fasteners in a line of fasteners.

504.1.11. **Secondary member:** Secondary member is that which is provided for stability and or restraining the main members from buckling or similar modes of failure.

504.1.12. **Welding terms:** Unless otherwise defined in this standard the welding terms used shall have the meaning given in IS:812-1957.

504.1.13. **Yield stress:** The minimum yield stress of the material in tension as specified in relevant Indian Standards.

504.1.14. **Warping stress:** Stresses in a box girder due to transverse bending of walls of the box and torsional and distortional warping.

504.2. **Symbols**

Symbols used in this code shall have the following meanings with respect to the structure or member or condition, unless otherwise defined elsewhere in this code :

λ	=	Slenderness ratio of the member, ratio of the effective length (l) to the appropriate radius of gyration (r)
λ_o	=	Characteristic slenderness ratio = $\sqrt{Py/Pe}$
θ	=	Ratio of the rotation at the hinge point to the relative elastic rotation of the far end of the beam segment containing plastic hinge
σ_{ac}	=	Maximum permissible compressive stress in an axially loaded strut not subjected to bending
σ_{at}	=	Maximum permissible tensile stress in an axially loaded tension member not subjected to bending
σ_{bc}	=	Maximum permissible compressive stress due to bending in a member not subjected to axial force.
σ_{bg}	=	Maximum permissible bearing stress on flat surface
σ_{bs}	=	Maximum permissible bending stress in slab base
σ_{bt}	=	Maximum permissible tensile stress due to bending in a member not subjected to axial force
σ_{cc}	=	Maximum permissible stress in concrete in compression
σ_{e}	=	Maximum permissible equivalent stress
σ_{p}	=	Maximum permissible bearing stress in a member
σ_{pf}	=	Maximum permissible bearing stress in a fastener
σ_{sc}	=	Maximum permissible stress in steel in compression
σ_{tf}	=	Maximum permissible stress in axial tension in fastener
$\sigma_{ac,cal}$	=	Calculated average axial compressive stresses
$\sigma_{at,cal}$	=	Calculated average stress in member due to an axial tensile force
$\sigma_{bc,cal}$	=	Calculated compressive stress in a member due to bending about a principal axis
$\sigma_{bt,cal}$	=	Calculated tensile stress in a member due to bending about both principal axes
$\sigma_{e,cal}$	=	Calculated equivalent stress
τ_{va}	=	Maximum permissible average shear stress in member
τ_{vm}	=	Maximum permissible shear stress in a member
τ_{vf}	=	Maximum permissible shear stress in fastener

$\tau_{v,cal}$	=	Calculated shear stress in a member
ω	=	Ratio of moment of inertia of the compression flange alone, to that of the sum of the moments of inertia of the flanges, each calculated about its own axis parallel to the y-y axis of the girder, at the point of maximum bending moment.
γ	=	Ratio of total area of both the flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment

Note: The subscripts x, y denote the x-x and y-y axis of the section respectively. For symmetrical sections, x-x denotes the major principal axis while y-y denotes the minor principal axis.

505. MATERIALS AND PROPERTIES

505.1. Steels

505.1.1. **Properties of steel:** The following properties shall be assumed for all grades of steel for design purposes:

Young's Modulus (Modulus of Elasticity)	= 2.11 x 10^5 Mpa
Shear Modulus	= 77 x 10^3 Mpa
Poisson's Ratio	= 0.30
Coefficient of Thermal Expansion	= 0.0000117/°c/unit length

505.1.2. **Structural steels:** Unless otherwise permitted herein, all structural steel shall, before fabrication comply with the requirements of the following Indian Standards, or their latest revisions as appropriate :-

IS:808-1989	Dimensions for hot rolled steel beam, column, channel and angle sections
IS:1161-1979	Steel tubes for structural purposes
IS:1239 (Pt 1)-1990	Mild steel tubes, tubulars and other wrought steel fittings:Part I Mild steel tubes
IS:1239 (Pt 2)-1992	Mild steel tubes, tubulars and other wrought steel fittings : Part 2 Mild steel tubulars and other wrought steel fittings

IS:1730-1989	Dimensions for steel plates, sheets, strips and flats for general engineering purposes
IS:1732-1989	Dimension for round and square steel bars for structural and general engineering purposes
IS:1852-1973	Rolling and cutting tolerances for hot rolled steel products
IS:2062-1992	Steel for general structural purposes
IS:4923-1985	Hollow steel sections for structural use
IS:8500-1992	Structural steel microalloyed (medium and high strength qualities)
IS:11587-1986	Structural weather resistant steels

The use of structural steel not covered by the above standards may be permitted with the specific approval of the authority.

505.1.3. **Other steels:** Except where permitted with the specific approval of the authority, steels for machined parts and for uses in other than structural members or elements shall comply with the following or relevant Indian Standards.

IS: 1875-1992	Carbon steel billets, blooms, slabs and bars for forgings
IS: 6911-1992	Stainless steel plate, sheet and strip

505.2. **Castings and Forgings**

Steel casting and forgings shall comply with the requirements of the following Indian Standards as appropriate :

IS:1030-1989	Carbon steel castings for general engineering purposes
IS:1875-1992	Carbon steel billets, blooms, slabs & bars for forgings
IS:2004-1991	Carbon steel forgings for general engineering purposes

IS:2644-1986	High tensile steel castings
IS:4367-1991	Alloy steel forgings for general industrial use

505.3. **Fasteners**

Bolts, nuts, washers and rivets shall comply with the following or relevant Indian Standards, as appropriate:

IS:1148-1982	Hot rolled rivet bars (upto 40mm dia) for structural purposes
IS:1149-1982	High tensile steel rivet bars for structural purposes
IS:1363-1992 (Pt 1 to Pt 3)	Hexagon head bolts, screws and nuts of product grade C (size range M5 to M64)
IS:1364-1992 (Pt 1 to Pt 3)	Hexagon head bolts,screw and nuts products grade A & B (size range M1.6 to M64)
IS:1367-1979~94 (Pt 1 to Pt 18)	Technical supply conditions for threaded steel fasteners
IS:1929-1982	Hot forged steel rivets for hot closing (12 to 36 mm diameter)
IS:2155-1982	Cold forged solid steel rivets for hot closing (6 to 16 mm diameter)
IS:3640-1982	Hexagon fit bolts
IS:3757-1985	High strength structural bolts
IS:4000-1992	High strength bolts in steel structures-code of practice
IS:5369-1975	General requirements for plain washers and lock washers
IS:5370-1969	Plain washers with outside dia ~ 3 x inside dia.
IS:5372-1975	Taper washers for channels (ISMC)
IS:5374-1975	Taper washer for I beams (ISMB)
IS:5624-1970	Foundation bolts
IS:6610-1972	Heavy washers for steel structures

IS:6623-1985	High strength structural nuts
IS:6649-1985	Hardened and tempered washers for high strength structural bolts and nuts
IS:7002-1991	Prevailing torque type steel hexagon nuts

505.4. **Welding Consumables**

Welding consumables shall comply with the following Indian Standards, as appropriate :

IS:814-1991	Covered electrodes for manual metal are welding of carbon and carbon manganese steel
IS:1395-1982	Low and medium alloy, steel covered electrodes for manual metal arc welding
IS:3613-1974	Acceptance tests for wire flux combination for submerged arc welding
IS:6419-1971	Welding rods and bare electrodes for gas shielded arc welding of structural steel
IS:6560-1972	Molybdenum and chromium - molybdenum low alloy steel welding rods and bare electrodes for gas shielded arc welding
IS:7280-1974	Bare wire electrodes for submerged arc welding of structural steel

505.5 **Welding**

IS:812-1957	Glossary of terms relating to welding and cutting of metal
IS:816-1969	Code of practice for use of metal arc welding for general construction in mild steel
IS:822-1970	Code of procedure for inspection of welds
IS:1024-1979	Code of practice for use of welding in bridges and structures subject to dynamic loading

IS:1182-1983	Recommended practice for radiographic examination of fusion welded butt joints in steel plates
IS:4853-1982	Recommended practice for radiographic inspection of fusion welded butt joints in steel pipes
IS:5334-1981	Code of practice for magnetic particle flaw detection of welds
IS:7307(Pt. 1) -1974	Approval tests for welding procedures: Part-1 fusion welding of steel
IS:7310(Pt. 1) -1974	Approval tests for welders working to approved welding procedures:Part-1 fusion welding of steel
IS:7318(Pt. 1) -1974	Approval tests for welders when welding procedure is not required : Part-1 fusion welding of steel
IS:9595-1980	Recommendations for metal arc welding of carbon and carbon manganese steels

505.6. **Wire Ropes and Cables**

These shall conform to the following or relevant Indian Standards except where use of other types is specifically permitted by the authority.

IS:1785 (Pt. 1) -1983	Specification for plain hard-drawn steel wire for prestressed concrete : Part-1 Cold drawn stress relieved wire
IS:1785 (Pt. 2) -1983	Specification for plain hard-drawn steel wire for prestressed concrete : Part-2 As- drawn wire
IS:2266-1989	Steel wire ropes for general engineering purposes
IS:2315-1978	Thimbles for wire ropes
IS:9282-1979	Wire ropes and strands for suspension bridges

506. LOADS AND STRESSES

506.1. **Combinations**

506.1.1. **Main effects:** For the purpose of computing stresses, the classifications (column 1) and combinations (column 2) as given in Table 6.1 below will be followed. For legend of symbols under combination (column 2) refer to Clause 202.1 of IRC:6-2000 (Fourth Revision).

506.1.2. **Other effects**

506.1.2.1. Secondary Effects (F_s) shall include, where applicable, the effects due to creep and shrinkage of concrete for composite deck and warping for box girder sections.

506.1.2.2. Erection effects shall include the loads and forces arising out of construction equipment and the effects of wind/seismic.

506.2. **Permissible Increase in Stress**

506.2.1. **Increase:** The permissible increase (per cent) in stress in the various members covered by this code due to combination stated in Clause 506.1 shall be as given under Increase (column 3) of Table 6.1.

Table 6.1. Permissible Increase in Stress

Classification (1)	Combination (2)	Increase (3)
I	$G + Q$ or $G_s + Q_{im} + F_{wc} + F_f + G_b + F_{cf} + F_{cp} + G_c$	Nil
II	$(I) + F_s + F_d + F_{te}$	15 per cent
III	$(II) + W + F_{wp}$	25 per cent
IV	$(II) + F_{eq} + F_{wp}$	40 per cent
V	$(II) + F_{im} + W$	25 per cent
VI	$G + F_{wc} + G_b + F_{cp} + F_{er} + F_f + W + G_c$	30 per cent
VII	$(VI) + F_{eq} - W$	40 per cent

506.2.2. **Limitation:** The above permissible increase in stress, shall, however, be limited to 90 per cent of yield stress.

506.3. **Worst Effect**

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

506.4. **Working Stresses**

506.4.1 **Basic permissible stresses:** The basic permissible stresses for steel work are given in Table 6.2.

Table 6.2. Basic Permissible Stresses

1.	Axial tension on net area	$0.6 f_y$
2.	Axial compression on effective section	$0.6 f_y$
3.	Bending	
	In plates, flats, tubes and similar sections	$0.66 f_y$
	In girders and rolled sections	$0.62 f_y$
4.	Shear Stress	
	Maximum	$0.43 f_y$
	Average	$0.38 f_y$
	For yield stress $fy <= 250$ Mpa	
	For $Fy > 250$ Mpa	$0.35 f_y$
5.	Bearing stress on flat surface	$0.8 f_y$

However, the permissible stresses in axial or flexural compression shall not exceed those as per relevant clauses considering the effect of buckling.

506.4.2. **Equivalent Stress**

506.4.2.1. When a member is subjected to a combination of stresses, the equivalent stress $\sigma_{e,cal}$ due to combination of shear stress $\tau_{v,cal}$ bearing stress $\sigma_{p,cal}$ and bending stress $\sigma_{bt,cal}$ tensile or $\sigma_{bc,cal}$ compressive is calculated from

$$\sigma_{e,cal} = \sqrt{(\sigma_{bt,cal})^2 + (\sigma_{p,cal})^2 + (\sigma_{bt,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$$\text{or } \sigma_{e,cal} = \sqrt{(\sigma_{bc,cal})^2 + (\sigma_{p,cal})^2 - (\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$\sigma_{cal}, \sigma'_{c}$ shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

506.4.2.2. Irrespective of the permissible increase of stress in other clauses, the equivalent stress $\sigma_{e,cal}$ calculated in Clause 506.4.2.1 above shall not exceed 92 per cent of yield stress.

506.5. **Permissible Stresses in Bolts, Rivets and Tension Rods**

506.5.1. **Fasteners:** All fasteners would be in accordance with Indian Standards. For bolts, the yield stress used for calculating the permissible stress would be derived from the property class chosen as per relevant Indian Standards. The nut should be of matching property class. For hot rolled and high tensile rivets, the yield stress would be in accordance with the relevant Indian Standards.

506.5.2. **Calculation of stresses:** In calculating shear and bearing stresses, the effective diameter of a rivet shall be taken as the hole diameter and that of bolt shall be taken as its nominal diameter. In calculating the axial tensile stress in a rivet, the gross area shall be used and in calculating the axial tensile stress in a bolt or screwed tension rod, the net area shall be used.

506.5.3. **Gross and net area**

506.5.3.1. The gross area of a rivet shall be taken as the cross-sectional area of the rivet hole. The nominal diameter of rivet shall be the diameter (cold) before driving. The nominal area of a rivet shall be based on the nominal diameter.

506.5.3.2. The net sectional area of a bolt or a screwed tension rod shall be taken as the area of the root of the threaded part or cross-sectional area of the unthreaded part whichever is lesser. The nominal diameter of a bolt shall be the diameter of the shank of the bolt. The nominal area of a bolt shall be based on the nominal diameter.

506.5.4. **Basic permissible stresses:** The basic permissible stresses for rivets, bolts, tension rods are given in Table 6.3

506.5.5. **Combined tensile and shear stresses:** Rivets and bolts subject to shear and externally applied tensile forces shall be so proportioned that the quantity.

$$[(\sigma_{tf,cal}/\sigma_{tf})^2 + (\tau_{vf,cal}/\tau_{vf})^2] \leq 1$$

Where

$\sigma_{tf,cal}$ = actual tensile stress in the rivet or bolt

σ_{tf} = permissible tensile stress in the rivet or bolt as given in Table 6.3

$\tau_{vf,cal}$ = actual shear stress in the rivet or bolt,

τ_{vf} = permissible shear stress in the rivet or bolt as given in Table 6.3

506.5.6. **HSFG bolts :** High strength friction grip (HSFG) bolts shall be used in conformity with IS:4000-1992.

$$\sigma_{e,cal} = \sqrt{(\sigma_{bt,cal})^2 + (\sigma_{p,cal})^2 + (\sigma_{bt,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$$\text{or } \sigma_{e,cal} = \sqrt{(\sigma_{bc,cal})^2 + (\sigma_{p,cal})^2 - (\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

σ_{cal}, σ_{e} shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

506.4.2.2. Irrespective of the permissible increase of stress in other clauses, the equivalent stress $\sigma_{e,cal}$ calculated in Clause 506.4.2.1 above shall not exceed 92 per cent of yield stress.

506.5. **Permissible Stresses in Bolts, Rivets and Tension Rods**

506.5.1. **Fasteners:** All fasteners would be in accordance with Indian Standards. For bolts, the yield stress used for calculating the permissible stress would be derived from the property class chosen as per relevant Indian Standards. The nut should be of matching property class. For hot rolled and high tensile rivets, the yield stress would be in accordance with the relevant Indian Standards.

506.5.2. **Calculation of stresses:** In calculating shear and bearing stresses, the effective diameter of a rivet shall be taken as the hole diameter and that of bolt shall be taken as its nominal diameter. In calculating the axial tensile stress in a rivet, the gross area shall be used and in calculating the axial tensile stress in a bolt or screwed tension rod, the net area shall be used.

506.5.3. **Gross and net area**

506.5.3.1. The gross area of a rivet shall be taken as the cross-sectional area of the rivet hole. The nominal diameter of rivet shall be the diameter (cold) before driving. The nominal area of a rivet shall be based on the nominal diameter.

506.5.3.2. The net sectional area of a bolt or a screwed tension rod shall be taken as the area of the root of the threaded part or cross-sectional area of the unthreaded part whichever is lesser. The nominal diameter of a bolt shall be the diameter of the shank of the bolt. The nominal area of a bolt shall be based on the nominal diameter.

506.5.4. **Basic permissible stresses:** The basic permissible stresses for rivets, bolts, tension rods are given in Table 6.3

506.5.5. **Combined tensile and shear stresses:** Rivets and bolts subject to shear and externally applied tensile forces shall be so proportioned that the quantity.

$$[(\sigma_{tf,cal}/\sigma_{tf})^2 + (\tau_{vf,cal}/\tau_{vf})^2] \leq 1$$

Where

$\sigma_{tf,cal}$ = actual tensile stress in the rivet or bolt

σ_{tf} = permissible tensile stress in the rivet or bolt as given in Table 6.3

$\tau_{vf,cal}$ = actual shear stress in the rivet or bolt,

τ_{vf} = permissible shear stress in the rivet or bolt as given in Table 6.3

506.5.6. **HSFG bolts :** High strength friction grip (HSFG) bolts shall be used in conformity with IS:4000-1992.

Table 6.3. Basic Permissible Stresses for Rivets, Bolts and Tension Rods

1.	In tension	
	Axial stress on nominal area of rivet and on net area of bolts and tension rods :	
	Power driven shop rivets	$0.33 f_y$
	Power driven field rivets	$0.27 f_y$
	Bolts over 38 mm dia	$0.53 f_y$
	Bolts 20 mm upto 38 mm dia	$0.40 f_y$
	Bolts less than 20 mm dia	$0.33 f_y$
	Tension rods	$0.53 f_y$
2.	In shear	
	Shear stress on gross area of rivets and nominal area of bolts :	
	Power driven shop rivets	$0.43 f_y$
	Power driven field rivets	$0.40 f_y$
	Hand driven rivets	$0.33 f_y$
	Turned and fitted bolts (IS:3640)	$0.43 f_y$
	Black bolts (IS:1363)	$0.37 f_y$
3.	In bearing	
	Bearing stress on gross diameter of rivets and nominal diameter bolts:	
	Power driven shop rivets	$1.00 f_y$*
	Power driven field rivets	$0.90 f_y$
	Hand driven rivets	$0.67 f_y$
	Turned and fitted bolts (IS:3640)	$1.00 f_y$
	Black bolts (IS:1363)	$0.87 f_y$

506.6. Permissible Stresses in Welds

506.6.1. **Basic permissible stresses:** The basic permissible stresses in weld shall be as per Indian Standards namely IS:816-1969 and as modified in IS:1024-1979.

506.6.2. **Shop welds**

506.6.2.1. **Butt welds:** Butt weld shall be treated as parent metal with a thickness equal to the throat thickness, and the stress shall not exceed those permitted in the parent metal.

506.6.2.2. **Fillet welds:** The basic permissible stress in fillet welds shall not exceed the permissible shear stress as follows :

Steel Conforming as per IS:815-1974	Electrode Designation	Shear Stress Mpa
IS:2062-1969	EXXX-43X	108
IS:8500-1991	EXXX-51X	131
Grade Fe 510 W-HT		

506.6.2.3. **Plug welds:** The permissible shear stress in plug welds will not exceed those given for fillet welds as above.

506.6.3. **Site welds:** The permissible stresses for shear and tension for site welds made during erection of structural members shall be reduced to 80 per cent of those given in Clause 506.6.2. Site welding should be proposed only if quality welds can be ensured at site including facilities for testing the welds as per codal requirements. The percentage of site welds to be tested should be 100 per cent as given under Clause 513.6.12.7.2 to 4.

506.6.4. **Combined stresses in a weld**

506.6.4.1. When a weld is subjected to a combination of stresses, the equivalent stress $\sigma_{e,cal}$ due to combination of shear stress $\tau_{v,cal}$ bearing stress $\sigma_{p,cal}$ and bending stress $\sigma_{bt,cal}$ tensile or $\sigma_{bc,cal}$, compressive is calculated from

$$\sigma_{e,cal} = \sqrt{(\sigma_{bt,cal})^2 + (\sigma_{p,cal})^2 + (\sigma_{bt,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$$\text{or } \sigma_{e,cal} = \sqrt{(\sigma_{bc,cal})^2 + (\sigma_{p,cal})^2 + (\sigma_{bc,cal})(\sigma_{p,cal}) + 3(\tau_{v,cal})^2}$$

$\sigma_{c,cal}$ shall not exceed the permissible stresses as indicated in relevant sections under different combination of stresses.

506.6.4.2. Irrespective of the permissible increase of stress in other clauses, the equivalent stress $\sigma_{e,cal}$ calculated in Clause 506.6.4.1 shall not exceed 92 per cent of yield stress f_y.

506.7. **Stress Analysis**

506.7.1. **General:** The global analysis of the structure should be done using an elastic method. For structures in which the load effects are not proportional to the loads and/or the secondary effects due to deformation are significant, the method of analysis should be suitable for treatment of non-linear behaviour.

506.7.2. **Sectional properties:** The sectional properties to be used in global analysis should generally be calculated for the gross section assuming the specified sizes. For beams or trusses on flexible supports account should, however, be taken of its influence of shear lag on their stiffnesses. The effect of shear lag should also be taken into account in analysis of conditions during erection of continuous girders of box construction or with integral decks.

506.7.3. **Longitudinal stress in beams:** The distribution of longitudinal stress between the flanges and web or webs of a beam may be calculated on the assumption that the section remains plane, but using effective widths of the flanges and the effective thickness of a deep web in accordance with the provisions of Clause 508, no further account need be taken of deformation of the plate out of its plane.

506.7.4. **Shear stress:** The design values of shear stress in webs of rolled or fabricated I, box or channel sections may be calculated in accordance with the provisions of Clause 508. Shear stresses in other

sections should be computed from the whole cross-section having regard to the distribution of flexural stress across the section.

506.8. **Stresses**

506.8.1. **Primary stresses:** In the design of triangulated structures, axial stresses in members are usually calculated on the assumption that :

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

These stresses are defined as primary stresses.

506.8.2. **Secondary stresses:** In practice these assumptions are not realised and consequently members are subjected not only to axial stress but also to bending and shear stresses. These stresses are defined as secondary stresses and fall into two groups :

(i) Stresses which are the result of eccentricity of connections and off-joint loading generally (i.e., loads rolling directly on chords, self weight of member and wind loads on member)

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

506.8.2.1. Structures shall be designed, fabricated and erected in such a manner as to minimise as far as possible secondary stresses.

506.8.2.2. Secondary stresses which are the result of eccentricity of connections and of off-joint loading [under Clause 506.8.2 (i)], shall be computed and combined with the co-existent axial stresses in accordance with appropriate Clause, but secondary stress due to the self weight and wind on the member shall be ignored in this case.

Note :- In computing the secondary stress due to loads being carried direct by a chord, the chord may be assumed to be a continuous girder supported at the panel points, the resulting bending moments, both at the centre and at the supports being taken as equal to 3/4 of the maximum bending moment in a simply supported beam of span equal to the panel length. Where desired, calculations may be made and the calculated bending moments may be taken. In computing such bending moments, the impact allowances shall be based on a loaded length equal to one panel length.

506.8.2.3. Secondary stresses which are the result of the elastic deformation of the structure [under Clause 506.8.2 (ii)] shall be either computed or assumed in accordance with Clause 506.8.3 and combined with the co-existent axial stresses.

506.8.3. **Deformation stresses:** In order to minimise the deformation stresses in girder, the ratio of the width of the members in the plane of distortion to their length between centre of intersections shall preferably be not greater than 1/12 of the chord members and 1/24 of web members. In the absence of calculations the deformation stresses shall be assumed to be not less than $16\frac{2}{3}$ per cent of the dead load and live load stresses including impact.

506.8.3.1. All open web girders of effective spans greater than 50 m may properly be cambered. Recommended procedure for cambering such girders is given in ***Appendix-B***. For such girders, deformation stresses (under Clause 506.8.3) may be ignored.

507. GENERAL DESIGN CONSIDERATIONS

507.1. Effective Spans

The effective span shall be as given below :

(a) *For main girders* - The distance between centres of bearings

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

(c) *For rail or road bearers* - The distance between the centres of cross members

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

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

507.2. Effective Depths

The effective depth of plate or truss girder should be taken as the distance between the centres of gravity of the upper and lower flanges or chords.

507.3. Spacing of Girders

The distance between centres between the main girders shall be sufficient to resist over turning or overstressing due to lateral forces and loading conditions. Otherwise special provisions must be made to prevent this. This distance shall not be less than 1/20 of the span.

507.4. Depth of Girders

Minimum depth preferably shall not be less than the following :

(a) For trusses : 1/10 of the effective span

(b) For rolled steel joists and plate girders : 1/25 of the effective span

(c) For composite steel and concrete bridge :

(i) Overall depth : 1/25 of the effective span

(ii) Steel beam or girder : 1/30 of the effective span

The effective depth of open web girders shall not be greater than three times the distance between the centres of main girders.

507.5. Deflection of Girders

507.5.1. Rolled steel beams, plate girders or lattice girders, either simple or continuous spans, shall be designed so that the total deflection due to dead load, live load and impact shall not exceed 1/600 of the span.

Additionally, the deflection due to live load and impact shall not exceed of 1/800 of the span.

507.5.2. The deflection of cantilever arms at the tip due to dead load, live load and impact shall not exceed 1/300 of the cantilever arm and deflection due to live load and impact shall not exceed 1/400 of the cantilever arm.

507.5.3. Sidewalk live load may be neglected in calculating deflection.

507.5.4. When cross bracings or diaphragms of sufficient depth and strength are provided between beams to ensure the lateral distribution of loads the deflection may be calculated considering all beams acting together.

507.5.5. The gross moment of inertia shall be used for calculating the deflection of beams or plate girders. In calculating the deflection of trusses, the gross area of each member should be used.

507.6. Camber

507.6.1. Camber, if any, shall be provided as specified by the engineer. Camber may be required to maintain clearance under all conditions of loading or it may be required on account of appearance.

507.6.2. In the absence of specific guidance, the following principles may be observed:

(a) Beams and plate girders upto and including 35 m span need not be cambered.

(b) In open web spans, the camber of the main girders and the corresponding variations in length of members shall be such that when the girders are loaded with full dead load plus 75 per cent of the live load without impact producing maximum bending moment, they shall take up the true geometrical shape assumed in their design. The camber diagram shall be prepared as indicated in *Appendix-B*.

507.7. Minimum Sections

507.7.1. For all members of the structure, except parapets and packing plates, the following minimum thicknesses of plates and rolled sections shall apply :

(a) 8 mm when both sides are accessible for painting or are in close contact with other plates or rolled sections, or are otherwise adequately protected against corrosion. When one side is not readily accessible for painting or is not in close contact with another member, or is not otherwise adequately protected and where the thickness required by calculation is less than 12.5 mm, 1.5 mm shall be added to the calculated thickness subject to the total thickness being not less than 10 mm.

(b) 6 mm for box members when the inside of the member is effectively sealed.

(c) For rolled steel beams and channels, the controlling thickness shall be taken as the mean thickness of the flange, regardless of the web thickness.

507.7.2. In floor plates and parapets, minimum thickness of 6 mm shall be used if both sides are exposed or 8 mm if only one side is exposed. For packing plates, the thickness shall not be less than 1.5 mm.

507.7.3. In riveted construction, no angle less than 75 x 50 mm shall be used for the main members of the girders.

507.7.4. No angle less than 65 x 45 mm and no flat less than 50 mm wide shall be used in any part of a bridge structure, except for hand railings and shear connectors.

507.7.5. End angles connecting stringers to cross girders or cross girders to main girders shall not be less than three quarters of the thickness of the web plates of the stringers and cross girders respectively.

507.8. **Sectional Area**

507.8.1. **Gross sectional area:** The gross sectional area shall be the area of the cross-section as calculated from the specified sizes.

507.8.2. **Effective sectional area**

507.8.2.1. Tension members- The effective sectional area of the member shall be the gross sectional area with the following deductions as appropriate.

507.8.2.1.1. Except as required in Clause 507.8.2.1.2 the areas to be deducted shall be the sum of the sectional areas of the maximum number of holes in any cross-section at right angles to the direction of stress in the member.

507.8.2.1.2. In the case of :

(a) all axially loaded tension members

(b) beams of structural steel conforming to IS : 2062 and with d/t greater than 85

(c) beams of structural steel conforming to IS:8500 and with d/t greater than 75

where,

t = thickness of web, and

d = depth of beams to be taken as the clear distance between flanges ignoring fillets.

and where bolt or rivet holes are staggered, the area to be deducted shall be the greater of the following :

(i) the maximum number of the holes in any cross-section at right angles to the direction of stress in the member.

(ii) the sum of the sectional areas of all holes in a chain of lines extending progressively across the member, less $s^2t/4g$ for each line extending between holes at other than right angles to the direction of stress, where, s, g and t are respectively the staggered pitch, gauge, and thickness associated with the line under consideration. The chain of lines shall be chosen to produce the maximum such deduction. For non-planer sections, such as, angles, with holes in both legs, the gauge, g, shall be the distance along the centre of the thickness of the section between hole centres.

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

507.8.2.1.3. *Angles and tees in tension*

(a) In the case of single angle connected through one leg the net effective sectional area shall be taken as :

$$A1 + A2 \times k$$

where,

$A1$ = effective cross-sectional area of the connected leg

$A2$ = the gross cross-sectional area of the unconnected leg, and

$$k = 3A1 \div (3A1 + A2)$$

Where lug angles are used, the effective sectional area of the whole of the angle member shall be considered.

(b) In the case of pair of angles back-to-back (or a single tee) connected by one leg of the angle (or by the flange of the tee) to the same side of a gusset, the net effective area shall be taken as

$$A1 + A2 \times k$$

where,

$A1$ and $A2$ are as defined in Clause 507.8.2.1.3 (a) and

$$k = 5A1 \div (5A1 + A2)$$

The angles shall be connected together along their length in accordance with the requirements as given in Clause 511.4.6.1.

507.8.2.2. **Compression members:** The gross sectional area shall be taken for all compression members subject to relevant clauses.

507.8.2.3. **Parts in shear:** The effective sectional area for calculating average shear stress for parts in shear shall be as follows :

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

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

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

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

507.9. Skew Bridges

For skew bridges, detailed analysis of forces shall be required. However, if the angle of skew is within 15°, such detailed analysis may not be necessary.

508. DESIGN OF BEAMS.

508.1. General

508.1.1. Beams are defined as members with solid webs or with openings, subjected primarily to bending, including members of rolled and hollow section, plate girders and box girders.

508.1.2. Beams shall be proportioned on the basis of the moment of inertia of the gross cross-section with the neutral axis taken at the centroid of that section. In computing the maximum stresses, the stresses calculated on this basis shall be increased in the ratio of the gross to the effective area of the flange section. For this purpose, the flange sectional area in riveted or bolted construction shall be taken to be that of the flange plates, flange angles and the portion of the web and side plates, if any, between the flange angles. In welded construction, the flange sectional area shall be taken to be that of flange plates and of the tongue plates (i.e., the thick vertical plates connecting flange to web), if any, upto a depth of the tongue plate equal to eight times its thickness, which shall not be less than twice that of the web.

508.2. Web Plates

508.2.1. **Minimum thickness:** The thickness of the web plate shall conform to the requirements of Clause 507.7 and further shall not be less than the following :

(a) *for unstiffened webs* :

$d_1/85$ for steel conforming to IS:2062

$d_1/75$ for steel conforming to IS:8500

(b) *for vertically stiffened webs* :

1/180 of the smaller clear panel dimension.

1/270 of the greater clear panel dimension

$d_2/200$ for steel conforming to IS:2062 or

$d_2/180$ for steel conforming to IS:8500

(c) *for webs stiffened, both vertically and horizontally* and with the horizontal stiffener at distance from the compression flange of 2/5 of the distance from the compression flange to the neutral axis :

1/180 of the smaller clear panel dimension,

1/270 of the greater clear panel dimension, and

$d_2/250$ for steel conforming to IS:2062 or

$d_2/225$ for steel conforming to IS:8500

(d) when there is also a horizontal stiffener at the neutral axis of the girder

1/180 of the smaller clear panel dimension,

1/270 of the greater clear panel dimension, and

$d_2/400$ for steel conforming to IS:2062 or

$d_2/360$ for steel conforming to IS:8500

In the above, d_1 is the clear distance between flange angles or, where there are no flange angles, between flanges (ignoring fillets); but where tongue plates having a thickness not less than twice the thickness of the web plate are used, d_1 is the depth of the girder between the flanges less the sum of the depths of the tongue plates or eight times the sum of the thicknesses of the tongue plates, whichever is less, and d_2 is twice the clear distance from the compression flange angles or plate, or tongue plate to the neutral axis.

508.3. **Flanges**

508.3.1. The effective sectional area of compression flanges shall be the gross area with the specified deduction for excessive width of plates (Clause 508.3.3., 508.3.4) and maximum deduction for open holes and holes for bolts occurring in section perpendicular to the axis of the member.

508.3.2. The effective sectional area of tension flanges shall be the gross sectional area with specified deduction for excessive width or projection of plates (Clause 508.3.5) and deduction of all holes as specified for rivet or bolt holes in tension members in Clause 507.8.2.1.

508.3.3. In riveted or bolted construction, flange angles shall form as large a part of the area of the flange as practicable (preferably not, less than 1/3) and the number of flange plates shall be kept to a minimum. Where flange plates are used, they shall preferably of equal thickness and atleast one plate of the top flange shall extend the full length of the girder, unless the top edge of the web is finished flush with the flange angles.

Compression flange plates unstiffened at their edges shall not project beyond the outer lines of connections to the flange angles by more than 16t' for steel conforming to IS:2062 or 14t' for steel conforming to IS:8500 where t' is the thickness of the thinnest flange plate or the aggregate thickness of the two or more plates when projecting portions of these plates are adequately tacked together.

508.3.4. In welded construction, compression flange plates unstiffened at their edges shall not project beyond the line of connection to the web or tongue plates by more than 12t'.

508.3.5. In all cases tension flange plates, stiffened or unstiffened at their edges, shall not project beyond the outer line of connections to the flange angles (or where there are no flange angles, to the web or tongue plates) by more than 20t'.

508.3.6. For the flanges of beams with vertical stiffeners only (see Clause 508.11.2.2), where d1/t is greater than 130 in the case of steel conforming to IS:2062 or 110 in the case of steel conforming to IS:8500 and when the average shear stress in the web is greater than 0.6 of the permissible stress given for mild steel in Clause 506.4.1, the quantity $I/(b^3t)$ shall not be less than 2.5×10^{-4} in the case of steel conforming to IS:2062 and 3×10^{-4} in the case of steel conforming to IS:8500.

where,

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

d_1 =effective depth of the girder as defined in Clause 508.2.1

b = spacing of stiffeners

t = thickness of web

508.4. **Effective Length of Compression Flanges**

The effective length of the compression flange for buckling normal to the plane of the girder shall be as given below.

508.4.1. Simply supported beams with no intermediate lateral support to compression flange, but with each end restrained against torsion.

508.4.1.1. When there is no intermediate lateral restraint to a compression flange, effective length l should be taken as

$l = k l \, L$

where,

L = span of the beam (i.e. between restraint at supports)

kl = 1.0 if the compression flange is free to rotate in plan at the points of support, or

= 0.85 if the compression flange is partially restrained against rotation in plan at one support and free to rotate in plan at the other, or

= 0.7 if the compression flange is fully restrained against rotation in plan at the points of support.

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

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

In the above formula :

l = has the value given in Clause 508.4.1.1.

f_{cb} = the critical stress in the flange as defined in Clause 508.6.2

f_{bc} = the calculated working stress in flange

δ = the deflection of the flange under the action of unit horizontal force as defined in Clause 508.4.2

508.4.2. Simply supported beams with compression flange laterally supported by *U-Frames*.

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

$$l = 2.5 \times \sqrt[4]{(E \, I_c \, a \, \delta)} \text{ but not less than "a"}$$

where,

E = Young's Modulus

I_c = maximum moment of inertia of compression flange about its centroidal axis parallel to the web of the girder

a = distance between frames

δ = the lateral deflection which would occur in the U-Frame at the level of the centroid of the flange being considered when a unit force acts laterally to the U-Frame only at this point and simultaneously at each corresponding point on the other flange or flanges connecting to the same U-Frame. The direction of each unit force should be such as to produce the maximum aggregate value of δ. The U-Frame should be taken as fixed in position at each point or intersection between the cross member and a vertical as free and unconnected at all other points.

when δ is not greater than $a^3/(40 \, E \, I_c)$

$l = a$

In cases of symmetrical U-Frames where cross members and stiffners are each of constant moment of inertia throughout their own length.

$$\delta = \frac{(d_1)^3}{3EI_1} + \frac{(d_2)^2 b}{EI_2}$$

where,

d_1 = distance of the centroid of the compression flange from the top of the cross member

d_2 = distance of the centroid of the compression flange from the neutral axis of the cross member

b = half the distance between centres of the main girders

I_1 = the moment of inertia of a pair of stiffeners about the centre of the web, or a single stiffener about the face of the web. A width of web plate upto 16 times the web thickness may be included on each side of centerline of connection

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

508.4.3. **Beams with laterally supported compression flanges:** When a compression flange is provided with effective discrete lateral restraints effective length l should be taken as the greatest distance centre to centre of restraint members between a restraint and a support. Where such restraint is provided by interconnecting bracing, consideration should be given to the possibility of lateral instability of the combined cross-section.

508.4.4. **Cantilever beams without intermediate lateral support:** When a cantilever beam is not provided with lateral support between its support and tip, l may be taken from Table 8.1 where L is the length of cantilever.

Table 8.1. Effective Length l for a Cantilever Beam without Intermediate Lateral Restraint

(Clause 508.4.4)

Restraint Conditions		Position of load	
At support	At tip	On tension flange where there is no lateral restraint to load or flange	At other position
1. Built in	(a) Free (b) Tension flange held against displacement (c) Both flanges held against lateral displacement	1.4 L 1.4 L 0.6 L	0.8 L 0.7 L 0.6 L
2. Continuous and both flanges held against lateral displacement	(a) Free (b) Tension flange held against displacement (c) Both flanges held against lateral displacement	2.5 L 2.5 L 1.5 L	1.0 L 0.9 L 0.8 L
3. Continuous with tension flanges held against lateral displacement	(a) Free (b) Tension flange held against displacement (c) Both flanges held against lateral displacement	7.5 L 7.5 L 4.5 L	3.0 L 2.7 L 2.4 L

Note . L is the length of the cantilever

508.4.5. **Beams continuously restrained by deck at compression flange level:** A compression flange continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (i.e., $l = 0$) if the frictional or positive connection of the deck to the flange is capable of resisting a lateral force of 2.5 per cent of the force in the flange at the point of maximum bending moment, distributed uniformly along length.

508.5. **Slenderness Ratio**

The slenderness ratio λ *(i.e., l / r_{yy})* of a beam shall not exceed 300 and it shall not exceed 150 for cantilevers.

Where, l = effective length of the compression flange as specified in Clause 508.4.

r_{yy} = the radius of gyration of the whole beam about its y-y axis based on the gross moment of inertia and the gross sectional area.

508.6. **Permissible Bending Stresses**

508.6.1. The tensile and compressive bending stresses calculated according to Clause 508.1.2 shall not exceed the appropriate permissible stresses in Table 6.2.

508.6.2. For beams and plate girders with I smaller than I_x

where,

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

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

The bending compression stress calculated according to Clause 508.1.2 shall not exceed the permissible bending compressive stress σ_{bc} given in Table 8.2 corresponding to f_{cb}, (elastic critical stress), calculated as follows :

Table 8.2. Values of σ_{bc} Calculated from f_{cb} for Different Values of f_y

(Clause 508.6.2)

All Units in MPa

$f_y \rightarrow$ / $f_{cb} \downarrow$	250	340	400
20	13	13	13
30	19	19	19
40	25	26	26
50	31	31	32
60	36	37	38
70	41	43	44
80	46	48	49
90	51	54	55
100	55	59	60
110	60	64	65
120	64	68	70
130	67	73	75
140	71	77	80
150	74	81	84
160	78	85	89
170	81	89	93
180	84	93	97
190	87	97	102
200	89	100	105
210	92	103	109
220	94	106	112
230	96	110	116

240	99	113	119
250	101	115	122
260	103	118	126
270	104	121	129
280	106	123	132
290	108	126	135
300	110	128	137
310	111	130	140
320	113	133	143
330	114	135	145
340	115	137	148
350	117	139	150
360	118	141	152
370	119	143	155
380	120	144	157
390	121	146	159
400	122	148	161
420	124	151	165
440	126	154	169
460	128	157	172
480	130	159	175
500	131	162	178
520	133	164	181
540	134	166	184
560	135	168	187
580	136	170	189
600	137	172	192
620	138	174	194
640	139	175	196
660	140	177	198
680	141	178	200

700	142	180	202
720	143	181	204
740	143	182	205
760	144	184	207
780	145	185	208
800	145	186	210
850	147	188	213
900	148	191	216
950	149	193	219
1000	150	195	222
1050	151	196	224
1100	152	198	226
1150	152	199	228
1200	153	200	230
1300	154	203	233
1400	155	205	236
1500	156	206	238
1600	157	208	240
1700	157	209	242
1800	158	210	243
1900	158	211	245
2000	159	212	246
2200	160	213	248
2400	160	215	250
2600	161	216	251
2800	161	216	252
3000	161	217	253
3500	162	218	255
4000	163	219	257
4500	163	220	258
5000	163	221	259
5500	163	221	259
[illegible]	164	[illegible]	260

Elastic critical stress

The elastic critical stress f_{cb} for beams and plate girders with l/r_y smaller than lx shall be calculated using the following formula :

$$f_{cb} = k_1 (X + k_2 Y) (c_2 / c_1)$$

where,

$$X = Y \sqrt{1 + (1 / 20) [(l T) / (r_y D)]^2} \quad MPa$$

$$Y = 26.5 \times 10^5 / (l / r_y)^2 \quad MPa$$

c_1, c_2 = respectively the lesser and greater **distances from the section** neutral axis to the extreme fibres

D = overall depth of the beam

T = mean thickness of the compression flange

l = effective length of compression flange

r_y = radius of gyration of the section about its axis of minimum strength (y-y axis)

k_1 = a co-efficient to allow for reduction in thickness or breadth of flanges between the points of effective lateral restraint and depends on ψ the ratio of total area of both flanges at the point of least bending moment to the corresponding area at the point of greatest bending moment between such points of restraint. Values of k_1 for different values of ψ are given in Table 8.3.

k_2 = a co-efficient to allow for the inequality of flanges and depends on ϖ, the ratio of the moment of inertia of the compression flange alone to that of the sum of the moments of inertia of the flanges each calculated about its own axis parallel to the axis of the girder, at the point of maximum bending moment. The values of k_2, for different values ϖ of are given in Table 8.4. Values of X and Y for appropriate values of D/T and l/r_y are given in Table 8.5.

Table 8.3. Value of k_1 for Beams with Curtailed Flanges

(Clause 508.6.2)

ψ	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k_1	1.0	1.0	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2

Note : Flanges should not be reduced in breadth to give a value of lower than 0.25

Table 8.4. Values of k_2 for Beams with Unequal Flanges

(Clause 508.6.2)

β	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0.0
k_2	-0.5	0.4	0.3	0.2	0.1	0	-0.2	-0.4	—.6	-0.8	-1.0

508.6.2.1. Values of f_{cb} shall be increased by 20 per cent when T/t is not greater than 2.0 and d_1/t is not greater than $1344/\sqrt{f_y}$ where d_1 is as defined in Clause 508.2.1 and t the thickness of the web and the value of f_y is expressed in MPa.

508.6.3. **Beams bent about the axis of minimum strength (y-y axis):** The maximum permissible bending stress in tension or in compression in beams bent about the axis of minimum strength shall not exceed the appropriate permissible stresses in Table 6.2.

508.6.4. **Angle and tee shapes:** The bending stress in the leg when loaded with the flange or table in compression shall not exceed the appropriate permissible stresses in Table 6.2. When loaded with the leg in compression, the permissible bending stress shall be calculated from Clause 508.6.2 with k_2 = -1.0 and T = thickness of leg.

Table 8.5. Values of X and Y for Calculating f_{cb}

(Clause 508.6.2)

D/T →	X															Y
l/r_y ↓	8	10	12	14	16	18	20	25	30	35	40	50	60	80	100	
40	2484	2222	2066	1965	1897	1849	1814	1759	1728	1709	1697	1683	1675	1667	1663	1656
45	2103	1856	1709	1612	1546	1409	1465	1411	1380	1362	1349	1335	1327	1319	1315	1309
60	1822	1590	1449	1357	1293	1248	1214	1161	1131	1113	1101	1086	1078	1070	1067	1060
55	1607	1389	1234	1166	1105	1060	1028	976	947	929	917	902	894	886	883	876
60	1437	1232	1104	1020	961	918	886	835	806	788	776	762	754	746	743	736
65	1301	1107	985	904	847	806	775	726	697	679	657	653	645	637	634	627
70	1188	1005	889	811	757	717	687	638	610	592	581	567	559	551	547	541
75	1094	920	810	735	682	644	615	567	540	522	511	467	489	481	478	471
80	1014	849	743	672	621	584	556	509	482	465	454	440	432	424	421	414
85	945	788	687	618	570	533	506	461	434	417	406	392	385	377	373	367
90	886	735	639	573	526	491	464	420	394	377	366	353	345	337	373	327
95	833	689	597	534	488	454	428	385	360	343	332	319	311	304	300	294
100	787	649	560	499	455	423	398	356	331	314	304	290	283	275	272	265
110	708	582	499	443	402	371	347	307	283	268	257	244	237	229	226	219
120	644	527	451	398	359	330	308	270	247	232	222	209	202	194	191	184
130	591	482	411	361	325	298	277	240	218	204	194	181	174	167	163	157
140	546	444	378	331	297	271	251	217	195	181	172	160	153	145	142	135
150	508	412	350	306	274	249	230	197	177	163	154	142	135	145	124	118
160	474	385	326	284	254	230	212	181	161	148	139	127	121	113	110	104
170	445	360	306	265	236	214	197	167	148	135	126	115	109	102	95	92
180	420	339	286	249	221	200	184	155	137	125	116	105	98	92	88	82
190	397	320	270	235	208	188	172	145	127	115	107	96	90	83	80	73
200	476	304	256	222	197	177	162	136	119	107	99	89	83	76	78	66
210	358	238	243	210	186	168	153	128	112	101	93	82	76	70	66	60
220	341	275	231	200	177	159	145	121	105	94	87	77	71	64	61	55
230	326	262	220	191	169	152	138	115	99	89	82	72	66	60	56	50
240	312	251	211	182	161	145	132	109	94	84	77	67	62	55	52	46
250	299	241	202	175	151	139	126	104	90	80	73	64	58	52	49	42
260	288	231	194	167	148	133	121	99	85	76	68	60	55	48	45	39
270	277	222	186	161	142	127	115	95	82	72	66	57	52	46	42	36
280	267	214	180	155	137	122	111	91	78	69	63	54	49	43	40	34
290	257	207	173	149	132	118	107	88	75	66	60	52	46	41	38	32
300	249	200	167	144	127	114	103	84	72	64	57	49	44	38	35	30

Note— Intermediate values may be obtained by linear interpolation.

508.7. Permissible Shear Stress

508.7.1. **Maximum shear stress:** The maximum shear stress in a member having regard to the distribution of stresses in conformity with the elastic behaviour of the member in flexure, shall not exceed the appropriate permissible stress in Table 6.2.

508.7.2. **Average shear stress:** The average shear stress in a member calculated on the cross-section of the web shall not exceed the maximum permissible average shear stress $_{va}$ as follows :-

(a) *For unstiffened webs*: Appropriate permissible stress in Table 6.2.

(b) *For stiffened webs:* The values given in Tables 8.6, 8.7 and 8.8 as appropriate yield stress values 250, 340 and 400 MPa respectively.

Note : The allowable stresses given in Tables 8.6, 8.7 and 8.8 apply provided any reduction of the web cross-section is due only to rivet/bolt holes, etc. where large apertures are cut in the web, a special analysis shall be made to ensure that the maximum permissible average shear stresses laid down in this standard are not exceeded.

Table 8.6. Permissible Average Shear Stress τ_{w} in Stiffened Webs of Steel with f_y=250 Mpa

(Clause 508.7.2)

Strees τ_{w} (Mpa) for different distances between stiffeners

d/t	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
90	100	100	100	100	100	100	100	100	100	100	100	100	100
95	100	100	100	100	100	100	100	100	100	100	100	100	99
100	100	100	100	100	100	100	100	100	100	100	99	99	98
105	100	100	100	100	100	100	100	100	100	99	98	97	96
110	100	100	100	100	100	100	100	100	99	98	96	95	94
115	100	100	100	100	100	100	100	100	98	96	95	94	93
120	100	100	100	100	100	100	100	98	96	95	93	92	91
125	100	100	100	100	100	100	98	97	95	93	92	91	90
130	100	100	100	100	100	99	97	96	94	92	90	89	88
135	100	100	100	100	100	98	96	94	92	90	89	87	86
140	100	100	100	100	99	96	95	93	91	89	87	86	85
150	100	100	100	100	97	94	92	90	88	85	84	83	81
160	100	100	100	98	94	92	89	88	85	83	81	80	78
170	100	100	100	96	92	89	87	85	82	80	78	76	75
180	100	100	98	94	90	87	84	82	80	77	75	73	72
190	100	100	97	92	88	84	82						
200	100	100	95	90	86	82	81						
210	100	99	93	88	83	81							
220	100	98	91	86	81	80							
230	100	96	90	84	79								
240	100	95	88	83	77								
250	100	93	86	82	74								
260	100	92	85	81									
270	100	90	84	81									

Non-applicable zone

Note— Intermediate values may be obtained by linear interpolation.

Table 8.7. Permissible Average Shear Stress τ_{w} in Stiffened Webs of Steel with f_y=340 Mpa

(Clause 508.7.2)

Strees τ_{w} (Mpa) for different distances between stiffeners

d/t	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
75	136	136	136	136	136	136	136	136	136	136	136	136	136
80	136	136	136	136	136	136	136	136	136	136	136	136	136
85	136	136	136	136	136	136	136	136	136	136	136	134	133
90	136	136	136	136	136	136	136	136	136	135	133	132	131
95	136	136	136	136	136	136	136	136	135	133	131	129	128
100	136	136	136	136	136	136	136	136	132	130	128	127	126
105	136	136	136	136	136	136	135	133	130	128	126	124	123
110	136	136	136	136	136	135	133	131	128	126	124	122	120
115	136	136	136	136	136	133	131	129	126	123	121	119	118
120	136	136	136	136	135	131	129	127	124	121	119	117	115
125	136	136	136	136	133	129	127	125	121	119	116	114	113
130	136	136	136	135	131	127	125	122	119	116	114	112	110
135	136	136	136	134	129	126	123	120	117	114	111	109	108
140	136	136	136	132	127	124	121	118	115	112	109	107	105
150	136	136	136	129	124	120	117	114	110	107	104	102	100
160	136	136	132	126	120	116	113	110				97	95
170	136	136	129	123	117	112	109	106	101	98	95	92	90
180	136	135	127	119	113	108	105	102	97	93	90	87	84
190	136	133	124	116	110	105	100						
200	136	130	121	113	106	101	96						
210	136	128	118	110	103	97							
220	136	126	116	107	99	93							
230	136	123	113	103	96								
240	134	121	110	100	92								
250	132	119	107	97	89								
260	130	116	104	94									
270	128	114	102	91									

Non-applicable zone.

Note— Intermediate values may be obtained by linear interpolation.

Table 8.8. Permissible Average Shear Stress τ_{va} in Stiffened Webs of Steel with f_y=400 Mpa

(Clause 508.7.2)

Strees τ_{va} (Mpa) for different distances between stiffeners

d/t_w	0.3d	0.4d	0.5d	0.6d	0.7d	0.8d	0.9d	1.0d	1.1d	1.2d	1.3d	1.4d	1.5d
70	160	160	160	160	160	160	160	160	160	160	160	160	160
75	160	160	160	160	160	160	160	160	160	160	160	160	159
80	160	160	160	160	160	160	160	160	160	160	159	157	156
85	160	160	160	160	160	160	160	160	160	158	156	154	152
90	160	160	160	160	160	160	160	160	157	155	152	151	149
95	160	160	160	160	160	160	159	157	154	152	149	147	146
100	160	160	160	160	160	160	157	155	151	149	146	144	143
105	160	160	160	160	160	157	154	152	149	146	143	141	139
110	160	160	160	160	159	155	152	149	146	143	140	138	136
115	160	160	160	160	156	152	149	147	143	140	137	135	133
120	160	160	160	159	154	150	147	144	140	137	134	132	129
125	160	160	160	157	152	147	144	141	137	134	131	128	126
130	160	160	160	155	150	145	141	139	131	131	128	125	123
135	160	160	160	153	147	143	139	136	132	128	125	122	120
140	160	160	158	151	145	140	136	133	129	125	122	119	116
150	160	160	155	147	141	135	131	128	123	119	115	112	110
										Non-applicable zone			
160	160	160	151	143	136	130	126	123	117	---	---	---	103
170	160	153	148	139	132	126	121	117	112	107	103	100	97
180	160	155	144	135	127	121	116	112	106	101	97	93	90
190	160	152	140	131	123	116	111						
200	160	149	137	127	118	111	106						
210	160	146	133	123	114	106							
220	157	143	130	119	109	101							
230	155	140	126	114	105	143							
240	153	137	123	110	100	140							
250	151	134	119	106	96	135							
260	148	131	116	102		130							
270	146	128	112	98									

Note— Intermediate values may be obtained by linear interpolation.

508.8. **Curtailment of Flange Plates**

Each flange plate shall be extended beyond its theoretical cut-off point adequately. The extension shall contain sufficient rivets, bolts and welds to withstand the forces developed at the theoretical cut-off point.

In welded construction, the use of curtailed flange plates shall be avoided as far as possible, local strengthening being provided by other means, such as, inserting by butt welding a thicker and or wider plate. The heavier section plate shall be suitably tapered to the lighter plate. If, in welded construction the use of curtailed flange plates cannot be avoided, the end of the plate shall be tapered in plan to a rounded end and all welds shall be continuous round the ends.

508.9. **Connection of Flanges to Web**

The flanges of plate girders shall be connected to the web by sufficient rivets, bolts or welds to transmit the horizontal shear force combined with any vertical loads which are directly applied to the flange. In welded construction, where the web is in close contact with the flange before welding, vertical loads causing compression may be deemed to be resisted by the bearing between the flange and the web.

508.10. **Dispersion of Load Through Flange to Web**

Where a load is directly applied to a flange, it shall be considered as dispersed uniformly through the flange and the web at an angle of 45°.

508.11. **Web Stiffeners**

Web stiffeners shall be provided as follows :

508.11.1. **Load bearing stiffeners**

508.11.1.1 **General:** Webs of plate girders and rolled beams shall be provided with load bearing stiffeners at points of supports and at points of concentrated load where reaction or concentrated load exceeds the value of

$$\sigma_{ac} . t . B$$

where,

σ_{ac} = maximum permissible axial stress for struts as given in Clause 506.4.2.1 for a slenderness ratio of $(d_s \sqrt{3})/t$

t = web thickness

d_s = clear depth of web between root fillets

B = the length of the stiff portion of the bearing plus the additional length given by dispersion at 45° to the level of the neutral axis. The stiff portion of a bearing is that length which cannot deform appreciably in bending and shall not be taken as greater than half the depth of the beams continuous over a bearing.

508.11.1.2. Details and design

(a) Load bearing stiffeners should be in pairs (i.e., two legs of plates, angles, etc.) placed symmetrically at both sides of the web. When the condition is not met the effect of the resulting eccentricity should be considered.

(b) The ends of the load bearing stiffener should be closely fitted or adequately connected to both flanges. They should be shaped to allow space for any root fillet or weld connecting the web to the flange, with a clearance not exceeding five times the thickness of the web.

(c) Load bearing stiffeners shall not be joggled and shall be solidly packed throughout.

(d) Outstanding legs or each pair of load bearing stiffeners shall be so proportioned that the bearing stress on that part of their area in contact with the flange and clear of the root of the flange or flange angles or clear of the flange welds, does not exceed the bearing stress specified in Clause 506.4.1.

(e) Load bearing stiffeners consisting of two legs shall be designed as struts assuming the section to consist of the pair of stiffeners together with a length of web on each side of the centre line of the stiffeners equal to twenty times the web thickness (but limited to the edge distance of the web and half the distance of the adjacent stiffener).

In case of bearing stiffeners consisting of four or more legs, the effective stiffener section should be taken to comprise the stiffeners, the web plate between the two outer legs and a portion of web plate not exceeding the length of the web as specified for single leg stiffeners on the outer sides of the outer legs.

(f) The radius of gyration shall be taken about the axis parallel to the web of the beam or girder, and the working stress shall be in accordance with appropriate allowable value for a strut, assuming the effective length equal to 0.7 times the length of the stiffener.

(g) The load bearing stiffeners shall be provided with sufficient rivets, bolts or welds to transmit to the web the whole of the load in the stiffeners.

(h) When load bearing stiffeners at supports are the sole means of providing restraint against torsion (see Clause 508.4.1.2) the moment of inertia I of the stiffener shall not be less than

$$(D^3 T_m / 250) \times (R / W)$$

where, I = moment of inertia of the pair of stiffeners about the centre line of the web plate.

D = overall depth of the girder

T_m = maximum thickness of compression flange

R = reaction of the bearing

W = total load on girder

(i) In addition, the base of the stiffeners in conjunction with the bearing of the girder shall be capable of resisting a moment due to horizontal force specified in Clause 508.4.1.3.

508.11.2. **Intermediate stiffeners**

508.11.2.1. **General:** When the thickness of the web is less than the limits specified in Clause 508.2.1. (a), transverse stiffeners shall be provided throughout the length of the girder. When the thickness of the web is less than the limits specified in Clause 508.2.1. (b), longitudinal stiffeners shall be provided in addition to the transverse stiffeners.

In no case shall the greater unsupported clear dimension of a web panel exceed 270 t nor the lesser unsupported clear dimension of the same panel exceed 180 t where t is the thickness of the web plate.

508.11.2.2. **Transverse stiffeners:** Where transverse stiffeners are required, they shall be provided throughout the length of the girder at a distance apart not greater than 1.5 d_1 and not less than 0.33 d_1, where d_1 is the depth as defined in Clause 508.2.1. Where horizontal stiffeners are provided d_1 shall be taken as the clear distance between the horizontal stiffener and the farthest flange ignoring fillets.

Transverse-stiffeners shall be designed so that I is not less than :

$$1.5 \times (d_1^3 \times t^3) / S^2$$

where,

I	=	the moment of inertia of a pair of stiffeners about the centre of the web or a single stiffener about the face of the web
t	=	minimum required thickness of the web
S	=	the maximum permitted clear distance between stiffener for thickness t

Note : If the thickness of the web is made greater, or the spacing of stiffener made smaller than that required by the standard, the moment of inertia of the stiffener need not be correspondingly increased.

Intermediate transverse stiffeners, when not acting as load bearing stiffeners, may be joggled and may be single or in pairs placed one on each side of the web. Where single stiffeners are used, they should preferably be placed alternatively on opposite sides of the web. The stiffeners shall extend from flange to flange. They can be connected or fitted to, or kept well clear of the flanges.

508.11.2.3. **Longitudinal stiffeners**

Where longitudinal stiffeners are used in addition to vertical stiffeners they shall be as follows:

One longitudinal stiffener, on one or both sides of the web, shall be placed at a distance from the compression flange equal to two fifths of the distance from the compression flange to the neutral axis, when the thickness of the web is less than $d_2/200$ for steel conforming to IS:2062 and $d_2/180$ for steel conforming to IS:8500 where d_2 is the depth of web as defined in Clause 508.2.1. This stiffener shall have a moment of inertia I not less than $4\, S_1\, t^3$ where I and t are as defined in Clause 508.11.2.2 and S_1 is the actual distance between stiffeners.

A second longitudinal, on one or both sides of the web shall be placed on the neutral axis of the girder when the thickness of the web is less than $d_2/250$ for steel conforming to IS:2062 or $d_2/225$ for steel conforming to IS:8500. The stiffener shall have a moment of inertia I not less than $d_2 t^3$ where I and t are as defined in Clause 508.11.2.2 and d_2 in Clause 508.2.1.

Longitudinal stiffeners shall extend between vertical stiffeners but need not be continuous over them or connected to them.

508.11.2.4. **External forces on intermediate stiffeners:** When vertical intermediate stiffeners are subject to bending moments and shears due to the eccentricity of vertical loads, or the action of transverse forces, the moment of inertia I of the stiffeners specified in Clause 508.11.2.2 shall be increased as follows:

(a) Bending moment on stiffener due to eccentricity of vertical loading with respect to the vertical axis of the web :

Increase of I = $(1.5\, M\, D^2) / (E\, t)$

(b) Lateral loading on stiffener

Increase of I = $(3\, P\, D^3) / (E\, t)$

where,

M = the applied bending moment

D = overall depth of girder

E = Young's modulus

t = thickness of web

P = lateral force to be taken by the stiffener and deemed to be applied at the compression flange of the girder.

508.11.2.5. **Connection of intermediate stiffeners to web:** Intermediate transverse and longitudinal stiffeners not subjected to external loads shall be connected to the web by welds or rivets, in order to withstand a shearing force in kilograms per millimetre run between each component of stiffener and the web, of not less than $12.6 t^2/h$, where, t equals web thickness in mm and h equals the projection in mm, of the stiffener component from the web.

508.11.2.6. **Outstand of all stiffeners:** Unless the outer edge of each stiffener is continuously stiffened, the outstand from the web shall not be more than the following :

For sections : 16t for steel conforming to IS:2062

14t for steel conforming to IS:8500

For flats : 12t for all steels

Where t is the thickness of the section or flat.

508.12. **Flange Splices**

Flange joints should preferably not be located at points of maximum stress. Where splice plates are used, their area shall be atleast 5 per cent in excess of the area of the flange element spliced, their centre of gravity shall coincide, as nearly as possible, with that of the element spliced. There shall be enough rivets or welds on each side of the splice to develop the load in the element spliced plus 5 per cent, but in no case should the strength developed be less than 50 per cent of the effective strength of the material spliced.

In welded construction, flange plates or angles shall be joined by full penetration butt welds wherever possible. These butt welds shall develop the full strengths of plates or angles. Where this is not possible, splice plate should be used.

508.13. Splices in Webs

Splices in the webs of plate girders and rolled sections shall be designed to resist the shears and moments at the spliced section.

In riveted construction, splice plates shall be provided on each side of the web. In welded construction, web plates shall be joined by full penetration butt welds wherever possible. Where this is not possible, splice plate may be used on both sides.

508.14. End Connections

Connections at the ends of all beams designed as simply supported beams shall have flexibility to take angular deffection..

508.15. Lateral Bracing

All spans shall be provided with a lateral bracing system extending from end to end of sufficient strength to transmit to the bearings all lateral forces due to wind, seismic effect etc., as applicable.

508.16. Expansion and Contraction

In all bridges, provision shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes. Provision shall also be made for changes in length of span resulting from live loads.

509. DESIGN OF COMPRESSION MEMBERS

509.1. General

Design of compression members should generally follow the considerations under Clause 511.3 under "Trusses or Open Web Girders" of this Code.

509.2. Base Plate

509.2.1. Base plate is a structural part which serves as medium for uniformly transferring load from member/stanchion/column to foundation.

509.2.2. Area of base plate should be such that at any point reactive pressure acting on it is less than allowable stress of concrete in compression.

$$A_F = N / \sigma_{cr}$$

where,

A_F = Area of base plate

N = Load in the member

σ_{cr} = Allowable compressive stress of concrete

For (Crushing value of concrete) IS:456 may be referred for guidance.

509.2.2.1. Width of base plate should be $B = b(or\ h) + 2t_s + 2c$,

where,

b and h = Size of member/stanchion/column,

t_s = Thickness of saddle plate, 8 - 10 mm

c = Cantilever portion restricted to 100 - 120 mm, but not less than 20 mm from outside member, stiffener to the edge of base plate.

Length of base plate $L = \dfrac{A_F}{B}$

A_F = Area of base plate

509.2.2.2. Thickness of base plate should not be less than 20 mm.

509.2.2.3. Thickness of plate is determined from its bending consideration due to reactive pressure of foundation on base plate.

$$P_F = N / A_F$$

where,

P_F = Reactive pressure on base plate

Base plate area in general can be divided in four types depending upon boundary conditions of support (stiffeners).

(i) Cantilever

(ii) Supported on two sides perpendicular to each other

(iii) Supported on three sides

(iv) Supported on four sides

509.2.2.3.1. Bending moment in case of cantilever for 1 cm width of base plate (case-1) :

$$M_1 = P_F c^2 / 2$$

509.2.2.3.2. Maximum bending moment in centre of free B-side in cases of plate having support at three sides and also at two perpendicular sides:

$$M_2 = P_F b^2$$

where,

b = Length of free shorter side of plate

α = Coefficient as per table below depending on a/b.

a/b	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	2.0	More than 2
α	0.06	0.074	0.088	0.097	0.107	0.112	0.120	0.126	0.132	0.133

If $a/b \leq 0.5$ the support of plate on a-side does not have any effect, as such for bending moment on base plate formula for cantilever type should be used with $c = a$.

509.2.2.3.3. Maximum Bending Moment in case of plate having support at four sides.

$$M_3 = \beta P_F (b_1)^2$$

where,

b_1 = is short side length

β = Coefficient as per table below, depending on a_1/b_1.

a_1/b_1	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	More than 2
β	0.048	0.055	0.063	0.069	0.075	0.081	0.086	0.091	0.094	0.098	0.1	0.125

In case of $a_1/b_1 > 2$, plate works as single span simply supported beam and bending moment,

$$M_3 = P_F (b_1)^2 / 8$$

509.2.2.3.4. Thickness of the base plate $t_B = \frac{\sqrt{6M}}{\sigma}$

where,

M = Maximum bending moment considering all areas in which base plate is divided.

σ_{bc} = Maximum permissible bending stress in slab base.

509.2.2.4. Section of stiffeners saddle element of base plate and its connections are designed for loads coming on them. Main stiffeners are designed as simply supported over hanging beam loaded with uniformly distributed load equal to $qs_1 = P_F x\, l_1$ and checked for bending and shear stresses.

509.2.2.4.1. Secondary stiffeners (considering cantilever) are designed for load equal to $qs_2 = P_F \times l_2$ and checked for bending and shear stress.

509.2.2.5. Welded and riveted connections are designed to transfer the loads coming on stiffeners to main member and also to connect base plate with stiffeners.

509.2.2.6. In case of heavy load transfer from member to the base plate machining of contact surface between base plate and member is recommended and the area of the base plate shall be sufficient to limit the stress in bearing for whole of the load. In such cases, the weld or rivet connecting base plate with stiffeners and main member should be designed for 25 per cent of total load coming on base plate (for resisting the unforeseen bending and shear) which should be resisted by total weld length or all rivets.

509.2.2.7. Base plate for eccentrically loaded members - Action due to bending moment in base plate along with compression causes non-uniform pressure on the foundation and value of max. and min. pressure can be computed as under:

$$P_{F\,Max,\,Min} = \frac{N}{BL} \pm \frac{6M_x}{BL^2} \pm \frac{6M_y}{LB^2}$$

where, B & L are width and length of base plate. M_x and M_y are moments in the length and width direction of base plate respectively.

509.2.2.8. Thickness of base plate is computed as per Clauses 509.2.2.3.1 to 509.2.2.3.4 and bending moment is calculated based on maximum pressure acting on each area in which base plate is divided, neglecting non-uniform pressure from foundation on base plate on conservative side.

509.2.2.9. Elements of base plate main and secondary stiffeners are designed as per Clauses 509.2.2.4 and 509.2.2.4.1.

509.3. **Cap Plate**

Cap plate serves as medium for transferring the axial load from structure above (beam, girder) uniformly to the member/stanchion

509.3.1. The thickness of cap plate should be preferably 16-25 mm and stiffener's thickness should not be less than

$$\frac{1}{15} \sqrt{\frac{2600}{F_y}}$$ times width of stiffener, where F_y is the yield stress of stiffener in kg/cm²

509.3.1.2. When the load from beam is transferred to stanchion member through bearing stiffeners extended beyond the beam, the cap plate serves as media to transmit this load to the stiffeners connected to web of stanchion/ member, or to tie beam for lattice member by rivet or weld. The cap plate is designed for specified load.

509.3.1.2.1. If the beam/girder is supported on stanchion in such a manner that loads are directly transmitted to the flange of stanchion or main element or lattice member, cap plate should be provided as per Clause 509.3.1 without calculation

509.3.1.3. The width of cap stiffeners is determined from bearing criteria $b_{sc} = \frac{N}{\sigma_{bg} \cdot tc}$ and also shear stress should not exceed specified stress : $\frac{N}{l_{sc} \cdot t_c} \le \sigma_s$

where,

l_{sc} = Length stiffener, to be sufficient for transmitting the load to web of stanchion by rivet or weld.

σ_{bp} = Basic permissible bearing stress

N = Load to be transmitted from girder/beam

b_{sc} = Width of stiffener

t_c = Thickness of the stiffener.

510. DESIGN OF TENSION MEMBERS

510.1. Design of tension member should generally follow the considerations under Clause 511.4 under "Trusses or Open Web Girders" of this Code.

511. DESIGN OF TRUSSES OR OPEN WEB GIRDERS

511.1. General

Trusses or open web girders are defined as triangulated skeletal **girders.**

For analysis of trusses, the following assumption may be made **unless** rigorous rigid frame analysis is adopted :

(a) All members are frictionless pin jointed.

(b) All members are straight and free to rotate at the joints.

(c) All loads including self weight of members are applied at the joints.

Stipulations made in this section are not applicable for design of **stiffening** trusses of suspension bridges.

511.2. Intersection at Joints

For triangulated trusses designed on the assumption of frictionless pin jointed connections, members meeting at a joint should, where practicable, have their centroidal axis meeting at a point,

and wherever practicable the centre of resistance of a connection shall lie on the line of action of the load so as to avoid an eccentricity moment on the connections.

If, at a joint, the centroidal axis of the adjacent members do not meet at a single point, the resulting flexural stresses in the members should be taken into account as secondary stress.

Where loads are not applied at truss joints, account should be taken of the following :

(a) resulting stresses where load is applied to a member in the plane of a truss other than at a joint.

(b) torsion and lateral flexure effects when the applied load is not in the plane of the truss. Where the load is applied to a cross member, the effect of interaction between the cross-member so loaded and the truss and adjacent cross member should be taken into account.

511.3. Compression Members

511.3.1. **General requirements:** This Clause covers the design of straight members of uniform cross-section subjected to axial compression or to combined compression and bending.

511.3.2. Effective Section

511.3.2.1. The properties of the cross-section should be computed from the effective sectional area. Where plates are provided solely for the purposes of lacing or battening these shall be ignored in computing the radius of gyration of the section.

511.3.2.2. The effective sectional area shall be the gross area less the specified deduction for excessive widths of plate (see Clauses 511.3.2.3 & 511.3.2.4) and the maximum deduction for open holes, including holes for pins and black bolts (see relevant clause of this code) occurring in a section perpendicular to the axis of the member within the critical zone of the compression member.

511.3.2.3. For members other than circular hollow section for calculating the effective cross-sectional area of a member in compression the effective width b_e of a plate, in terms of its width b measured between adjacent lines of rivets, bolts or welds connecting it to other parts of the section, unless effectively stiffened shall be taken as :

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

For b/t not above 45, $b_e = b$

For b/t above 45, $b_e = 45t$

with a maximum value of $b/t = 90$

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

For b/t not above 45, $b_e = b$

For b/t above 45, $b_e = 40t$

with a maximum value of $b/t = 80$

(iii) For welded members (not stress relieved) in mild steel or in high tensile steel

For b/t not above 30, $b_e = b$

For b/t above 30, $b_e = 40t \times [(b/t - 18) / (b/t - 14)]$

with a maximum value $b/t = 80$

In the above, "t" is the thickness of a single plate or the aggregate thickness of two or more plate, provided these are adequately tacked together considering maximum allowable pitch and edge distance of rivets or bolts.

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

(i) 16t for Mild Steel

(ii) 14t for HT Steel

Where, "t" is as defined above. However, in case of compression flanges 't' is the thickness of the thinnest flange plate or the aggregate thickness of two or more plates when the projecting portions of these plates are adequately tacked together.

511.3.3. **Permissible Stress and Slenderness Ratio**

511.3.3.1. **Permissible stress** : Values of permissible stress in axial compression in MPa for some of the structural steels corresponding to various slenderness ratios are given in Table 11.1.

Table 11.1. Permissible Stress σ_{ac} (MPa) in Axial Compression for Steels with various Yield Stresses
(Clause 511.3.3.1)

	Yield stress (f_y) MPa		
$\lambda = l/r$↓	250	340	400
10	150	204	239
20	148	201	235
30	145	194	225
40	139	183	210
50	132	168	190
60	122	152	168
70	112	135	147
80	101	118	127
90	90	103	109
100	80	90	94

110	72	79	82
120	64	69	71
130	57	61	62
140	51	54	55
150	45	48	49
160	41	43	43
170	37	38	39
180	33	34	35
190	30	31	31
200	28	28	28
210	25	26	26
220	23	24	24
230	21	22	22
240	20	20	20
250	18	18	19

where, l = effective length of the member and r = radius of gyration

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

120 for main members, and

140 for wind bracings and subsidiary members.

511.3.3.3. All values of Permissible Stress in Axial Compression in MPa for Structural Steel with Yield Stress other than those shown in Table 11.! may be calculated by using the following formula subject to the condition that s_{ac} shall not exceed $0.6 f_y$

$$\sigma_{ac} = \frac{0.6 f_{cc} x f_y}{[(f_{cc})^n + (f_y)^n]^{1/n}}$$

where,

σ_{ac} = *permissible stress in axial compression, in MPa*

f_y = *yield stress of steel, in MPa*

f_{cc} = *elastic critical stress in compression,* $= \frac{\pi^2 E}{\lambda^2}$

E = *modulus of elasticity of steel;* 2×10^5 *MPa;*

$\lambda (= l/r)$ = *a slenderness ratio of the member, ratio of the effective length to appropriate radius of gyration; and*

n = *a factor assumed as 1.4*

511.3.4. **Lacing and Battening:** The open sides of built up compression members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand for mild steel and 14 times the mean thickness of the outstand for H.T. Steel.

511.3.5. Lacing and battening plates shall be designed in accordance with Clauses 511.3.9 and 511.3.10 and shall be proportioned to resist a total transverse shear force Q at any point in the length of the member equal to at least 2.5 per cent of the axial force in the member together with all shear due to external forces, if any, in the plane of lacing. The shear force Q shall be considered as divided equally among all transverse system and plating in parallel planes.

511.3.6. Compression members composed of two or more components connected as Described in Clauses 511.3.8, 511.3.9 and 511.3.10 may be designed as homogeneous members.

511.3.7. **Effective length of compression members other than lacings**

511.3.7.1. In riveted, bolted or welded trusses, the compression members act in a complex manner and the effective length to be used in computing allowable working stresses for compression members shall be taken as given in Table 11.2 except that, for battened struts, all values given in table shall be increased by 10 per cent.

Table 11.2. Effective Lengh of Compression Members

Member		Effective length l of member: For buckling in the plane of the truss	For buckling normal to the plane of the truss: Compression chord or (compression) member effectively braced by lateral system	For buckling normal to the plane of the truss: Compression chord or (compression) member unbraced
Chords		0.85 x distance between centres of intersection with the web members	0.85 x distance between centres of intersection with the lateral bracing members or rigidly connected cross girders	See Clause 511.3.7.4
Web	Single triangulated system	0.70 x distance between centres of intersection with the main chords	0.85 x distance between centres of intersections	Distance between centres of intersection
Web	Multiple intersection system where adequate connections are provided at all points of intersection	0.85 x greatest distance between centres of any two adjacent intersection	0.70 x distance between centres of intersection with the main chords	0.85x distance between centres of intersection with the main chords

Note :- The intersection referred to are those of the centroidal axis of the members.

511.3.7.2. For single angle discontinuous struts connected to gussets or to a section either by riveting or bolting by not less than two rivets or bolts in line along the angle at each end, or by their equivalent in welding, the eccentricity of the connection with respect to the centroid of the strut may be ignored and the strut designed as an axially loaded member provided that the calculated average stress does not exceed the allowable stresses given in Table 11.1 of Clause 511.3.3 in which "l" is the length of the strut, centre to centre of fastenings at each end and 'r' is the minimum radius of gyration.

511.3.7.3. For single angle discontinuous struts intersected by, and effectively connected to another angle in cross bracing, the effective length in the plane of bracings shall be taken as in Table 11.2 in Clause 511.3.7.1 and normal to the plane of bracing the effective lengths shall be taken as the distance along the bracing members between the points of intersection and the centroids of the main member. In calculating the ratio of slenderness, the radius of gyration about the appropriate rectangular axis shall be taken for buckling normal to the plane of the bracing and the least radius of gyration for buckling in the plane of the bracing.

511.3.7.4. **Effective length of unbraced compression chords:** For simply supported trusses with ends restrained at the bearings against torsion, the effective length l of the compression chord for buckling normal to the plane of the truss shall be taken as follows:

(a) With no lateral support to compression chord; where there is no lateral bracing between compression chords and no cross frames:

l = span

(b) With compression chords supported by U-frames, where there is no lateral bracing of the compression chord but where cross members and verticals forming U-frames provide lateral restraints:

$$l = 2.5 \times \sqrt[4]{(E\, I\, a\, \delta)} \text{ but not less than "}a\text{"}$$

where,

E = Young's modulus

I = maximum moment of inertia of compression chord about the axis lying in the plane of the truss.

a = distance between frames, and

δ = the virtual lateral displacement of the compression chord at the frame nearest to mid span of the truss, taken as the horizontal deflection of the vertical members. This deflection shall be computed assuming that the cross member is free to deflect vertically and that the tangent to the deflection curve at the centre of its span remains parallel to the neutral axis of the unrestrained cross member.

when δ is not greater than $\frac{a^3}{40EI}$

$l = a$

In case of symmetrical U-frames, where cross member and verticals are each of constant moment of inertia throughout their own length; it may be assumed that :

$$\delta = \frac{(d_1)^3 + (d_2)C}{3EI_1\, EI_2}$$

where, d_1 = distance of the centroid of the compression chord from the top of the cross member

d_2 = distance of the centroid of the compression chord from the neutral axis of the cross member

C = half the distance between centres of the main trusses

E = Young's Modulus

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

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

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

$$F = \frac{1.4 \times 100 - 3l}{\delta\left(\frac{C_o}{f_a} - 1.7\right)}$$

In the above formula :

$$l = 2.5 \times \sqrt[4]{(E\, I\, a\, \delta)}$$

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

C_o = Euler critical stress in chord = $\frac{\pi^2 E}{(l/r)^2}$

where, E = Young's Modulus

r = radius of gyration

f_a = calculated working stress in the chord.

(c) With compression chord supporting continuous deck, a compression chord continuously supporting a reinforced concrete or steel deck shall be deemed to be effectively restrained laterally throughout its length (e.g., l = 0) if the friction or positive connection of the deck to the chord is capable of resisting a lateral force, distributed uniformly along its length of 2.5 per cent of the maximum force in the chord, in addition to other lateral forces.

511.3.8. Compression member composed of two components back to back.

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

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

511.3.8.2. Where the members are separated back-to-back the rivets or bolts in these connections shall pass through solid washers or packings, and where the connected angles, legs or tables of tees are 125 mm wide or over or where webs of channels are 150 mm wide or over, not less than two rivets or bolts shall be used in each connection, one on the line of each gauge mark.

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

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

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

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

511.3.9. **Design of lacing of compression members**

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

511.3.9.2. Lacing bars shall be inclined at an angle of 40 to 70 degrees to the axis of the member where a single intersection system is used. and at an angle of 40 to 50 degrees where a double intersection system is used.

511.3.9.3. Except for tie plates as specified in Clause 511.3.9.8, double intersection lacing systems shall not be combined with members of diaphragms perpendicular to the longitudinal axis of the main member, unless all forces resulting from deformation are calculated and provided for in the lacing and its fastenings.

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

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

511.3.9.6. The required section of lacing bars shall be determined by using permissible stresses for compression and tension members given in Clause 511.3.3.1 and Table 11.1. The ratio l/r of the lacing bars shall not exceed 140. For this purpose, the effective length l shall be taken as follows :

(a) **In riveted or bolted construction :** The length between the inner ends of rivets or bolts of the lacing bar in single intersection lacing and 0.7 times this length for double intersection lacing effectively connected at intersections.

(b) **In welded construction :** The distance between the inner ends of effective lengths of welds connecting the bars to the components in single intersection lacings and 0.7 times this length for double intersection lacing effectively connected at intersections.

511.3.9.7. Lacing bars shall be connected to the main members either by riveting or bolting by one or more rivets or bolts. in line along the lacings or by welding at each end sufficient to transmit the load to the bars. Any eccentricity of the connection with respect to the centroid of the lacing bar may be ignored and the lacing designed as an axially loaded strut provided that the calculated average stress does not exceed the allowable stress given in Table 11.1 or s_c. Where welded lacing bars over lap the main component, the amount of lap shall not be less than four times the thickness of the bar or four times the mean thickness of the flange of the component to which the bars are attached, whichever is less . Welding shall be provided at least along each side

of the bar for the full length of the lap and returned along the ends of the bar for a length equal to at least four times the thickness of the bar or width of the bar whichever is less.

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

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

511.3.9.9. The length of end tie plates measured between end fastenings along the longitudinal axis of the member shall not be less than (a) the perpendicular distance between the lines of rivets connecting them to the flanges or (b) the distance between vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts and the length of intermediate tie plates shall be not less than 3/4 of (a) above.

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

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

511.3.9.12. When angles, channels, etc. are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The l/r shall not exceed 140.

511.3.10. **Battening of compression members:** Battened compression members shall comply with the following requirements:

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

511.3.10.2. In battened compression members when the slenderness ratio about the *Y-Y* axis (axis perpendicular to the battens) is not more than 0.8 times the ratio about the *X-X* axis, the spacing of battens between centre-to-centre of end fastenings shall be such that the ratio of slenderness *l/r* of the lesser main component over this distance shall not be greater than 50 or 0.7 times the ratio of slenderness of the member as a whole about its *X-X* axis (axis parallel to the battens).

In battened compression members in when the slenderness ratio about the *Y-Y* axis is more than 0.8 times the ratio about the *X-X* axis, the spacing of battens between centre -to-centre of end fastenings shall be such that the ratio of slenderness *l/r* of the lesser main component over this distance shall not be greater than 50 or 0.7

times the ratio of the slenderness of the member as a whole about its weaker axis.

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

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

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

Q = the transverse shear force as defined in Clause 511.3.5

n = the number of parallel planes of battens

511.3.10.4. The effective length of a batten parallel to the axis of a member shall be taken as the longitudinal distance between the end fastenings. End battens shall have an effective length of not less than (a) the perpendicular distance between the lines of rivets connecting them to the components, or (b) the distance between the vertical side plates of the main chords whichever is greater and shall be at least equal to (c) the depth of the cross girders where these are directly attached to the struts; and intermediate battens shall have an effective length of not less than 3/4 of (a) above, but in no case shall the length (of any batten) be less than twice the width of the smaller component in the plane of the battens.

511.3.10.5. The thickness of batten plates shall not be less than 1/50 of the minimum distance between the innermost lines of connecting rivet, bolts or welds, except when effectively stiffened at the free edges, in which case the minimum thickness may be 8 mm;

for the purpose the edge stiffeners shall have a slenderness ratio not greater than 170.

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

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

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

511.3.10.8. Battened compression member composed of two angles forming a cruciform cross-section shall conform to the above requirements except as follows :

(i) the battens shall be in pairs placed in contact one against the other, unless these are welded to form cruciform battens.

(ii) a transverse shear force $\frac{Q23}{\sqrt{2}}$ shall be taken as occurring separately about each rectangular axis of the whole member.

(iii) a longitudinal shear force of $\frac{QD}{a\sqrt{2}}$ and the moment shall be $\frac{QD}{2\sqrt{2}}$ taken in respect of each batten in each of the two planes, except where the maximum value l/r can occur about a rectangular axis, in which case each batten shall be designed to resist a shear force of 2.5 per cent of the total axial force.

Note : Q, D and a as given in above formula are as defined in Clause 511.3.10.3

511.4. **Tension Members**

511.4.1. **General requirements:** Tension members should preferably be of solid cross-section. However, when composed of two or more components these shall be connected as described in Clauses 511.4.6, 511.4.7 and 511.4.8.

511.4.2. **Effective sectional area:** The properties of the cross section shall be computed from the effective sectional areas as given in Clause 507.8.2. When plates are provided solely for the purposes of lacing or battening, they shall be ignored in computing the radius of gyration of the section.

511.4.3. **Slenderness ratio:** For main members, the ratio of unsupported length to the least radius of gyration shall not exceed 300.

511.4.4. **Lacing and battening:** The open sides of built up compression members of channel or beam sections shall be connected by lacing or battening where the length of the outstand towards the open side exceeds 16 times the mean thickness of the outstand for mild steel and 14 times the mean thickness of the outstand for H.T. Steel.

511.4.5. Lacing and battening shall be designed in accordance with Clauses 511.4.7 and 511.4.8 and shall be proportioned to resist all shear forces due to external forces, if any, in the plane of lacing. The shear shall be considered as divided equally among all transverse systems and plating in parallel planes.

511.4.6. **Tension members composed of two components back-to-back**

511.4.6.1. Tension members formed by sections placed back-to-back, either in contact or separated by a distance not exceeding 50 mm shall be connected together in their length at regular intervals by riveting, bolting or welding so spaced that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause 511.4.3.

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

511.4.6.3. When the components are separated they shall be connected through solid washers or packings, riveted, bolted or welded.

511.4.7. **Design of lacing of tension members**

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

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

511.4.7.3. Except for tie as specified in Clause 511.4.7.7 double intersection lacing systems shall not be combined with members or diaphragms perpendicular to the longitudinal axis of the member, unless all forces resulting from deformation of the member are calculated and provided for in the lacing and its fastenings.

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

511.4.7.5. The required section of the lacing bars shall be determined by using the permissible stresses for compression and tension members given in Clause 511.3.3 and σ_c. The ratio l/r of the lacing shall not exceed 170. For this purpose, the effective length shall be

taken as follows :

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

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

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

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

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

511.4.7.8. The length of end tie plate parallel to the axis of the member shall not be less than (a) the perpendicular distance between the centroids of the main components and shall be at least equal to (b) the depth of the cross girders where these are directly attached to the member, and the length of the intermediate tie plates shall not be less than 3/4 of (a) above.

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

511.4.7.10. When angles, channels, etc. are used instead of end tie plates or are provided where the lacing system is interrupted, these shall be designed by the same method as for battens. The end batten or the intermediate batten and its fastenings shall be capable of carrying the forces for which the lacing has been designed. The l/r shall not exceed 140.

511.4.8. **Battening of tension members:** Battened tension members shall comply with the following requirements :

511.4.8.1. The spacing of battens, measured as the distance between the centres of adjacent end pitches of rivets or bolts or, for welded construction, the clear distance between the battens, shall be such that the maximum ratio of slenderness of each element is not greater than that specified for main members in Clause 511.4.3.

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

End battens shall have an effective length of not less than (a) the perpendicular distance between centroids of the main components and shall be at least equal to (b) the depth of the cross girders where these are directly attached to the members; and the length of the intermediate battens shall have an effective length of not less than one-half of (a) above.

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

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

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

511.5. **Splices**

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

511.5.1.1. Wherever possible both surfaces of the parts spliced shall be covered or other means taken to maintain the alignment of the abutting ends.

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

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

511.5.2. **For tension members**

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

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

511.6. **Connections at Intersection**

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

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

511.6.3. Gusset plates shall be so shaped, and connectors so arranged as to avoid severe stress concentrations.

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

511.7. **Lug Angles**

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

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

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

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

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

511.8. **Section at Pin Holes in Tension Members**

In pin-connected tension members (generally used for erection purpose) the longitudinal net section beyond the pin hole parallel with the axis of the member shall be not less than the required net section of the member. The net section through the pin hole transverse to the axis of the member shall be at least 33 per cent greater than the required net section of the member. In the case of members without stiffened edges the ratio of the net width of the members (through the pin hole transverse to the axis of the member) to its thickness shall not be more than 16. Where the thickness of the main material is not sufficient to resist the load from the pin in bearing, or where the net section through the pinhole requires reinforcement, pin plates (see Clause 511.9) shall be provided and the total thickness shall comply with the above requirements.

511.9. **Pin Plates**

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

511.10. **Diaphragms in Members**

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

511.11. **Lateral Bracings**

511.11.1. All girders shall be provided with a lateral bracing system extending from end-to-end of sufficient strength to transmit the effect of wind, seismic and centrifugal forces, if any to the bearings.

511.11.2. The bracing on the loaded chord shall be so designed as to transmit to the main girders the longitudinal loads due to tractive effort and/or braking effect in order to relieve the cross girders of horizontal bending stress.

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

The floor system may be taken as part of the bracing system provided it is designed for that purpose.

511.11.4. The lateral bracings between compression chords shall be designed to resist a transverse shear at any section equal to 2.5 per cent of the total compressive force carried by both the chords at the section under consideration. This force should be considered in addition to the wind, and centrifugal forces.

511.11.5. **Sway bracings:** Wherever the depth of girder allows, the intermediate cross bracings or sway bracings between vertical web members shall be proportioned to transmit to the chord supported on bearings through the web members at least 50 per cent of the panel lateral load and the vertical members shall be designed to resist the resulting bending moment. The sway bracing so provided shall not be taken as affording any relief to the lateral bracing system or portal system.

511.11.5.1. **Portal bracings :** Through truss spans shall be provided with suitably designed portal system, as deep as the clearance will allow. The portal system shall be designed to take the full end reaction of the top chord lateral system and the end posts of the portal shall be designed to transfer this reaction to the bearings. In addition, the portal system shall be designed to resist a lateral shear equal to $1^1/_4$ per cent of the total compressive force in the end posts or in the top chords in the end panel whichever is greater.

512. CONNECTIONS

512.1. Composite Connections

In any connection which takes a force directly communicated to it and which is made with more than one type of fastening, only rivets and turned and fitted bolts may be considered as acting together to share the load. In all other connections sufficient number of one type of fastening shall be provided to communicate the entire load for which the connection is designed.

512.2. Welded Connections

512.2.1. **Types of welds:** The following types of welds can be used :

(a) Continuous full penetration or partial penetration butt welds

(b) Continuous or intermittent fillet welds

(c) Plug welds

Intermittent butt welds shall not be used.

Partial penetration butt welds shall not be used for transmitting tensile forces or bending moments along longitudinal axis of the welds. Plug welds shall not be used for transmitting loads or moments and shall be used only to prevent the buckling or separation of lapped parts or to joint components of built-up members.

512.2.2. **Strength of weld**

512.2.2.1. **Butt weld**

The strength of a full penetration butt weld shall be taken as equal to the strength of the weaker of the parts joined provided the yield stress of the weld metal is atleast equal to that of the parent metal.

The strength of a partial penetration butt weld together with its reinforcing fillet weld, if any, shall be calculated as for a full penetration

fillet weld. The throat thickness shall be taken as

(a) the depth of weld preparation where this is of the J or U type.

(b) the depth of weld preparation minus 3 mm where the preparation is the V or bevel type

512.2.2.2. **Fillet weld:** The strength of a fillet weld shall be based on the effective throat thickness and the effective length

The effective throat thickness shall be considered as the height of a triangle that can be inscribed within the weld and measured perpendicular to its outer side.

The effective length shall be considered as the actual length minus twice the leg length . In case of fillet welds with end returns as per Clause 512.2.3.1 the effective length shall be considered as the actual length.

512.2.3. **General requiring welds**

512.2.3.1. **Fillet welds:** Maximum leg length of a fillet weld shall be 1 mm less than the thickness of the connected parts at the edge.

Minimum leg length of a fillet weld shall be in accordance with Clause 502.1.8, IS:9595-1980.

Intermittent fillet welds: Intermittent welds shall not be used in tension region of any member, considering their weakness in fatigue. When used in other regions, the undernoted stipulations shall apply.

The clear unconnected gap between the ends of the welds whether in line or staggered shall not be more than 200 mm and also shall not be more than:

(a) 12 times the thickness of the thinner part when the part is in compression

(b) 16 times the thickness of the thinner part when the part is in tension

(c) One-quarter of the distance between stiffeners when used to connect stiffeners to a plate or other part subject to compression or shear

In a line of intermittent welds, there shall be a weld at each end of the part connected.

In built-up members in which plates are connected by intermittent welds, continuous side fillet welds shall be provided at the ends of each side of the plate for a length at least equal to three quarter of the width of the narrower plate concerned. In exceptional cases, where this is not possible, the intermittent plug or slot weld shall be provided to prevent separation.

End returns

The fillet weld shall be returned continuously around the corner at the end of the side of a part for a length beyond the corner of not less than twice the leg length of the weld.

End connections by side fillets

If side fillets alone are used in end connections, both sides of the part shall be welded and the length of the weld on each side shall not be less than the distance between the welds nor less than 4 times the thickness of the thinnest part connected. Where the distance between the welds exceeds 16 times the thickness of thinnest part connected, intermediate plug or slot welds shall be used to prevent separation.

End connections by transverse welds

The overlap between the connected parts shall not be less than four times thickness of the thinnest part and the parts shall be connected by two transverse lines of welds. Where the distance between the weld exceeds 16 times the thickness of the thinnest part connected intermediate slot or plug welds shall be used to prevent separation.

Welds with packings

Where two parts connected by welding are separated by packing having thickness less than the leg length of a weld necessary to transmit the force, the required leg length will be increased by thickness of the packing. The packing shall be trimmed flush with the edge of the part which is to be welded. Where two parts connected by welding are separated by packing having a thickness equal to or greater than the leg length of weld necessary to transmit force, each of the parts shall be connected to the packing by a weld capable of transmitting the design force.

Welds in holes and slots

Fillets welds in holes or slots may be used to transmit shear in lap joints or to prevent the buckling or separation of the lapped parts or to join components of built-up members.

512.2.3.2. **T Butt Joints:** Butt welds in T joints shall be completed by means of fillet welds each having a size of not less than 25 per cent of the thickness of the outstanding part.

512.2.3.3. **Plug welds:** The entire area of the hole or slot shall be filled with weld metal having a thickness:

(a) equal to the thickness of the holed or slotted part where it is 16 mm or less.

(b) In other cases, not less than any of the following :

(i) 16 MM

(ii) 0.45 times the diameter of the hole or the width of the slot.

(iii) One-tenth of the length of slot but not greater than the thickness of the holed or slotted part.

The diameter of the hole or the width of a slot shall not be less than the thickness of the hole or slotted parts plus 8 mm.

The distance between centres of holes or between the centre lines of slots shall not be less than four times the diameter of the hole or the width of the slot. The distance between the centres of the slots measured in the direction of their length shall not be less than double the length of the slot.

The ends of the slot shall be semi-circular except where the slot terminates at the edges of the part where it can be square.

Welding procedure

The welding procedure and details shall be in accordance with IS:9595-1980 unless otherwise stipulated in this chapter.

512.3. **Connections (other than welded)**

512.3.1. **General:** Connections and splices in all members shall be made by the use HSFG bolts, bolts, rivets or other acceptable fasteners. The arrangement of plates, rolled sections and other connecting elements shall be such as to make proper provision for all axial, flexural shear and/or torsional stresses in the members being connected.

Bolted or riveted splices in all compression members shall be located as near as practicable to points of effective lateral support.

A member carrying a calculated stress shall not have a splice or connection with a single rivet or bolt.

Connections and splices for minor members, such as light bracing members, railings etc. may be permitted to have single bolted or riveted connections.

Minimum dia of bolts and rivets used in load bearing members shall be 16 mm diameter.

512.3.2. **Connections and splices in flexural members**

(a) The connection between a flange and a web of a built-up girder shall be designed to transmit the longitudinal shear force in the flange combined with any vertical loads which are directly applied to the flanges.

(b) **Flange splices**

(i) **General:** Flange splices may be made to join flange components made from the same grade of steel but may be of different cross-sections.

(ii) **Bolted or riveted splices:** Where bolted or riveted splice plates are used to obtain a splice in a flange the sum of their areas shall be at least 5 per cent more than the area of the flanges as spliced. The centres of gravity of the sections on either side of the splice shall coincide as far as practicable. The splice connections on each side of the splice shall be capable of transmitting at least the greater of -

(1) 1.10 times the computed force in the flange at the splice point.

(2) 0.80 times the maximum safe force in the weaker flange, calculated from the basic allowable stress, the net section being used for tension flanges and the gross section for compression flanges.

(c) **Web splice:** A splice in the web of a plate girder or rolled section used as a beam shall be designed to resist the shearing forces and the portion of the design moment resisted by the web, and for the moment due to the eccentricity of shear introduced by the splice connection.

Web plates shall be spliced symmetrically by plates on each side. The splice plates shall extend as far as practicable for the full depth of the web. There shall not be less than at least two rows of rivets or bolts on each side of the joint.

512.3.3. Connections in triangulated structures

(a) **Eccentric connections:** Axially stressed members meeting at a joint shall have their gravity axes intersect at a point if practicable; if not, provision shall be made for bending stresses due to the eccentricity.

(b) **Connections at Interconnections:** Connection of members at an intersection shall develop at least 1.10 times the design loads and moments transmitted by the members. Due regard to the nature and distribution of stress over the cross-section of the members shall be given in determining the distribution of the fasteners.

All members shall, where possible, be so connected that the load is appropriately distributed over their cross-section.

If this is impracticable, consideration shall be given to the way in which the stresses at the joint are distributed to those parts of the cross-section of the member which are not directly connected at the joint. For this purpose the angle of distribution of stress may be taken as 45°.

Gusset plates shall be capable of sustaining 1.05 times the design loads and moments transmitted by the members. If an unsupported edge of a gusset plate is in compression and if the length of such edge exceeds

50 times the thickness of the gusset plate, the edge shall be suitably stiffened.

(c) **Splices in tension members and compression members of non bearing type:** Such splices shall be made symmetrical about the gravity axes of the members as far as is practicable.

Bolted riveted splices shall be designed for any applied moment and the greater of-

(i) 1.10 times the computed forces in the member, and

(ii) 0.80 times the safe load in the member calculated from the appropriate basic allowable stress.

The ends of the members need not be in close contact.

(d) **Splices in compression members of bearing type:** In bearing type splices in a compression member, the ends of the members shall be machined and assembled to be in close contact with each other. For a bearing splice it may be assumed that the machined faces transmit 50 per cent of the compressive force in the member. The splice plates and connection shall be however designed to transmit 60 per cent of the compressive force in the member and the applied moment, if any.

Note : Before specifying bearing splices the designer shall, however, satisfy himself that such facilities for machining are available in the particular project.

512.3.4. Rivets and bolts

(a) **Gross and net areas of rivets and bolts:** The gross area of a rivet shall, be taken as the cross-sections area of the rivet hole.

The net sectional area of a bolt or screwed tension rod shall be taken as the tension area for the particular diameter of bolt as given in the table below:

Nominal Thread Dia (mm)	12	14	16	18	20	22	24	27	30	33
Nominal Stress Area (mm^2)	84	115	157	192	245	303	353	459	561	694

(b) Calculation of stresses: Calculation of stresses in rivets and bolts shall be as per Clause 506.5.2.

(c) Diameter of rivet and bolt holes: The diameter holes of a rivet of upto 25 mm nominal diameter shall be taken as 1.5 mm larger than the nominal diameter of the rivet and 2.0 mm larger than the nominal rivet diameter in case of larger diameter rivets.

The diameter of a bolt hole shall be taken as the nominal diameter of the bolt plus 1.5 mm unless otherwise specified.

(d) Edge distances:

(i) In case of rolled, machine flame cut, sawn or plane edges the distance between the centre of the rivet or bolt hole to such edge shall not be less than 1.5 times the diameter of the hole.

(ii) In case of sheared or hand flame cut edges the edge distance shall be 1.75 times the diameter of the hole.

(e) Pitch of rivets or bolts

(i) The minimum distance between the centres of any two adjacent rivets or bolts shall not be less than 2.5 times the nominal diameter of the connector.

(ii) The maximum distance between the centres of any two adjacent rivets or bolts connecting members either in tension or in compression shall not exceed either 32t or 300 mm, where t is thickness of the thinner outside element.

(iii) The distance between centres of two adjacent rivets or bolts in a line along the direction of stress shall not exceed 16t or 200 mm in tension members, and 12t or 200 mm in compression members. In the case of compression members transferring forces through butting faces the pitch shall not exceed 4.5 times the diameter of rivet or bolt from the abutting faces. This pitch will apply for a distance equal to 1.5 times the width of the member.

(iv) When rivets bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distance as specified in (ii) and (iii) above between centre of adjacent connectors may be increased by 50 per cent.

(f) **Long rivets:** The grip of rivets carrying calculated loads shall not exceed 8 times the diameter of the holes. Where the grip exceeds 6 times the diameter of the hole, the number of rivets required by normal calculation shall be increased by not less than half a per cent for each additional millimeter of length of grip above 6 times the hole diameter.

(g) **Rivets with counter sunk head:** For counter sunk rivets, half of the depth of the counter sinking shall be neglected in calculating the length of the rivet in bearing. As far as possible rivets in tension shall be avoided. However, when rivets with contour sunk heads are in tension, the tensile value of the rivets shall be reduced by 33-1/3 per cent. No reduction need in shear.

(h) **Rivets or bolts through packing**

Number of rivets or bolts carrying calculated shear through a packing shall be increased above the number required by normal calculations by 2.5 per cent for each 2.0 mm thickness of packing except that, for packing having a thickness of upto 6 mm, no increase need be made. For double shear connections packed on both sides, the number of additional rivets or bolts required shall be determined from the thickness of the thicker packing. The additional rivets or bolts shall be placed in an extension of the packing.

(i) **Staggered pitch:** When rivets and bolts are staggered at equal intervals and the gauge does not exceed 75 mm, the distances between centres of rivets and bolts as specified earlier may be increased by 50 per cent.

513. FABRICATION AND INSPECTION

513.1. General

All work shall be in accordance with the drawings and clauses of this code unless otherwise agreed.

513.2. Laminations in Plates

The following areas of plate shall not have laminations exceeding the prescribed limits :

(a) Steel plate and sections in which tension stresses are transmitted through thickness of plate or in region in which lamination could affect the buckling behaviour under compression and bending compression.

(b) On each side of welded bearing diaphragm, strip of flange and web plate having width equal to 25 times of their thickness.

(c) The strip of web plate having 25 times thickness on each side of single sided bearing stiffener welded to web.

(d) For welded cruciform joints transmitting tensile stress through the plate thickness on strip having width four times the thickness of plate on each side of attachment.

(e) For edges of plates where corner welds are provided on to the surface of such plates.

Other areas of plate, section specified by the Engineer shall not have lamination exceeding prescribed limits.

513.3. Storage of Materials

All material, consumable, including raw steel or fabricated material shall be stored specificationwise and sizewise above the ground

upon platforms, skids or other supports. These shall be kept free from dirt and other foreign matter and shall be protected from corrosion and distortion. The electrodes shall be stored specificationwise and shall be kept in dry warm condition in properly designed racks. The bolts, nuts, washers and other fasteners shall be stored on racks above the ground with protective oil coating in gunny bags. The paint shall be stored under cover in air tight containers.

513.4. Straightening, Bending and Pressing

513.4.1. Straightening and flattening of steel shall be done by methods that will not injure the metal. Hammering shall not be permitted.

Straightening by heating shall be done under controlled procedure. Temperature of the steel shall not be more than 650°C or the temperature timing and cooling rate shall be appropriate to the particular type of steel and shall be agreed by the authorities. Accelerated cooling shall not be used without the approval of Engineer.

513.4.2. **Bending and curving**

513.4.2.1. Steel having yield stress more than 360 MPa shall not be heat curved.

513.4.2.2. Heating procedure - Rolled beams and girders may be curved by either continuous or V-type heating as approved by Engineer.

(a) For the continuous method, a strip of sufficient width along the edge of top and bottom flange shall be heated simultaneously to desired temperature to obtain required curve.

(b) For V-type of heating, the top and bottom flanges shall be heated in truncated triangular or wedge-shaped areas having their base along the flange edge and spaced at regular intervals along each flange. The truncated triangular pattern shall have an angle 15 to 30 degrees with base not more than 250 mm. The spacing and temperature shall be as required to obtain the required curvature and heating shall be at approximately same rate along the top and bottom flange.

For flange thickness of 32 mm or more, both inside and outside surfaces shall be heated concurrently.

513.4.2.3. **Temperature:** The heat bending shall be conducted so that the temperature of steel does not exceed 620°C. The girder shall not be artificially cooled until temperature comes down to 315°C by natural cooling. The method of artificial cooling has to be approved by Engineer.

513.4.2.4. **Camber:** Camber for rolled beams may be obtained by heat curving methods approved by Engineer. For camber in plate girders, the web shall be cut to prescribed camber with suitable allowance for shrinkage due to cutting, welding and heat curving

513.5. Workmanship

513.5.1. Fabricator has to submit a Quality Assurance Plan according to the nature of fabrication work, such as, welded fabrication or riveted fabrication and the same should be approved by the client. Quality Assurance Plan should elaborate Nodal point checking and inspection during the stage of fabrication and also the materials.

513.5.2. Fabrication work shall be taken up only after receipt of approved fabrication/working drawing.

513.5.3. All members shall carry mark number and item number and, if required, serial number. Method of marking shall commensurate with the process of manufacture and shall ensure retention of identity at all stages.

513.5.4. **Preparation of edges ends and surfaces:** Material shall be cleaned and any burring, scales or abnormal irregularities shall be removed.

513.5.4.1. **Edge and end planing/cutting:** End/edge planing and cutting shall be done by any one of the following prescribed methods or left as rolled.

(a) Shearing, cropping, sawing, machining, machine flame cutting.

(b) Hand flame cutting with subsequent grinding to a smooth edge.

(c) Sheared edges of plate not more than 16 mm thick with subsequent grinding to smooth profile, which are for secondary use such as stiffeners and gussets.

If ends of stiffeners are required to be fitted they shall be ground so that the maximum gap over 60 per cent of the contact area does not exceed 0.25 mm.

513.5.4.2. **Shearing and flame cutting:** Where flame cutting or shearing is used as specified in Clause 513.4.1 atleast one of the following requirements shall be satisfied.

(a) The cut edge is not subjected to applied stress.

(b) The edge is incorporated in weld.

(c) The hardness of cut edge does not exceed 350 HV 30.

(d) The material is removed from edge to the extend of 2 mm or minimum necessary, so that hardness is less than 350 HV 30.

(e) Edge is suitably heat-treated by approved method to the satisfaction of Engineer and shown that cracks had not developed by dye penetrant or magnetic particle test.

(f) Thickness of plate is less than 40 mm for machine flame cutting for materials conforming to IS:2062. The requirement of hardness below 350 HV 30 of flame cut edges should be specified by Engineer. Wherever specified by the Engineer the flame cut edges shall be ground or machined over and above requirement (a) to (f).

513.5.4.3. Where machining for edge preparation in butt joint is specified, the ends shall be machined after the members have been fabricated.

513.5.4.4. Outside edges of plate and section, which are prone to corrosion shall be smoothened by grinding or filling.

513.5.5. **Rivet and bolt holes**

513.5.5.1. Holes for rivets, black bolts, high strength bolts and countersunk bolts/rivets (Excluding close tolerance and turn fitted bolts): All holes for rivets or bolts shall be either punched or drilled. The diameter of holes shall be 1.5 mm larger for bolts/rivets upto 25 mm dia and 2 mm for more than or equal to 25 mm.

All holes shall be drilled except for secondary members such as, floor plate, handrails etc., and members which do not carry the main load can be punched subject to the thickness of member does not exceed 12 mm for material conforming to IS:2062.

Holes through more than one thickness of material or when any of the main material thickness exceeds 20mm for steel to IS:2062 or 16 mm to IS:8500, IS:8500 shall either be subdrilled or subpunched, less than 3 mm diameter than required size and reamed to full diameter. The reaming of material more than one thickness shall be done after assembly.

513.5.5.2. Holes for close tolerance and turn fitted bolts. The diameter of the holes shall be equal to - 0.15 mm to - 0.0 mm, of the bolt shank.

The members to be connected with close tolerance or turn fitted bolts shall be firmly held together by service bolts or clamped and drilled through all thickness in one operation and subsequently reamed to required size within specified limit of accuracy as specified in IS:919 tolerance grade H8.

The holes not drilled through all thicknesses at one operation shall be drilled to smaller size and reamed after assembly.

513.5.5.3. Holes for high strength friction grip bolts - All holes shall be drilled after removal of burrs. Where the number of plies in the grip does not exceed three, the diameters of holes shall be 1.5 mm larger than those of bolts and for more than three plies in grip, the diameter of hole in outer plies shall be as above and diameter of holes in inner plies shall be not less than 1.5 mm and not more than 3 mm larger than those in bolts, unless otherwise specified by Engineer.

513.5.6. **Bolted construction**

513.5.6.1. All joint surface for bolted connection including bolts, nuts, washers, shall be free of scale, dirts, burrs other foreign material and other defects that would prevent solid seating of parts. The slope of surface of bolted parts in contact with bolt head and nuts shall not exceed 1/20, plane normal to bolts axis, otherwise suitable tapered washer shall be used.

All fasteners shall have a washer under nut or bolt head, whichever is turned in tightening.

Each fastener of joint shall be tightened to specified value or equal to 70 per cent of specified minimum tensile strength by hand wrenches (turn of nut method) or calibrated wrenches or manual torque wrenches. Impact wrench or any other method specified by Engineer.

When turn of nut method is used for tightening the bolts in joint first all bolts shall be brought to "snug tight" condition, that is tightening by full effort of man using ordinary wrench or by few impacts of any impact wrench. All bolts in the joint shall be then tightened additionally by applicable amount of nut rotation specified below for guidance :

Bolt length (from underside of head to edge)	Disposition of outer faces of bolted parts	
	Bolt face normal to bolt axis	One face Normal to bolt axis and other face slopped less than 1:20
Upto and including 4 dia	1/3 turn	1/2
Over 4 dia but less than 8 dia	1/2 turn	2/3
Over 8 dia but less than12dia	2/3 turn	5/6

513.5.6.2. **High strength friction grip bolts and bolted connections:** The general requirement shall be as per relevant IS specifications mentioned in Clause 505.3 (Fasteners) of this code. Unless otherwise specified by the Engineer, bolted connections of structural joints using high tensile friction grip bolts shall comply with requirements mentioned in IS:4000-1992.

513.5.6.3. **Riveted construction**

513.5.6.4. Assembled riveted joint surfaces including those adjacent to the rivet head shall be free of scale, dirt, loose scale, burrs, other foreign material and defects that would prevent solid seating of parts.

513.5.6.5. The part/members to be riveted shall be firmly drawn together with bolts, clamps or tack weld. Every third hole of the joint shall have assembly bolts till riveted. Drift shall be used only for matching of holes of the parts/members, but not to the extent as to distort the holes. Drift of larger size than the normal diameter of the holes shall not be used.

513.5.6.6. Rivets shall be heated uniformly to a "light cherry red colour" between 650°C to 700°C for hydraulic riveting and "Orange colour" for pneumatic riveting of mild steel rivets. High tensile steel rivets shall be heated upto 1100°C. Any rivet whose point is heated more than prescribed shall not be driven.

513.5.6.7. Rivet shall be driven in hole when hot so as to fill the hole as completely as possible and shall be of sufficient length to form a head of the standard dimension. When countersunk head is required the head shall fill the countersunk hole. Projection after countersinking shall be ground off wherever necessary.

513.5.6.8. The riveting shall be done by hydraulic or pneumatic machine unless otherwise specified by the Engineer.

513.5.6.9. Any defective rivet due to defect in head size or head driven off the centre shall be removed and replaced.

513.5.6.10. The parts not completely riveted in the shop shall be secured by bolts to prevent damage during transport and handling.

513.5.7. **Welded construction**

513.5.7.1. Surfaces and edges to be welded shall be smooth, uniform and free from fins, tears, cracks and other discontinuities. Surface shall also be free from loose or thick scale, slag rust, moisture oil and other foreign materials.

513.5.7.2. The general welding procedures including particulars of the preparation of fusion faces for metal arc welding shall be carried out in accordance with IS:9595.

513.5.7.3. The welding procedures for shop and site welds including edge preparation of fusion faces shall be submitted in writing in accordance with Clause 22 of IS:9595 for the approval of the Engineer before commencing fabrication, and shall also be as per details shown on drawings. Any deviation for above has to be approved by the Engineer.

513.5.7.4. Electrodes to be used for metal arc welding shall comply with relevant IS specifications mentioned in Clause 505.3 of this code. Procedure test shall be carried out as per IS:8613 to find out suitable wire-flux combination for welded joint.

513.5.7.5. Assembly of parts for welding shall be in accordance with Clauses 14 to 16 of IS:9595.

513.5.7.6. The welded temporary attachment should be avoided as far as possible, otherwise the method of making any temporary attachment shall be approved by Engineer. Any scars from temporary attachment shall be removed by cutting, chipping and surface shall be finished smooth by grinding to the satisfaction of Engineer.

513.5.7.7. For welding of any particular type of joint, welders shall qualify to the satisfaction of Engineer in accordance with appropriate welders qualification test as prescribed in any of the Indian standards IS:817-1966, IS:1393-1961, IS:7307 (Part-I)-1974, IS:7310 (Part-I)-1974 and IS:7318 (Part I)-1974 as relevant.

513.5.7.8. In assembling and joining parts of a structure or of built-up members, the procedure and sequence of welding shall be such as to avoid distortion and minimise shrinkage stress.

513.5.7.9. All requirements regarding preheating of present material and interpass temperature shall be in accordance with provisions of IS:9595.

513.5.7.10. Peening of weld shall be carried out wherever specified by the Engineer.

(a) If specified, peening may be employed to be effective on each weld layer except first.

(b) The peening should be carried out after weld has cooled by light blows from a power hammer, using a round nose tool.

Care shall be taken to prevent scaling or flecking of weld and base metal from over peening.

513.5.7.11. Where the Engineer has specified the butt welds are to be ground flush, the loss of parent metal shall not be greater than that allowed for minor surface defects.

513.5.7.12. The joints and welds listed in are prohibited type, which do not perform well under cyclic loading:

(a) Butt joints not fully welded throughout their cross section.

(b) Groove welds made from one side only without any backing strip.

(c) Intermittent groove welds.

(d) Intermittent fillet welds.

(e) Bevel-grooves and J-grooves in butt joints for other than horizontal position.

(f) Plug and slot welds.

513.5.7.13. The run-on and run-off plate, extension shall be used providing full throat thickness at the end of butt welded joints. These plates shall comply with following requirements.

(i) One pair of "run-on" and one pair of "run-off" plates prepared from same thickness and profile as the parent metal shall be attached to start and finish of all butt welds preferably by clamps.

(ii) When "run-on" and "run-off" plates shall be removed by flame cutting, it should be cut at more than 3 mm from parent metal and remaining metal shall be removed by grinding or by any other method approved by the Engineer.

513.5.7.14. **Welding of stud shear connectors:**

(a) The stud shear connectors shall be welded in accordance with the manufacturer's instructions including preheating.

(b) The stud and the surface to which studs are welded shall be free from scale, moisture, rust and other foreign material. The stud base shall not be painted, galvanised or cadmium-plated prior to welding.

(c) Welding shall not be carried out when temperature is below 0°C or surface is wet.

(d) The welds shall be visually free from cracks and lack of fusion and shall be capable of developing atleast the nominal ultimate strength of studs.

(e) The procedural trial for welding the stud shall be carried out when specified by the Engineer.

513.5.8. **Annealing and stress relieving :** The members which are indicated in the contract or specified by Engineer, to be annealed or stress relieved shall have finish machining, boring etc. done subsequent to heat treatment. The stress relief treatment shall conform to the following unless specified by the Engineer.

(a) The temperature of the furnace shall not be more than 300°C at the time welded assembly is placed in.

(b) The rate of heating shall not be more than 220°C per hour divided by max. metal thickness subject to max. 220°C per hour.

(c) After max. temperature of 600°C is reached, the assembly shall be held within specified limit of time based on weld thickness. The temperature shall be maintained uniformly throughout the furnace during holding period such that temperature at no two points on the member will differ by more than 80°C.

(d) The cooling shall be done in closed furnace when temperature is above 300°C at the max. rate of 260°C per hour divided by max. metal thickness. The local stress relieving shall be carried out if specified and procedure approved by the Engineer.

513.5.9. **Pins and pin holes:** The pins shall be of required length, parallel throughout and of smooth surface free from flaws. The pin holes shall be bored smooth, straight and true to gauge and right angles to the axis of the member. Boring shall be done only after member is finally riveted, bolted or welded unless otherwise approved by the Engineer. To facilitate insertion and extraction, pins may be chamfered beyond the required length and provided with suitable holes in the chamfered portion.

513.5.10. **Rectification of surface defects and edge laminations:** The surface defects revealed during fabrication or cleaning shall be repaired as specified. The repair by welding on any surface defect or exposed edge lamination shall be carried out only with approval of the Engineer.

513.5.11. **Shop assembly:** The steel work shall be temporarily assembled at place of fabrication. Assembly shall be full truss or girder, unless progressive truss or girder assembly, full chord assembly, progressive chord assembly or special, complete structure assembly is specified by the Engineer.

The field connections of main members of trusses, arches, continuous beam spans, bents, plate girders and rigid frame assembled, aligned, accuracy of holes, camber shall be checked by the Engineer and then only reaming of subsize holes to specified size shall be taken up.

The assembly will be dismantled after final drilling of holes and approval of the Engineer.

The camber diagram showing camber at each panel point, and method of shop assembly and any other relevant detail shall be submitted to the Engineer for approval.

513.5.12. **Fabrication tolerances**

513.5.12.1. In general all parts in an assembly shall fit together accurately within tolerances specified in Table 13.1, unless otherwise specified by the Engineer and agreed in contract.

Table 13.1. Fabrication Tolerances

(Clause 513.5.12)

Individual Components :

(1) Length :			
	(a)	Member with both ends finished for contact bearing	± 1 mm
	(b)	Individual components of members with end plate connection	+ 0 mm - 2 mm
	(c)	Other members -	
		(i) Upto and including 12 m	± 2 mm
		(ii) Over 12 m	± 3.5 mm
(2) Width :			
	(a)	Width of built-up girders	± 3 mm
	(b)	Deviation in the width of members required to be inserted in other members	0 mm - 3mm
(3) Depth :		Deviation in the depths of solid web and open web girders	3 mm -2 mm
(4) Straightness:			
	(a)	Deviation from straightness of columns to a maximum of 15 mm	L/300 subject to
		(i) In elevation	+ 5 mm
		(ii) In plan	- 0 mm
(5)		Deviation of centre line of web from centre line of flanges in built-up members at contact surfaces	3 mm
(6)		Deviation from flatness of plate webs of built-up members in a length equal to the depth of the member	0.005 d to a max. 2 mm
(7) Tilt of flange of plate girders			
	(a)	at splices and stiffeners, **at supports,** at the top flanges of crane girders, at bearings.	**0.005 b to a max. of 2 mm**
	(b)	at other places	0.015 b to a maximum of 4 mm
(8)		Deviation from squareness of flange to web of columns and box girders	L/1000, where L is the nominal length of the diagonal
(9)		Deviation from squareness of fixed base plate (not machined) to axis of column. This dimension shall be measured parallel to the longitudinal axis of the column at points where the outer surfaces of the column sections make contact with the base plate.	D/500, where D is the distance from the column axis to the point under consideration on the base plate.
(10)		Deviation from squareness of machined ends to axis of columns	D/1000, where D is as defined in 9 above.
(11)		Deviation from squareness of machined ends to axis of beam or girders	D/1000, where D is as defined in 9 above.
(12)		Ends of members abutting at joints through cleats or end plates, permissible deviation from squareness of ends.	1/600 of depth of member subject to a max. of 1.5 mm.

513.5.12.2. A machined bearing surface, where specified by the Engineer, shall be machined within a deviation of 0.25 mm for surfaces that can be inscribed within a square of side 0.5 m.

513.5.13. **Alignment at splice and butt joints**

513.5.13.1. Bolted splice shall be provided with steel packing plates where necessary to ensure that the sum of any unintended steps between adjacent surfaces does not exceed 1 mm for HSFG bolted joints and 2 mm for other joints.

513.5.13.2. In welded butt joints, mis-alignment of parts to be joined shall not exceed the lesser of 0.15 times the thickness of thinner parts or 3 mm. However, if due either to different thicknesses arising from rolling tolerances or a combination of rolling tolerances with above permitted mis-alignment, this deviation is more than 3 mm, it shall be smoothened by a slope not steeper than 1:4.

513.6. **Inspection and Testing**

513.6.1. **General :** No protective treatment shall be applied to the work until the appropriate inspection and testing has been carried out. The stage inspection shall be carried out for all operations so as to ensure the correctness of fabrication and good quality.

513.6.2. **Testing of Material**

513.6.2.1. Structural steel shall be tested for mechanical and chemical properties as per various IS as may be applicable and shall conform to requirements specified in IS:2062-1984, IS:11587-1986, IS:1977-1973, IS:8500-1977, IS:961-1975, etc.

513.6.2.2. Rivets, bolts, nuts, washers, welding consumables, steel forging, casting and stainless steel shall be tested for mechanical and chemical properties as applicable and shall conform to requirements, as specified in the appropriate Indian Standard.

513.6.3. Rolling and cutting tolerance shall be as per IS:1852. The thickness tolerance check measurements for the plates and rolled sections shall be taken at not less than 15 mm from edge.

513.6.4. Laminations in plates shall be carried out for areas specified in Clause 513.2 by ultrasonic testing or any other specified methods. Flame cut edges without visual signs of laminations need not be tested for compliance with Clause 513.2 unless specified otherwise by the Engineer.

513.6.5. Steelwork shall be inspected for surface defects and exposed edge laminations during fabrication and blast cleaning. Significant edge laminations found shall be reported to the Engineer for his decision.

Chipping, grinding, machining or ultrasonic testing shall be used to determine depth of imperfection.

For dynamically loaded structures recommended criteria for allowable discontinuities for edge defects and the repair procedure shall be as given in Table 13.2, until and unless specified otherwise. The weld procedure shall be as appropriate to the material.

Table 13.2. Discontinuity of Edge

	Discontinuity	Repairs required
1.	Discontinuities of 3 mm in maximum depth, any length for material thickness upto 200 mm	None
2.	Discontinuities of 3 mm to 6 mm in depth and over 25 mm in length for thickness upto 100 mm and 6 mm to 12 mm depth, over 50 mm in length for thickness 100 mm to 200 mm	Remove, need not be welded
3.	Discontinuities of 6 mm to 25 mm in depth, over 25 mm in length for thickness upto 100 mm and 12 mm to 25 mm in depth, over 25 mm in length for thickness over 100 to 200 mm	Remove and weld. No single repair shall exceed 20 per cent of edge being repaired
4.	Discontinuities over 25 mm in depth, any length for thickness 100 to 200 mm	With approval of Engineer remove to depth of 25 mm and repair by weld block
5.	On edges cut in fabrication, discontinuities of 12 mm maximum depth any length	None

513.6.6. **Measurement of curvature and camber:** Horizontal curvature and vertical camber shall not be measured for final acceptance before all welding and heating operations are completed and flanges have cooled to uniform temperature. Horizontal curvature shall be checked with girder in the vertical position by measuring offsets from a string line or wire attached to both flanges or by any other suitable means. Camber shall be checked by adequate means.

513.6.7. **Tolerance for drilled and reamed holes:** Acceptable deviation in holes drilled and reamed for mild steel and high strength rivets, bolts of normal accuracy and also for high strength friction grip bolts should be as per appropriate Indian Standard.

513.6.8. **Bolted connections**

513.6.8.1. Bolted connection joints with black bolts and high strength bolts shall be inspected for compliance of requirements mentioned in Clause 513.5.5.

The Engineer shall observe the installation and tightening of bolts so as correct tightening procedure is used and shall determine that all bolts are tightened. Regardless of tightening method used, tightening of bolts in a joint should commence at the most rigidly fixed or stiffest point and progress towards the free edges, both in initial snuggling and in final tightening.

The tightness of bolts in connection shall be checked by inspection wrench. which can be torque wrench, power wrench or calibrated wrench.

Tightness of 10 per cent bolts, but not less than two bolts, selected at random in each connection shall be checked by applying inspection torque. If no nut or bolt head is turned by this application connection can be accepted as properly tightened, but if any nut or head has turned all bolts shall be checked and if necessary retightened.

513.6.8.2. Bolts, and bolted connection joints with high strength friction grip bolts shall be inspected and tested according to IS:4000-1967.

513.6.9. Rivets and riveted connection shall be inspected and tested for compliance or requirements mentioned in Clause 513.5.6.

The firmness of the joint shall be checked by 0.2 mm filler gauge, which shall not go inside under the rivet head by more than 3 mm. There shall not be any gap between members to be riveted.

Driven rivets shall be checked with rivet testing hammer. When struck sharply on head with rivet testing hammer, rivet shall be free from movement and vibration.

All loose rivets and rivets with cracked, badly formed or deficient heads or with heads which are unduly eccentric with shanks, shall be cut out and replaced.

513.6.10. **Alignment of joints:** The alignment of plates at all bolted splice joint and welded butt joints shall be checked for compliance with requirements of Clause 513.5.13.

513.6.11. Testing of flame cut and sheared edges is to be done where the hardness criteria of Clause 513.5.4.2 (a) to (d) are adopted. Hardness testing shall be carried out on six specimens.

513.6.12. **Welding and welding connection**

513.6.12.1. Welders qualification test shall be carried out as per requirements laid down in IS:7318 (Part I), for respective approved welding procedure, they shall satisfy relevant requirements of IS:7310 (Part-I)-1970.

Welding procedure, welded connection and testing shall be in compliance of requirements mentioned in Clause 513.5.7.

513.6.12.2. All facilities necessary for stage inspection during welding and on completion shall be provided to the Engineer or their inspecting authority by manufacturer.

513.6.12.3. Adequate means of identification either by an identification mark or other record shall be provided to enable each weld to be traced to the welder(s) by whom it was carried out.

513.6.12.4. All metal arc welding shall be in compliance with IS:9595 provisions.

513.6.12.5. The method of inspection shall be according to IS:822-1970 and extent of inspection and testing shall be in accordance with the relevant standards or in the absence of such a standard, as agreed with the Engineer.

513.6.12.6. **Procedure tests:** The Destructive and Non-Destructive test of weld shall be carried out according to IS:7307 (Part I).

513.6.12.7. **Non-Destructive testing of welds:** One or more of following methods may be applied for inspection or testing of weld.

513.6.12.7.1. **Visual inspection:** All welds shall be visually inspected, which should cover all defects of weld, such as, size, porosity, crack in the weld or in the HAZ (Heat affected zone), etc. Suitable magnifying glass may be used for visual inspection. A weld shall be acceptable by visual inspection if it shows that :

(a) The weld has no crack.

(b) Through fusion exist between weld and base metal and between adjacent layers of weld metal.

(c) Weld profiles are in accordance with requisite clauses of IS:9595 or as agreed with the Engineer.

(d) The weld shall be of full cross-section, except for the ends of intermittent fillet welds out side of their effective length.

(e) When weld in transverse to the primary stress, under-cut shall not be more than 0.25 mm deep in the part that is under-cut and shall not be more than 0.8 mm deep when the weld is parallel to the primary stress in the part that is undercut.

(f) The fillet weld in any single continuous weld shall be permitted to underrun the nominal fillet weld size specified by 1.6 mm without correction provided that under-size portion of the weld does not exceed 10 per cent of the length of the weld. On the web-to-flange welds on girders, no under-run is permitted at the ends for a length equal to twice the width of the flange.

(g) The piping porosity in fillet welds shall not exceed one in each 100 mm of weld length and the maximum diameter shall not exceed 2.4 mm, except for fillet welds connecting stiffeners to web where the sum of diameters of piping porosity shall not exceed 9.5 mm in any 25 mm length of weld and shall not exceed 19 mm in any 300 mm length of weld.

(h) The full penetration groove weld in butt joints transverse to the direction of computed tensile stress shall have no piping porosity. For all other groove welds, the piping porosity shall not exceed one in 100 mm of length and the maximum diameter shall not exceed 2.4 mm.

513.6.12.7.2. **Magnetic particle and radiographic inspection:** Welds that are subject to radiographic or magnetic particle testing in addition to visual inspection shall have no crack.

Magnetic particle test shall be carried out for detection of crack and other discontinuity in the weld according to IS:5334.

Radiographic test shall be carried out for detection of internal laws in the weld, such as, crack, piping porosity, inclusion, lack of fusion, incomplete penetration etc. This test may be carried out as per IS: 1182 and IS: 4853.

Acceptance criteria:

The weld shall be unacceptable if radiograph or magnetic particle testing shows any of the type discontinuities listed hereunder, unless agreed by the Engineer.

(a) For welds subjected to tensile stress, the greatest dimension of any porosity or fusion type discontinuity that is 1.6 mm or larger in greatest dimension shall not exceed the size, "B", for the effective throat or weld size. The distance from any porosity or fusion type discontinuity described above to another such discontinuity to an edge, or to the toe or root of any intersecting flange to web shall not be less than the minimum clearance allowed C, for the size of discontinuity under examination.

(b) For welds subjected to compressive stress only, the greatest dimension of porosity or a fusion type discontinuity that is 3.2 mm or larger in greatest dimension shall not exceed the size, B, nor shall the space between adjacent discontinuities be less than the minimum clearance allowed, C, for the size of discontinuity under examination.

(c) The discontinuities having greatest dimension of less than 1.6 mm shall be unacceptable, if the sum of their greatest dimension exceed 9.5 mm in any 25 mm length

of weld, over and above requirements mentioned in (a) and (b) above.

(d) The limitations for 38.1 mm effective throat shall be applicable to all effective throats greater than 38.1 mm thickness.

(e) For welded bridge girders structural steel to IS:2062 and IS: 8500 only shall be used except for secondary members, such as, bracings, etc.

513.6.12.7.3. **Ultrasonic inspection:** The ultrasonic testing in addition to visual inspection shall be carried out for detection of internal flaws in the weld, such as, cracks, piping porosity inclusion, lack of fusion, incomplete penetration, etc. Acceptance criteria as per IS:4260 or any other relevant IS Specification and as agreed by the Engineer.

513.6.12.7.4. **Liquid penetrant inspection:** The liquid penetrant test shall be carried out for detection of surface defect in the weld, as per IS:3658, in addition to visual inspection.

513.6.12.7.5. The non-destructive testing of following welds be carried out using one of the methods or methods described in Clause 513.6.12.7.2 to 4 as may be agreed by the Engineer.

(a) All transverse butt weld in tension flange.

(b) 10 per cent of length of longitudinal butt welds in tension flange

(c) 5 per cent of the length of longitudinal and transverse butt welds in compression flanges

(d) All transverse butt welds in webs adjacent to tension flanges as specified by the Engineer.

The particular length of welds to be tested shall be agreed with the Engineer, in case of (b) and (c).

Where specified by the Engineer, bearing stiffeners or bearing diaphragms adjacent to welds, flange plates adjacent to web/flange welds, plates at cruciform welds, plates in box girder construction adjacent to corner welds or other details shall be ultrasonically tested after fabrication.

Any lamination, lamellar tearing or other defect found shall be recorded and reported to the Engineer for his decision.

513.6.12.8. **Testing of welding for cast steel:** The testing of weld for cast steel shall be carried out as may be agreed by the Engineer.

513.6.12.9. **Stud shear connectors:** Stud shear connectors shall be subject to the following tests :-

(a) The fixing of studs after being welded in position shall be tested by striking the side of the head of the stud with 2 kg hammer, to the satisfaction of the Engineer.

(b) The selected stud head stroked with 6 kg hammer shall be capable of lateral displacement of approximately 0.25 height of the stud from its original position. The stud weld shall not show any signs of cracks or lack of fusion.

The studs whose welds have failed the tests given in (a) and (b) shall be replaced.

513.6.12.10. **Inspection of members and components**

513.6.12.10.1. **Inspection requirement:** The fabricated member/ component made out of rolled and built-up section shall be checked for compliance of the tolerances given in Table 13.1. Inspection of member/components for compliance with tolerances, the check for deviations shall be made over the full length.

During checking the inspection requirement shall be placed in such a manner that local surface irregularities do not influence the results.

For plate, out-of-plane deviation shall be checked at right angle to the surface over the full area of plate.

The relative cross girder or cross frame deviation shall be checked over the middle third of length of cross girder or frame between each pair of webs and for cantilever at the end of member.

The web of rolled beam or channel section shall be checked for out-of-plane deviation in longitudinal direction equal to the depth of the section.

During inspection, the component/member shall not have any load or external restraint.

513.6.12.10.2. **Inspection stages:** The inspection to be carried out for compliance of tolerances shall include but not be limited to the following stages:

(a) For completed parts, component/members on completion of fabrication and before any subsequent operation, such as, surface preparation, painting transportation, erection.

(b) For webs of plate and box girder, longitudinal compression flange stiffeners in box girders, and orthotropic decks and all web stiffeners at site joints, on completion of site joint.

(c) For cross girders and frames, cantilevers in orthotropic decks and other parts in which deviations have apparently increased on completion of site assembly.

513.6.12.10.3. Where, on checking member/component for the deviations in respect of out-of-plane or out-of-straightness at right angles to the plate surface, and any other instances, exceed tolerance, the maximum deviation shall be measured and recorded. The recorded measurements shall be submitted to the Engineer, who will determine whether the component/member may be accepted without rectification, with rectification, or rejected.

514. HANDLING, TRANSPORTATION AND ERECTION

514.1. Scope

514.1.1. This clause lays down guidelines of general nature for handling, transportation and erection of bridges and their components.

514.1.2. It deals with the action to be taken for various operations in handling, transportation in shop floor and in transit as also in the erection site.

514.2. Transportation and Handling

514.2.1. The Engineer should plan the transportation and mention the mode of transportation, packing, placement, fastening of components or materials to ensure carriage free from damages or undue distortion. When deciding the mode, the route should be surveyed and local restriction in terms DO's/DON'T's statement for proper handling/transportation to be issued.

514.2.2. All transportable consignments should carry dispatch advice/challan as per directions to party concerned. Depending on "Target Factor" requirement of materials to be adjusted.

514.2.3. Loose assembled or sub-assembled items should have clear match mark number of the erection drawing. Critical items should be taken special care.

514.2.4. Protruded members to be specially protected during transit. Threaded and machined portion of fabricated structures should be carefully handled against damage.

514.2.5. Small items, e.g., nuts, bolts, washers, packing plates rivets electrodes shall be dispatched in containers and details fully listed to ensure proper receipt and storage. Under loaded consignments should be normally avoided.

514.2.6 In case of heavy and unusual structures, availability of the transportation medium should be checked in advance and arrangements tied up. Stability of the members shall be checked during loading or transportation. Necessary safety measures shall be ensured.

514.2.7. For access to the erection site, it may be necessary to erect temporary road bridges which can allow safe movement of the fabricated materials and equipment.

514.3. Storage

514.3.1. Suitable area for storage of structures and components shall be located near the site of work. The access road should be free from water logging during the working period and the storage area should be on a leveled and firm ground.

514.3.2. The store should be provided with adequate handling equipment, e.g., road mobile crane, gantries, derricks, chain-pulley blocks, winch of capacity as required. Stacking area should be planned and have racks, sleeper stands, access tracks and properly lighted.

514.3.3. Storage should be planned to suit erection work sequence and avoid damage or distortion.

514.3.4. Fabricated materials are to be stored on non-corroding surfaces with erection marks visible, such as, not to come in contact with earth surface or water and should be accessible to handling equipment.

514.3.5. Small fittings, hand tools, are to be kept in containers in covered stores.

514.4. Handling

514.4.1. IS:7293 and IS:7969 dealing with handling of materials and equipment for safe working should be followed. Safety nuts and bolts as directed are to be used while working.

514.5. **Erection Scheme**

514.5.1. Design of a bridge should take into consideration th method of erection. A detailed scheme must be prepared showin stage-wise activities, with complete drawings and working phase-wis instructions. This should be based on detailed stage-wise calculatior and take into account specifications and capacity of erection equipmen machinery, tools, tackles to be used and temporary working loads as per codal provisions.

514.5.2. The scheme should be based on site conditions, e.g., hydrology rainfall, flood timings and intensity, soil and subsoil conditions in the riverbed and banks, max. water depth, temperature and climatic conditions, available working space, etc.

514.5.3. The scheme should indicate detail of materials required with specifications and quantities, type of storage required, etc.

514.5.4. The scheme should indicate precisely the type of temporary fasteners to be used as also the min. percentage of permanent fasteners to be fitted during the stage erection. The working drawing should give clearly the temporary jigs, fixtures, clamps, spacers supports, etc. Adequate provision of spares of vulnerable items to be made.

514.6. **Procedure of Erection**

514.6.1. Prior to actual commencement of erection, all equipment, machinery, tools, tackles, ropes, etc. need to be tested to ensure their efficient working. Frequent visual inspection is essential in vulnerable areas to detect displacements, distress, damages, etc.

514.6.2. Deflection and vibratory tests shall be conducted in respect of supporting structures, launching truss as also the structure under erection and unusual observations reviewed. Looseness of fittings are to be noted.

514.6.3. For welded structures, welders' qualification and skill are to be checked as per standard norms. Non-destructive tests of joints as per designer's directives are to be carried out.

514.6.4. Precision non-destructive testing instruments available in the market should be used for noting various important parameters of the structures frequently and systematic record is to be kept.

514.6.5. Safety requirements should conform to IS:7205, 7273 & 7269 as applicable and should be a consideration of safety economy and rapidity.

514.6.6. Erection work should start with complete resources mobilized as per latest approved drawings and after a thorough survey of foundations and other related structural work. In case of work of magnitude, maximum mechanization is to be adopted.

514.6.7. The structure should be divided into modules, as per the scheme. This should be pre-assembled in a suitable yard/platform and its matching with members of the adjacent module checked by trial assembly before erection. Such assembled girders may be tested with simulated loads in case of erection on difficult terrain.

514.6.8. The structure shall be set out to the required lines and levels. The strokes and masses are to be carefully preserved. The steelwork should be erected, adjusted and completed in the required position to the specified line and levels with sufficient drifts and bolts.

Packing materials are to be available to maintain this condition. Organized "Quality Surveillance" checks need to be exercised frequently.

514.6.9. The method of erection, the drawing of temporary work and the use of erection equipment shall be subject to the approval by the Engineer.

514.6.10. **Joints:** Any connection to be riveted or bolted shall be secured in close contact by service bolts or specified no. of permanent bolts before final connection. Service bolts are to be fully tightened and kept as such by torque wrenches when the joint is assembled. Joints shall be made by filling not less than 50 per cent of the holes with service bolts and drifts in the ratio of 4:1. Connections to be completed by close tolerance bolts or as specified, as such, practicable.

514.6.11. Any connection to be site welded shall be securely held in position by approved means to ensure accurate alignment, camt and position before welding is commenced.

Appendix-A

LIMITATIONS

(Clause 502)

A1. The following special steel bridge structures have not been covered in the present code :

(a) *Curved Bridges*

For curved bridges, rigorous analysis should be made and detailing must follow the needs of the curvature effects. Having taken into consideration the above aspects, provisions of this code can be applied to curved bridges as appropriate.

(b) *Cable Stayed Bridges* and

(c) *Suspension Bridges*

These are special types of bridges calling for specialised treatment both for analysis and design. Also, erection conditions need to be thoroughly analysed.

(d) *Temporary* and

(e) *Pedestrian Bridges*

Because of their nature of use, certain provisions of the code (such as, permissible deflection, live load, etc.) can be relaxed subject to the approval of the Engineer

(f) *Swing Bridges and*

(g) *Bascule Bridges*

These type of bridges involve mechanical equipment for which relevant codes need to be referred. For structural portion of analysis and design, provisions of the code as appropriate can be applied.

(h) *Box Girder Bridges*

(i) *Prestressed Steel Bridges*

A2. Also, the following aspects of steel road bridges have not been covered in this code :

(a) *Ratings of Bridges*

For this aspect, IRC:SP:37-1991 "Guidelines for Evaluation for Load Carrying Capacity of Bridges" may be followed.

(b) *Fatigue*

Fluctuation of stresses may cause fatigue failure for members or sections at lower stresses than those at which they would fail under the load. However, for steel road bridges, this aspect is not always critical and has, therefore, not been included in this Code. Members subjected to fluctuation of stresses need to be examined in accordance with IS:1024-1979 (Ref. Clause 505.5).

Appexdix-B

RULES FOR CAMBERING PRE-DEFORMED OPEN WEB GIRDER SPANS

[Clauses 506.8.3.1 & 507.6.2 (b)]

B1. **Preparation of Camber Diagram**

B1.1. Contract drawings are dimensioned for the main girder without camber and in order to ensure that its fabrication and erection shall be, such as, to eliminate deformation stresses in the loaded span, a camber diagram shall be prepared on which shall be clearly indicated the amounts by which the nominal lengths (i.e.) the lengths which will not give camber) of members shall be increased or decreased in order that the outline of the girder under full load (dead load and 75 per cent live load without impact), shall be the nominal outline. A further change as indicated in para 1.4 may be made when the outline of the girder shall be normal outline, enlarged (1 + K) times in the case of a through span and reduced (1 - K) times in the case of a deck span (see para 1.4 below for definition of K).

B1.2. The stress camber change in each member shall be equal to the change of length of member due to the above loading, but of opposite sign.

B1.3. For the purpose of calculating the change in length of members under stress, the modulus of elasticity for both high tensile and mild steel shall be taken as 2.11×10^5 MPa. The effective length shall be taken between the theoretical intersection points of adjacent members.

B1.4. To ensure that the length of the floor system of a span shall be constructed to its nominal dimensions, i.e., to avoid changes in lengths of floor and loaded chord lateral system a further change in length shall be made in the lengths of all members equal to :

$$\frac{\text{Loaded chord extension or contraction}}{\text{Loaded chord length}} \times \text{length of member} = K \times L$$

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

B1.5. The nominal girder lengths altered in accordance with paragraphs 1.1 and 1.4 give a girder correctly stress cambered but with the loaded chord length identical with that shown on the contract drawings, thus requiring no modifications to floor and loaded chord lateral systems.

B1.6. The nominal lengths and camber lengths shall be rounded off to the nearest half a millimeter.

B1.7. The difference between nominal lengths and camber lengths thus, modified is the practical camber changes.

B1.8. The ordinates corresponding to the required camber at nodes may be obtained either by drawing a Williot Mohr Diagram or any other acceptable method.

B1.9. Adjustments of the lengths shall be made to top lateral bracing members to suit camber lengths of the top chords in the case of through girder spans and to the bottom lateral bracing members in the case of deck spans. The average value of the pre-stressed length of top or bottom lateral member, as the case may be. shall be adopted throughout.

B2. Fabrication

B2.1. The actual manufactured lengths of the members are to be the lengths "with camber" given on the camber diagram.

B2.2. The positions and angular setting out lines of all connection holes in the main gussets and also the positions of the connection holes in the chord joints and the machining of

the ends shall be exactly as shown on the contract drawings. This will permit the butts in the chord segments to be exactly as shown on the contract drawings.

B2.3. The groups of connection holes at the ends of all the members are to be as shown on the contract drawings, i.e., without any allowance for camber but the distance between the groups at the ends of each member shall be altered by the amount of the camber allowance in the member.

B3. Erection

B3.1. The joints of the chords shall be drifted, bolted and preferably riveted to their geometric outline.

B3.2. All other members are to be elastically strained into position by external forces, so that as many holes as possible are fair when filled with rivets.

B3.3. Drifting of joints shall be avoided as far as possible, and when necessary, should be done with great care and under close expert supervision. Hammers not exceeding one kg. in weight should be used with turned barrel drifts and a number of holes drifted simultaneously, the effect of the drifting being checked by observation of adjacent unfilled holes.

B3.4. The first procedure during erection consists of placing camber jacks in position on which to support the structure. The camber jacks should be set with their tops level and with sufficient run out to allow for lowering of panel points except the centre by the necessary amounts to produce the required camber in the main girders. It is essential that the camber is accurately maintained throughout the process of erection and it should be constantly checked. The jacks shall be spaced so that they will support the ends of the main girders and the panel points. The bottom chord members shall then be placed on the camber jacks, carefully leveled and checked for straightness and the joints made and riveted up.

B3.5. The vertical and diagonal web members, except the posts, shall then be erected in their proper positions on the bottom chords. It is recommended that temporary top gussets, the positions of the holes in which are corrected for the camber change of length in the members, should be used to connect the top ends of the members; this will ensure that the angles between the members at the bottom joints are as given by the nominal outline of the girders. The vertical and diagonal shall then be riveted to the lower chords.

B3.6. All panel points, except the centre, shall now be lowered by amounts to produce the correct camber in the main girders as shown on the camber diagram.

B3.7. The top chord should be erected piece by piece working symmetrically from the centre outwards, and the joint made by straining the members meeting at the joint and bringing the holes into correct registration.

B3.8. The temporary gussets, if used, shall be replaced by the permanent gussets in the same sequence as the erection of the top boom members.

B3.9. The end posts shall be erected last. The upper end connection should preferably be made first and if there is no splice in the end raker, the final closure made at the bottom end connection. If there is a splice, the final closure should be made at the splice.

B3.10. When cantilever method of erection is used, the above procedure does not apply.

Appendix-C

PROTECTION AGAINST CORROSION

C1. Introduction

C1.1. Phenomenon of corrosion is essentially recognized as an electro-chemical process through formation of anode (steel substrata), cathode (top mill scale) in the all pervading presence of electrolyte (moisture).

C1.2. Concentration of oxygen, carbon dioxide, chlorides causing acid film and unfavourable temperature, residual stresses, etc. are some of the factors which initiate and accelerate corrosion. An everlasting solution to the problem of protection against corrosion is yet to be evolved.

C1.3. All steel work should be designed and detailed to minimise the risk of corrosion. Provision of IS:8629 should be taken into account for corrosion protection.

All parts should be accessible for inspection, cleaning and painting or should be effectively sealed against corrosion. Where these methods are not possible either the surface of the steel should be given a system of protective coating or treatment selected with due regard to the design life of the part, together with an additional thickness of the steel or the steel used should have corrosion resistant properties as suitable for the design environment.

Adequate drainage should be provided where there are chances of collection of water. Hollow members without access for internal inspection and maintenance should be effectively sealed against corrosion.

C2. Aspects to be considered

C2.1. Prior to choosing and specifying the type of protective system for bridge structures, it is necessary to:

(a) Identify the environment

(b) Assess life cycle costs for some prima-facie suitable systems

(c) Compare and select the preferred system

(d) Define the system as completely as possible

Selection of a protective system and the correct specifications of material including testing and their application methods are essential and important for satisfactory performance of the chosen system under given environmental conditions

C2.2. *Environment*

Following aspects need to be studied :-

C2.2.1. *Broad environmental conditions :*

(a) Exterior exposed

(b) Polluted/non polluted (Inland)

(c) Polluted/non polluted (Coastal)

(d) Exterior sheltered

(e) Saline/non-saline

C2.2.2. *Local conditions :*

(a) Splash/Fully immersed zone;

(b) Presence of harmful salts, viz. sulphates/chlorides

(c) Likelihood of abrasion and impact

(d) Presence of fungi and bacteria

C2.2.3. *Other factors :*

(a) Appearance (after painting) vis-à-vis the surroundings

(b) Maintenance aspects

- Access for effective inspection and maintenance
- Possibility of effective maintenance.

- Time schedule for renewal of coating system

(c) Tolerance of coating

- In different surface preparation
- In different application treatments

(d) Suitability of coating system from consideration such as :

Ready availability, compatibility, application methods such as brushing airless spray, etc., time schedule between first coating and cost effectiveness final treatment, damage during transport and handling, etc.

(e) Past performance of the coating system

(f) Level of expertise required in application of coating system.

C3. **Control of Corrosion**

Corrosion can be minimised/controlled by :

- Improved design and detailing;
- Cathodic protection;
- Concrete encasement;
- Use of special corrosion resistant steels;
- Protective coating on surface.

Salient features concerning the above are given hereunder :-

C3.1. *Improved design and detailing*

This essentially will entail :

- Selection of appropriate quality of steel;
- Proper slope and geometry of structures to avoid accumulation of deleterious matter and debris;
- Easy accessibility to all parts of structures;
- Proper drainage hole locations; and draining out without affecting structural members.

- Provision of good seal in bolted connections; (to avoid corrosion in the crevices; by trapped moisture);
- Avoidance of sharp cut-edges;
- Ensuring Electro-chemical compatibility between the coating and parent-metals;
- Arrangements for safeguarding painted members from accidental damage during transport/erection).
- Extra thickness may be provided as per clause in the relevant chapter.

C3.2. *Cathodic Protection*

There are two basic types of cathodic protection :

(a) Sacrificial anode system: This system does not require external power source. Magnesium, Zinc, Aluminium are used as anode and the metal to be protected becomes cathode. Hence, there is no corrosion in the metal.

(b) Impressed current system: This system requires D.C. power supply.

C3.3. *Concrete Encasement*

The concrete itself acts as a protective barrier. Adequate cover and good quality of concrete is however, essential. Loose mill scale and rust should be removed but other surface preparations are not necessary. The steel work must not be painted but a cement-wash may be given to the steel surface to prevent corrosion until it is encase.

C3.4. *Use of Corrosion Resistant Steel*

C3.4.1. Weathering steels are low-alloyed steels containing a total of 1 per cent - 2 per cent alloys, in particular, copper, chromium, nickel and phosphorous. The chemical composition of this steel is such that increased protection is provided by the patina which develops after a short period of oxidation. Atmospheric that no painting is required for such steels is not justified as recent experience of such steels,

exposed to saline environment, (where chlorides ions act as de-passivators indicated that weathering steels act as de-passivators) indicates that weathering steels have also some corrosion problems. Fabrication and erection of such steels need greater care.

C3.4.2. *Stainless Steel*

Another well known non-corrosive type of steel is the stainless steel. Stainless steels are known for their resistance to atmospheric corrosion. However, such steels also tend to corrode particularly where they are protected from direct oxygen supply or where oxygen availability is reduced. Adverse environmental conditions, such as, warm moisture, saline charged atmosphere is also detrimental to long term satisfactory performance of these steels. Use of these may be considered only as special components.

C3.5. *Protective Surface Coating*

C3.5.1. All protective coatings are classified separately as

(a) Metallic; e.g. galvanizing, metallising, etc.

(b) Non-metallic, e.g., paints

C3.5.2. *Galvanising*

The process of providing zinc coating to steel surface is called galvanisation.

C3.5.3. *Metallising*

Metallising consists of projecting an atomised stream of molten metal at a high velocity from a special gun on to a 'prepared' surface.

C3.5.4. *Painting*

Paints are a mixture of film forming material called a vehicle or binder and pigments for colouring and protection. A drying agent may also be added. The vehicle or binder may be natural oil or resin or alkyd varnish or synthetic resin.

Pigments, are solid particles, insoluble in the binder and add colour to the paint or corrosion resistance.

C3.5.4.1. *Surface Preparation*

Steel surface to be painted either at the fabricating shop or at the site of work shall be prepared in a thorough manner with a view to ensuring complete removal of mill scale by one of the following processes as agreed to between the fabricator and the Engineer (or the purchaser).

(a) Grit/sand blasting

(b) Pickling which should be restricted to single plates, bars and sections

(c) Flame cleaning

(d) Scraping and wire brushing

The last process, viz., scraping and wire brushing should be avoided as far as possible and restricted to temporary bridges only.

Primary coat shall be applied as soon as practicable after cleaning and in case of flame cleaning, primary coat shall be applied while the metal is warm.

All slag from welds shall be removed before painting. Surfaces shall be maintained dry and free from dirt and oil. Work out of doors in frosty or humid weather shall be avoided.

C3.5.4.2. *Type of Paints*

C3.5.4.2.1. *Ordinary paints*

These include paints based on drying oils, alkyd resin, modified alkyd resin, phenolic varnish epoxy, etc.

Alkyd resin paints for the protection of steel structures are based partly on natural oils and partly on synthetic resins. These paints are used for steel structures in atmospheres which are not too aggressive.

Oil based paints are only used for steel structures in cases where the surface preparation cannot be ideal. Ordinary painting system can generally be sub-divided into two groups.

(a) *Primary Coats*

This is applied immediately after the surface preparation and should have the properties of

- Adhesion
- Corrosion inhibition and
- Imperviousness to water and air.

(b) *Finishing Coats*

These are applied over the primary coats and should have the properties of

- Durability
- Abrasion resistance, and
- Aesthetic appearance and smooth finish.

Depending on the aggressive nature of the environment the number of coats can be increased, e.g., A primer, under-coat and top-coat can be given for aggressive marine atmosphere.

C3.5.4.2.2. *Chemical resistant paints*

The more highly corrosion resistant paints can be divided in two main groups

- One pack paints (ready for use)
- Two pack paints (mixed before use).

The two pack paints are mixed together immediately before use and are workable thereafter only during a restricted period of time. They dry as a result of a reaction between their components and yield hard tough films with resistance to abrasion.

C3.5.4.2.3. *Vinyl paints*

Vinyl paints are based on polyvinyl resins, such as polyvinyl-chloride (PVC) and polyvinyl-acetate, etc.

Certain types of vinyl resin paints yield thick, relatively soft and rubber, like, coatings with good chemical resistance. They can be repainted without difficulty.

C3.5.4.2.4. *Chlorinated rubber paints*

These paints have also good chemical resistance. The main fields of applications are in aggressive environments and in the chemical industry. In general, chlorinated rubber paints do not have a high gloss.

C3.5.4.2.5. *Bituminous paints*

As a paint vehicle, bitumen is inferior to other vehicles but because of the low price, this could be applied in greater thickness (up to several millimeters) and may be suitable for some situation. A significant advantage of bitumen paints is their impermeability to ingress of water. However, bituminous paints do not withstand effectively detrimental effects of oil.

C3.5.4.2.6. *Epoxy paints*

These resin paints have good adherence to a very well prepared surface. They are mechanically strong and resistant to chemicals. A disadvantage of epoxy resin paints is that it can rapidly become dull when exposed to strong sun light. These disadvantages do not. however, greatly influence their protective power.

C3.5.4.2.7. *Polyurethane paints :*

The chemical and mechanical behaviour of polyurethane paint resemble very much those to the epoxy paints. However, polyurethane paint retains its gloss for a longer period.

C3.5.4.2.8. *Zinc rich paints :*

Instead of introducing an inhibitive pigment into paint, metallic zinc can be used, and such paints can provide cathodic protection to steel.

C3.5.4.3. *Choice of Painting System*

The choice of suitable painting system is dependent on factors such as :

- Available application methods viz. brush, roller or spray;
- Durability in a specific environment
- Availability of skilled manpower
- Cost/benefit. etc.

It is therefore necessary to consult various manufacturer of paint and ascertain the above aspects while deciding on the appropriate choice of painting system.

C3.5.4.4. Typical guidelines for epoxy based paints and the conventional painting system for bridge girders are given below :

(a) *Epoxy Based Painting*

(i) Surface preparation - Remove oil/grease by use of petroleum hydrocarbon solution (ISI :1745-1978). Grit blasting to near white metal surface.

(ii) Paint system - 2 coats of epoxy zinc phosphate primer = 60 micron.

Total 5 coats = 200 micron.

(b) *Conventional Painting System*

(i) (For areas where corrosion is not severe)

Priming coat:

One heavy coat of ready mixed paint, red lead primer to IS:102.

or

One coat of ready mixed paint zinc chrome primer to IS: 104 followed by one coat of ready mixed paint red oxide chrome primer to IS: 2074.

or

Two coats of zinc chromate red oxide primer to IS:2074

Finishing Coats:

Two cover coats of red oxide paint to IS: 123 or of any other approved paint shall be applied over the primer coat. One coat shall be applied before the fabricated steel work leaves the shop. After the steel work is erected at site, the second coat shall be given after touching up the primer and the cover coats, if damaged in transit.

(ii) (For areas where corrosion is severe)

Priming Coat

Two coats of ready mixed paint red lead primer to IS:102.

or

One coat of ready mixed zinc chrome primer to IS:104 followed by one coat of zinc chromate oxide primer to IS:2074.

Finishing Coats

Two coats of aluminium paint to IS:2339 shall be applied over the primer coats. One coat shall be applied before the fabricated steel work leaves the shop. After the steel work is erected at site the second coat shall be given after touching up the primer and the cover coat if damaged in transit.

C3.5.4.5. *General*

Surface which are inaccessible for cleaning and painting after fabrication shall be painted as specified before being assembled for riveting.

All rivets, bolts, nuts, washers, etc. are to be thoroughly cleaned and dipped into boiling linseed oil to IS:77.

All machined surfaces are to be well coated with a mixture of white lead to IS:34 and Mutton tallow to IS:887.

For site paintings the whole of the steel work shall be given the second cover coat after final passing and after touching up the primer and cover coats, if damaged in transit.

C3.5.4.6. *Quality of Paint*

The paints which have been tested for the following qualities as per the specification given in the relevant IS codes should only be used.

- Weight Test (weight per 10 lit. of paint thoroughly mixed);
- Drying times;
- Flexibility and adhesion;
- Consistency;
- Dry thickness and rate of consumption

C4. **Guidelines for Protective Coating System in Different Environments**

Corrosion has to be controlled in an economical way. Since the seriousness of the problem depends on atmospheric conditions and these vary enormously, there is no single protective system or method of application that is suitable for every situation.

C4.1. However, as a guide, broad recommendations are given in Table C-1 for various types of coatings in various environmental conditions, based on past experience as well as trials conducted overseas (both in laboratories and also in the field). Approximate life to first maintenance is also indicated and can be used as a guide for deciding on the maintenance schedule.

Table C-1. Recommendations for Types of Protective Coatings

	System	Environment
1.	Wire brush to remove all loose rust and scale; 2 coats drying oil type primer, 1 undercoat alkyd type paint; 1 finishing coat alkyd type. Total dry film thickness = 150μm.	Suitable for mild conditions where appearance is of some importance and where regular maintenance is intended This system may deteriorate to marked extent if it is exposed to moderate aggressive atmospheric conditions for lengthy period.
2.	Wire brush to remove all loose and scales; 2 coats drying oil type primary, 2 undercoats micaceous iron oxide (MXO) pigmented phenolic modified drying oil : Total dry film thickness : 170μ m.	Similar to (i) but where the appearance is not very important, provides longer life in mild condition. Will provide up to 5 years life to first maintenance in polluted inland environment.
3.	Blast clean the surface; 2 coats of quick drying primer under-coat alkyd type paint; Total dry film thickness 130 - 150μ m.	Compared with (i) this would provide a longer life in mild conditions and could be used in less mild situation, e.g. inland polluted, where maintenance could easily be carried out at regular intervals.
4.	Blast clean the surface; 2 coats of drying oil type primer, 1 under-coat micaceous iron oxide pigmented drying oil type paint; Total dry film thickness=169-190μm.	Suitable for general structural steelwork exposed to ordinary polluted inland environments where appearance is not of primary importance.
5.	Blast clean the surface; 2 coats of metallic lead pigmented chlorinated rubber primer, 1 under-coat of high build chlorinated rubber, 1 finishing coat of chlorinated rubber : Total dry film thickness=200μm.	Suitable for structures in reasonably aggressive conditions, e.g., near the coast. Will provide long-term protection than (iv) in non-coastal situations. Also, suitable for aggressive interior situations such as industrial areas.
6.	Blast clean the surface : 350 μm. - 450 thickness coal tar epoxy.	This system would be suitable for sea-water splash zones or for conditions of occurrence of frequent salt sprays.
7.	Pickle; hot dip galvanise (Zinc) Total thickness = 85μ m.	Suitable for steel work in reasonably mild conditions. A life of 15-20 years before first maintenance could be expected in many situations.
8.	Grit blast, hot dip galvanised (Zinc) Total thickness = 140μ m.	Provides a longer life than (vii) because of the thicker zinc coating.
9.	Grit blast, 1 coat of sprayed zinc/aluminium followed by suitable sealer, Total thickness = 150μ m.	Expected to provide long term protection approx. 15-20 years in aggressive atmosphere.

Appendix-D

POST-CONSTRUCTION INSPECTION AND PREVENTIVE MAINTENANCE GUIDELINES

D1. General

Bridge structures, permanently exposed to atmosphere are subjected to effect of adverse environmental conditions. Investment made in the structural facility can be protected by well programmed, monitored-inspection and maintenance schedule adopted for its designed life. Such systems developed ensures structural safety by recording the state of the structure periodically and providing feed back information to designers while actually identifying actual and potential sources of trouble and taking remedial measures in time.

This clause lay down the desired inspection procedure for determining physical condition and programming maintenance needs of the bridges. Systematic periodical inspection required for various elements and the responsibilities of the inspection group have been specified. The scope of maintenance work involved does not include correcting measures for known deviations introduced during construction stage and no attempt has been made in this direction. It is necessary to understand that for proper inspection of various components of the bridge structure in built facilities should be developed at the detailing and construction stage, for accessibility to important areas.

D2. Inspection

Bridge inspection is done by use of well tried and established techniques required for assessing the physical condition of the structure. IRC Special Publication 35 : "Guidelines for Inspection and Maintenance of Bridges" may be referred in this connection.

D2.1. *Personnel*

D2.1.1. The in-charge for bridg possses the following qua

(a) Be a qualified en experience in Bri

(b) Have a minimum of 10 years experience in bridge inspection assignments in a responsible capacity.

D2.1.2. He shall be responsible for a methodical and thorough field inspection, the detailed analysis of all observation recorded, arrive at findings to recommend rectification of defects, imposition of speed restriction or load limitations and any other measures as necessary.

D2.1.3. The problems encountered in this work are variable and complex as such matured judgment is often required for evaluation of the recordings.

D2.1.4. He must be thoroughly familiar with design and construction features of the bridge so as to make a correct interpretation and capable of determining the safe load carrying capacity of the existing structure. He should be capable of recognising any deficiency in the structure, assess the seriousness and suggest appropriate remedial measures to ensure safety. His experienced knowledge to recognise problem areas (actual and potential) and to ensure preventive maintenance is an important requisite.

D2.1.5. He should be able to utilise the expert knowledge and skills of associate Engineers in respect of structural design, construction methods, material, hydro dynamics, equipment, soil technology, maintenance methods for permanent and emergency measures, etc. and should have access to resources and expert systems.

D2.1.6. Definite guidelines should be given to ensure availability of technical assistance from other agencies/where regular staff is not available. In case of specialised structure consultation with expert bodies is a necessity.

D3. Training

Bridge management requires extensive team work involving various levels of responsibility and skills. Training programmes need be framed in order to develop expertise. Training facilities may be set up at central level for training of trainees and at local level for actual imparting training to field staff, workshop on topical interest may be held regularly to acquaint the concerned people with accepted technical methods and their correct application.

D4. Frequency of Inspection

D4.1. *Detailed Inspection*

D4.1.1. The details and frequency to which Bridges are to be inspected will depend on such factors as age, traffic characteristics, state of the structure, vulnerability and having known history of deteriorating condition. Evaluation of these factors will be the responsibility of the individual in-charge of the inspection programme.

D4.1.2. Each bridge has to be inspected in detail at regular intervals not exceeding 5 years.

D4.2. *Periodic Routine Inspection*

Certain items in each bridge have to be inspected at definite intervals of time atleast once a year, irrespective of whether anything alarming has taken place or not.

D4.3. *Special Inspection*

Such Inspection are required for any bridge with known deficiencies, like, restriction on weight/speed, loss of

camber and are considered necessary on the basis of routine inspection or unusual occurrence.

Recommended frequencies of various inspection item are shown in Table D-1. These are for guidance only.

Table D-1. Bridge Inspection Record Sheet

Sl. No.	Inspection Items		5 Year	1 Year	6 Months
1.	Main Bridge Structure	a) Steel Girders and Stringers	*	*	
			*	*	
		b) Trusses			
2.	Bearing	a) Wearing surface	*	*	
3.	Decks	a) Drainage System	*	*	As situation demands
			*	*	
		b) Steel Deck	*	*	
		c) Curbs	*	*	
		d) Foot Path	*	*	
4.	Expansion Joints		*	*	
5.	Railing and Crash Barriers		*	*	
6.	Signs		*		
7.	Services and Utilities		*		

Note : In the case of distressed bridge, special instructions to be issued by the competent authority.

D5. Inspection Procedure

D5.1. *General*

D5.1.1. The field inspection of a bridge should be conducted in a systematic and organised manner and observation recorded on a format to ensure that no item is overlooked. Notes must be clear and detailed to the extent that they can be interpreted at a later date when report is prepared. Sketches and photographs should be included in an effort to record actual conditions.

D5.1.2. As far as possible the inspecting officer should schedule bridge inspection in those periods of the year which offer the most favourable conditions. Inspections during temperature extremes should be made at bearing, joints etc. Inspection should not be confined only to search for defects which exist, but for conditions of anticipatory nature and marking those zones. Preventive maintenance is equally important to corrective ones.

D5.2. *Inspection Items*

Inspection of all items, such as, approaches, waterways, basic floor conditions, substructure which may affect the safety of the steel superstructure need be done, like, other types of bridges.

D6. **Main Structure**

D6.1. Steel Girders and Stringers in the deck structure should be examined for signs of corrosion, cracks along the flanges around rivet or bolt heads, its contact surfaces and where water enters and stands or debris may collect at the ends.

D6.2. Flanges and webs shall be checked for any damage or misalignment. Web-stiffeners are to be examined for signs of deformation due to buckling. Unusual vibration or excessive deflection under passage of heavy loads should be noted and cause investigated.

D6.3. All end connections should be inspected to make sure than they are secure.

D6.4. Weld areas should be inspected to check crack. Special care should be exercised to inspect corners, curved, sections and

areas where there is an abrupt change in the size of metal or in configuration which may produce an area of concentrated stress or in areas where vibration or movement could produce stress concentration. Damages or deformation caused to the main-structural members due to vehicular impact should be particularly watched. Fatigue failure and welded joints being a cause of concern in bridge structures with age such structures need more careful check and watch.

D6.5. Creep - The longitudinal movement of a girder is termed as creep. This point should be checked and girders pulled back if necessary to the proper position.

D6.6. Distortion - With variation of temperature, the girder is likely to have longitudinal movement due to expansion and contraction, absolute freedom of movement is impracticable and there remains a residual force which develops internal stresses, causing tendency to distort. Distortion is measured in the bottom top chord.

D6.7. *Lateral Bracing*

Normally a span of a bridge consists of two or more girders braced together with lateral bracings. These bracings should be thoroughly checked for corrosion on loose rivets and deformity, viz., bracing distortion, etc.

D6.8. *Loose Rivets*

Field rivets are to be examined for looseness. This is caused due to running traffic and consequent vibratory effect and corrosion around the rivets. A joint with loose rivets should not be touched unless more than 20 per cent rivets in the joint are loose.

D6.9. *Bearings*

D6.9.1. All bearing devices should be examined to ascertain that they are functioning properly. Changes in other parts of the structure, such as, pier/abutments settlement and tilt may be reflected in the bearing. Bearings should be seated properly on their bed plates and provided with gaps at both ends. Bearing assembly should be checked for possible cracks by magnifying glass after removal of paint cover in doubtful cases. In case of roller bearing, the relative position of the top castings, bottom castings and rollers variation to the temperature noted. Longitudinal movement of the free-end shall be recorded under moving load. After unusual occurrences bearings and support pads must be examined for cracks, etc. Lateral shear keys in skew bridges also invite special check.

D6.9.2. *Lubrication of Bearings*

Oiling and greasing of the bearing is done periodically once in 3 years. Improper and failure in timely lubrication may lead to corrosion of bearings resulting in reduction in strength and consequent damages.

Where bearings are encased in oil baths and stay submerged in recommended brand of lubricating oil the level of oil should be maintained on checking every year. It should be ensured that the oil baths are always sealed.

D6.9.3. *Elastomeric Bearings*

The physical conditions of elastomeric bearing pads should be inspected for observing any abnormal flattening, bulging or splitting which may indicate overloading or excessive unequal distribution of loading, shifting from original position should be checked particularly.

D6.9.4. *Condition of Bed-Block and H.D. Bolts*

Bed Blocks receive the full load from the bearings of the bridge and distribute and transmit the same to the masonry below. Restriction of free movement in superstructure may result in :

(1) Development of transverse cracks in piers/abutments

(2) Failure of bed blocks joints leading to shaking bed blocks

(3) Shearing of holding down bolts (Particular importance due to introduction of greater longitudinal forces)

It must be ensured that the anchor bolts are well secured.

D7. Trusses

D7.1. Camber of the trusses should be checked and the ambient temperature recorded at the time of detailed inspection. A camber diagram should be made in the inspection register. Loss of camber may be assessed from comparative readings.

D7.2. All truss members should be checked. The compression members should be checked for straightness absence of kink or bows and the connections are undisturbed. (Tension members should not show signs of cracking).

D7.3.1. The truss should be checked against damage due to collision with vehicular traffic, portal bracings and sway bracings are usually the most restrictive to overload movements and consequently susceptible to damage.

D7.3.2. The condition of pins at the connections and rivets, bolts should be checked to see that none are loose, worn-out or sheared. Particular care to be given to following locations :

(a) Connections of stringers to cross girders

(b) Connections cross girder to main girder

(c) End connection of bracings

(d) Chord joints and web-member connection

D8. Corrosion and Painting

Steel structure is sensitive to the atmospheric moisture and vehicular smoke and therefore should be protected by paints or anti-corrosive measures. The condition of the members should be examined and the extent of corrosion recorded. The portions of steel work where water is likely to stangate or which are subjected to alternate wetting or drying need special care. Deformation in riveted or bolted multiple sections should be examined to check if moisture has entered and corroded the contact surfaces of the plates causing them to be pushed apart. The exact location and area of the affected portion should be recorded. This area should be got cleaned, thoroughly scraped, old paint, rust, scaling removed and repainted and appropriate remedial measures taken up immediately.

D9. Decks

D9.1. Steel decks should be checked for corrosion and unsound welds. It is important to maintain an impervious surface over a steel plate deck to protect against corrosion in aggressive environmental condition.

D9.2. It is necessary to have effective drain holes to prevent collection of water on the deck.

D10. Expansion Joints

D10.1. Maintenance of these joints need special attention and should be carefully examined. The joint should be clear of debris and be able to have free thermal expansion as designed.

D10.2. Finger type joints and sliding plate joints should be checked for loose anchorage, cracking or separation of welds or other defects. Such defects cause structural deformation and is hazardous to traffic. Deck adjacent to expansion joints should be carefully examined for voids and cracks. Underside of expansion joints also need careful inspection. Systematic documentation of the movement of expansion joints need to be kept to judge proper functioning of the bridge structure.

D11. Railings and Safety Barriers

D11.1. Handrails are to be examined for unusual damage, deformation, corrosion and paintings. The embedment of posts to be checked for rust stains, which are signs of rusting. Extent of corrosion need to be checked when signs exist on the surface.

D11.2. All handrails are to be checked for any damage for traffic. The vertical and horizontal alignment are to be maintained.

D12. Services

The number and types of utilities, such as pipelines, cables, etc. must be inspected and observations kept for record with the details suitably displayed. Special care need be kept for hazardous utilities, regular joint inspection in such case with suitable guidelines is a necessity.

D13. Special Structures

D13.1. *Moveable Bridges*

The most common type of moveable bridge are the swing span, vertical lift bascule (Single or double leaf). Inspection of the trusses, floor system, and other structural elements will require inspection procedures suitably modified as per guidelines mentioned in the Code. Ensuring proper seating of the girder after operation is an absolute necessity.

D13.2. In case of other structures, like, suspension bridges, cable stayed bridges detailed inspection manuals should be prepared and staff trained to observe the same.

D14. Documentation

D14.1. The most important function of bridge maintenance unit is to prepare a complete, methodical and current record for each bridge on the system. Much of the usefulness of the information obtained from field investigation depend upon its reliability and availability on a concise format. The record must be preserved systematically and readily available.

D14.2. Records should provide a full history of the structure including all recommendations for strengthening and restoration works undertaken and the behaviour of the structure thereafter. This record should indicate clearly the life carrying capacity of the structure with supporting document calculations.

D14.3. Complete record in an usable format is vital for the continued service ability of the bridge. It is essential computerised data system is introduced as soon as practicable.

D14.4. A sample record sheet is shown in Table D-1.

D15. **Standard Tools**

A list of standard tools required for inspection is given in Table D-2 as guide.

Table D-2. Standard Tools and Equipment

A. STANDARD TOOLS

(1) Clip Board, Chalk, Markers, Clamps, etc.

(2) Pocket Tapes, Folding Rules, Tapes (10 m to 50 m) Feeler Gauges, Callipers, Micrometer Gauges,

(3) Straight Edge, Plumb Bob, Protector, Spirit Level.

(4) Thermometer, Inspection Mirror, Binoculars, Magnifying Glass, Camera.

(5) Scrappers, Emergy Paper, Portable Torque Wrenches, Light hammer, Piano-wire, Portable Ladder, Rope.

(6) Flash Light, Pocket Knife, Wire Brush, Chipping Hammer, Thin steel rod (for use as proper) (8 to 20 mm dia)

(7) Hydraulic Jacks, Pulley Blocks, Wire-Ropes, Chains, Slings, etc. of Ppropriate Capacity.

(8) Safety Equipment for Inpsecting Staff.

B. ASSESSMENT POSSIBLE

Steel	
Cracks	Ultrasonic, radiographic
Cable/Wire failure	Electric half cell potential
Corrosion meter	Electrical resistivity
Global behaviour	
Movements Instruments dial gauges	Modern Surveying
Extensometric measurements	Strain Gauges, and extensometer
Pressures, Forces	Pressures transducers, or load cells
Miscellaneous	
	Thickness of coating Paint film gauge (digital electrometer)
Water proofing membranes	Electric resistance
Vibration	Accelerometer
Widening of Cracks	Glass cell tabs
Metal thickness	Ultrasonic metal thickness measuring

D16. **Signs**

D16.1. All signs required to indicate restrictive load limit, reduced speed or impaired clearance should be inspected to ascertain they are visible and located in proper places. This inspection is to include sign at or on the structure and any advance warning sign. Examination should include that indications are legible and sign posts are well secured.

D16.2. For bridges over navigable, channels, it is necessary to inspect if the navigational signs for water-traffic are in place and secured. Navigational lights and serial obstructional lights should be inspected often to ensure that these are operating efficiently.

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STANDARD PROCEDURE FOR EVALUATION AND CONDITION SURVEYS OF STABILISED SOIL ROADS

1. INTRODUCTION

1.1. The Standard Procedure for Evaluation and Condition Surveys of Stabilised Soil Roads was approved by the Stabilised Soil Roads Committee (personal given below). This was later considered by the Executive Committee in their meeting held on the 24th March, 1962 and later it was circulated to the members of the Council on the 10th August, 1962. The comments of the members of the Council were considered by the Stabilised Soil Roads Committee in their meeting held on the 1st October, 1967 and later in their meeting held from the 19th to the 21st Septemper, 1968. This was then placed before the Executive Committee in their meeting held on the 13th March, 1969 and it was approved by the Council in their 71st Council meeting held at Bhubaneswar on the 26th and 27th May, 1969 for being published as the finalised standard of the Indian Roads Congress.

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1.2. There is real need for adopting a standard procedure for evaluating the condition and performance of stabilised soil roads. The approach to the rating or evaluation of a pavement may be deemed a negative one, inasmuch as it deals with the amount of destruction or the amount of failure that has taken place since its construction. In the case of a pavement, it is rather difficult to define precisely, what exactly constitutes its "failure". The term unfortunately has come to signify all things to all men and there is a prevalent tendency to use it to describe all manners of phenomena, some of which are not failures at all, but merely evidence of some condition which makes the pavement less than perfect. Any reasonable definition of failure of a pavement will have to take into account a stated amount of maintenance, because maintenance is a necessary feature in any type of failure. This will naturally entail deciding what should be an acceptable amount of maintenance. It is felt that this is a matter on which a general agreement would be rather difficult. In the circumstances, for the purposes of this standard, the term "failure" is applied somewhat loosely to an unsatisfactory condition in the pavement which is of sufficient severity to warrant attention.

1.3. Failures of flexible pavements may be grouped under three distinct heads depending on the primary cause or source of the trouble. In the first group are included the types of failure or unsatisfactory performance that are attributable solely to the quality of the surfacing itself. The second group represents a type of failure which may manifest itself in many forms, but in effect indicates slippage caused by lack of bond between the surfacing and the underlying layer. In the third group are failures which are attributable to the deficiencies of the base, sub-base or subgrade.

1.4. In the evaluation of a pavement which shows signs of distress or which does not satisfactorily perform the function for which it is intended, as a first step, it is important to determine the group to which the type of failure belongs or, in other words, to determine the component of the pavement in which the distress has occurred. In the following paragraphs, an attempt has been made to enumerate in brief the basic types of failures of flexible pavements, and approximate means of their identification.

2. DEFICIENCIES OF SURFACING

These would normally be noticed in the form of scaling, stripping, ravelling, disintegration, cracking and instability (or plastic distortion) of the road surface which may develop irrespective of the foundation support. Ravelling can be caused by low bitumen content, improper coating of aggregate, inadequate compaction. It may develop into pot-holing which may sometimes be attributed, by mistake, to failure in the lower layers of the pavement.

3. SLIPPAGE DUE TO LACK OF BOND WITH THE BASE

When the surfacing is not adequately keyed with the base, slippage will occur. This condition has often been noticed when

the surface of the stabilised soil base is exceedingly dusty or not properly primed before the application of the wearing course. Slippage can also cause ravelling of the surface and subsequent poor riding qualities.

4. DEFICIENCIES OF UNDERLYING LAYERS

Pavement defects falling in this category may be caused by (a) inadequate thickness of sub-base and base, and (b) inadequate compaction of subgrade, sub-base, and base. These are dealt with separately hereunder:

4.1. **Thickness :** The thickness of sub-base, base and surfacing above a given layer is considered inadequate, if detectable shear deformation occurs in that layer. Shear deformation, which is also referred to as plastic movement or plastic deformation, may be defined as change in shape with no change in volume. During traffic, materials move out from under wheel paths, creating a depression in the traffic lane and an upheaval outside the traffic lane. Incidentally, it may be mentioned that the thicknesses shown in the standard CBR design charts are intended to prevent all shear deformation in the layer with the given CBR.

Determination of the occurrence of shear deformation in a particular layer can be made by a study of (i) deflection measurement, (ii) in-place strength tests, (iii) cracking of the pavement, (iv) upheaval of the surface, and (v) position of the layers.

(i) **Deflection :** Deflection in the downward movement under load. Deflections at the level being considered (e.g. the subgrade) are measured in accelerated traffic tests under standing loads at intervals throughout the period of the test. Surface deflections can also be used, if the overlying layers are of high quality and adequately compacted, so that little compression occurs under load. Curves of deflections versus coverage (on a semi-logarithmic chart) are of important value in determining whether shear deformation is taking place. Curves which show a decreasing or constant deflection with coverage are typical of conditions where there is no shear deformation.

(ii) **In-place Strength Tests :** In-place CBR and other strength tests can also be used to indicate the development of shear deformation in any layer, if the tests are made at intervals. Where no shear deformation occurs, the CBR value will remain constant or increase with the traffic; where shear deformation is taking place, there will be a significant drop in the CBR value.

(iii) **Cracking :** The cracking that develops in a bituminous pavement, when shear deformation occurs, follows a typical pattern. In the early stages, the cracks are generally parallel to the direction of traffic. As repetitions are continued, transverse cracks are formed and an alligator pattern is developed. Closely spaced cracking indicates shear deformation in a layer near the surface, widely spaced cracking indicates shear deformation in a deep layer.

(iv) **Upheaval :** Upheaval of the surface adjacent to the traffic lane is definite evidence of shear deformation. The width of a rut indicates in a general way the depth of the failed layer. This should not be taken as a rigid rule, but only as a criterion with some reservations. Subgrade shear failures will exhibit surface upheaval at some distance from the depressed rut, whereas shear failures in the surface will result in upheavals relatively close to the tyre track.

(v) **Position of Layers :** A cross-section of the face of trench, cut across the traffic lane, can show whether or not a layer has been overstressed to the point where shear deformation has occurred. A thinning of the concerned layer in the traffic lane, accompanied by its thickening outside the traffic lane, is evidence of shear deformation in the layer. Also upheaval of the subgrade outside the traffic lane is evidence of shear deformation in the subgrade.

4.2. **Compaction :** Compaction is defined as a change in shape accompanied by a change in volume, as opposed to shear deformation where no change in volume occurs. The compaction of the pavement is caused by repetitive and vibratory movements of traffic and results in a depression beneath the wheel path. The shape of the depression is a clue to the layer that has densified. Compaction in a layer near the surface will produce a sharp depression; compaction at a depth will cause a broader depression.

5. GENERAL

5.1. It may be stressed that although the causes of pavement distress have been neatly separated in the preceding paragraphs, in practice it will never be the case. When distress occurs in a pavement, both compaction and shear deformation are involved, and it is necessary to try to separate the two. Compaction, though contributing to undesirable surface irregularity, increases the structural strength of a pavement and becomes successively reduced under given intensity of traffic. Shear deformation, on the other hand, becomes progressively more pronounced in its resultant effects.

5.2. These points have been set down here at some length to serve for guidance in the preliminary evaluation of stabilised soil pavements.

5.3. The condition survey should give a qualitative as well as a quantitative appraisal of the defects in the pavement. The qualitative study will involve rating of the defects according to their severity, the quantitative approach will take into account the extent of the distress and relate it to the total area of the pavement.

5.4. In addition, for a proper condition survey, the authorities concerned should give detailed data regarding cost of maintenance of the stabilized soil roads in service. The average traffic volume per day of commercial vehicles, passenger cars and bullock carts should also be included in the data. The information in Tables 1 and 2 should be recorded in the manner indicated below.

6. DIRECTIONS FOR FILLING UP OF THE DATA IN TABLES 1 & 2

TABLE No. 1

General Columns :

"Name of the road" will be main name, e.g., Delhi-Mathura road, and the section will mean a part of that road such as Agra-Mathura section. The "section" will be chosen in such a way that the type of construction equipment employed and the "type of stabilization" (e.g., soil-lime, soil-cement, soil-moorum, etc.) are uniform for in whole length of that section. The "type of construction equipment" will include only those equipment used for soil-stabilization proper such as pulverising and mixing of soil with the stabiliser, as also adding of moisture (e.g., rotavator, disc harrows, water sprinkler, pulvimixer single-pass stabiliser, etc.). For nature of shoulders, its brief specification and whether it is surfaced or not should be mentioned.

Detailed Columns :

(1) "Location" (col. 2) should be indicated by the actual kilometerage and chainage of the point relating to which the information is being supplied.

(2) Under the item "Composition and thickness of pavement layers" (col. 4) will come the details as indicated below (sample figures) :

(1) Wearing coat—premixed carpet/2 coat surface dressing, etc.

(2) Base coat —10 cm w.b.m., 15 cm soling, etc.

(3) Sub-base — (i) 10 cm lime-soil with 4 per cent lime; (ii) 7.5 cm lime-soil with 3 per cent lime, etc.

(3) "Gradation of soil" (cols. 8 & 9)—This should indicate the percentage passing by weight of soil in I.S. Sieve numbers 2.36 mm, 425 microns, and 75 microns (10, 40, 200 ASTM).

(4) "Degree of pulverisation" (col. 18)—The percentage by weight of soil passing through 25 mm and 4.75 mm I.S. Sieves should be indicated.

(5) Under the items "CBR and unconfined compressive strength of stabilised soil" (col. 19), the values should be determined on the soil-stabilised mix collected from the field just before the commencement of rolling and compacted in the laboratory at the field moisture content to achieve the degree of compaction as is expected in the field and then cured and soaked under a surcharge equivalent to the weight of the pavement (of course, no surcharge will be necessary in case of determination of unconfined compressive strength).

TABLE No. 2

Detailed Columns :

(1) Under col. 3, the "water table" should be determined by boring holes at the edge of the pavement at the end of the monsoon period when the water table is likely to be the highest.

(2) Cols. 4 and 5 have to be determined with the help of a bump integrator.

(3) Under col. 8, "Surface cracking", the alligator type of cracking should be indicated by area, whereas for ordinary cracking the length of the cracks should be indicated.

(4) Under col. 9, "Condition of surface", the rating should be "good", "fair", and "bad".

(5) Col. 10, "Cost of patch repairs per year", should give the figure of expenditure incurred only for repairing and maintenance of the surface for the period in between the renewals. No maintenance cost for laying of a renewal coat should be included in this figure.

TABLE No. 1

Details of Stabilised Soil Construction

1. State ______________________ 5. Type of Stabilization ______________________
2. District ______________________ 6. Width of Pavement ______________________
3. Name of Road ______ Section ______ Length in km ______ 7. Width and Nature of Shoulders ______________________
4. Type of Construction Equipment Employed ______________

Serial No.	Location	Months during which constructed	Composition and thickness of pavement layers	Subgrade			Soil used in stabilised layer									Field stabilised soil			Remarks
							Gradation of soil												
				L.L.	P.I.	C.B.R. with M.C. %	I.S. Sieve	% Passing by weight	L.L. %	P.I. %	Organic Content %	Sulphate Content %	Proctor Density	O.M.C. %	M.C. at Compaction	Degree of Compaction (% of Proctor)	Degree of Pulverization (% Passing Sieve No.)	C.B.R. & Unconfined Compressive Strength of Stabilized Soil	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20

TABLE No. 2

Service Performance

1. State ______________________ District ______________________
2. Name of Road ______________________ Section ______________________ Length in km
3. Rainfall : (A) Average Annual Rainfall ______________ (B) During Monsoon
4. Drainage of Subgrade : (A) Well Drained /(B) Poorly Drained /(C) Waterlogged
5. Date of Opening Road to Traffic ______________________ 6. Date of Observation ______________________
7. No. of Commercial Vehicles of different Tonnage using up to the data of Observation : (A) 8 Tonnes ______ Nos. (B) 5 Tonnes ______________ Nos. () 3 Tonnes ______________ Nos. (D) Bullock Carts

Serial number	Location	Depth of highest water table below formation level	Unevenness Index		No. of patches & total of patch area	Rut depth	Surface cracking	Condition of surface*	Cost of patch repairs per year/km	Remarks including type of failure including failed area—sq. m. per cent
			Before opening to traffic	On the date of observation						
1	2	3	4	5	6	7	8	9	10	11

*Present condition may be rated as : G for Good
F for Fair
B for Bad

IRC : 33-1969

RECOMMENDATIONS
FOR
ROAD CONSTRUCTION
IN
WATERLOGGED AREAS

THE INDIAN ROADS CONGRESS
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RECOMMENDATIONS FOR ROAD CONSTRUCTION IN WATERLOGGED AREAS

1. INTRODUCTION

A Panel Discussion* on the subject of construction of roads in waterlogged areas was held during the Hyderabad session of the Indian Roads Congress in January 1959. As a result of this discussion, the Soil Research Commitee (personnel given below) took upon itself the task of framing recommendations for road construction in these areas:

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Addl. Secy. to the Government of India.
(*Ex Officio*)

The recomendations drafted by the Committee were subsequently reviewed by the Specifications and Standards Committee (personnel given on inside front cover) and after approval of the Council at their 72nd meeting held on the 4th October, 1969 are now suggested for general adoption in the country.

2. SCOPE

The recommendations deal with the problem of road construction in waterlogged areas, including those subject in addition to flooding and/or infested with detrimental salts like the sulphates and carbonates. The recommendations relate both to the construction of new roads and to the remedial measures to be adopted in the case of existing constructions.

* The background note for the Panel Discussion is published in the Journal of the Indian Roads Congress, Volume XXIII-part 2 and discussion in the Volume XXIII-part 4.

For the purpose of these recommendations, waterlogged areas are considered to be those where the level of subsoil or standing water is such that for prolonged periods the subgrade immediately below the pavement is well within the capillary fringe of the water-table, i.e., within about 1.5 metres.

3. THE PROBLEM

3.1. Due to Waterlogging

As a result of migration of water by capillarity from the high water-table, the soil immediately below the pavement gets more and more wet and this leads to a gradual loss in its bearing value.

3.2. Due to Flooding

Where flooding for continuously long periods also takes place side by side with waterlogging, the progressive deformation of the subgrade, as well as of the pavement, is accentuated by ingress of water from the top of the wearing surface comprising usually of a thin bituminous treatment. The already inadequate waterproofness of the surface is impaired further by stripping of the binder due to prolonged contact with water. Infiltration of flood waters through the shoulders is another factor aggravating the situation.

3.3. Due to Presence of Detrimental Salts

The problem is made still more complicated if in addition to waterlogging, and/or flooding, injurious salts like sulphates of sodium, calcium or magnesium and sodium carbonate are present either in the subgrade soil or in the ground water. Damage to crust from injurious salts can be in two ways—due to physical effect or chemical.

3.3.1. Physical effect of the injurious salts: In waterlogged areas infested with detrimental salts, the salts keep on moving up with capillary moisture. During subsequent evaporation of the salt-laden water the salts are left behind and they get concentrated in the surface layers. The salts increase many times in volume upon hydration under suitable humidity and temperature conditions*. Alternate hydration and dehydration results in repeated formation of salt crystals occupying much more volume than the amorphous salts lodged in the voids. In due course these repeated volume changes break up the structure of the pavement, working from the top downwards.

*As a result of research carried out in India, it has been demonstrated that crystallisation of sodium sulphate takes place when temperature is below 32°C and relative humidity above 80 per cent. In northen parts of the country these conditions exist generally during the period of winter rains.

3.3.2. Chemical effect of the injurious salts: The damage due to chemical action is mainly on account of sulphates of calcium, magnesium and sodium. Constructions specially vulnerable to this type of attack are those containing cement like the cement concrete pavements and soil-cement base/sub-base course.

Cement concrete

The sulphates present in the subsoil which migrate to the top by capillarity react with the free lime liberated from cement resulting in the formation of gypsum. This reaction is accompanied by a considerable increase in the volume of the solids which is known to lead to the destruction of the hydrated cement matrix. After gypsum has been formed, or if calcium sulphate itself is found in the soil or ground water, tricalcium aluminate in the hydrated cement combines with gypsum to form needle-shaped crystals of double salts like calcium sulpho-aluminate which give rise to further expansion of volume and damage. Apart from the formation of gypsum and calcium sulpho-aluminate, the decomposition of hydrated calcium silicate by sulphate solution is an additional factor affecting the durability of concrete. However, this would occur normally only in the case of magnesium sulphate solution which is found generally in waterlogged areas near the sea.

Dissolved carbon dioxide and bicarbonate salts present in certain marshy and rice growing areas also take part in the leaching of lime liberated from cement and slowly attacking the cementitious calcium silicate hydrates formed. Such solutions are characterised by pH value in the acidic range, usually below 5.

Concrete is not directly attacked by solid sulphate salts, but only by their solutions in water, so that it is the amount of salts dissolved in the ground water that determines the rate of attack. As a result of chemical action of sulphates, the cement concrete pavements suffer internal disintegration and gradual spalling from the underside. The process described is, however, slow.

Soil cement

Presence of sulphates in waterlogged areas has a detrimental effect on soil cement mixtures akin to that in the case of cement concrete pavements.

Bituminous constructions

It does not appear that these salts, in the concentration they normally occur in soil, ground water or sea water, have any detrimental chemical effect on bituminous [illegible]

Water-bound macadam

Salts do not directly affect unsurfaced water-bound macadam constructions provided the filler material used in them is inert and free of injurious constituents.

4. RECOMMENDATIONS ON METHODS OF ROAD CONSTRUCTION IN WATERLOGGED AREAS

4.1. The recommendations are divided into the following three groups:

(a) Road construction in areas where the problem is one of waterlogging alone and is not tied up with flooding or salt infestation.

(b) Road construction in areas where in addition to waterlagging flooding for prolonged periods is also expected.

(c) Road construction in areas where in addition to waterlogging injurious salts are present in the suboil or ground water.

4.2. Different treatments are suggested under each group. Some of these can be made use of only on new constructions, and others on old, while some hold good for both. Broad guidance about these is provided at the beginning of each section.

5. RECOMMENDATIONS FOR ROAD CONSTRUCTION IN AREAS WHERE THE PROBLEM IS ONE OF WATERLOGGING ALONE AND IS NOT TIED UP WITH FLOODING OR SALT INFESTATION

The remedial measures recommended under paras 5.1, 5.2 and 5.4 could be utilised both on new constructions and existing roads. However, the capillary cutoff technique described under para 5.3 will be found economical only on new roads.

5.1. Depressing the Level of Subsoil Water by Drainage Measures

Satisfactory results could be achieved by providing 5 to 6 ft deep drainage channels as close to the road bank as possible and connecting these by suitable outfalls to either channels of irrigation system or natural drainage. Alternatively, buried drains of suitable design such as French drains could be provided at the edges of the pavement for the lowering of water-table. Either of these measures will help

in keeping the top of the subgrade above the capillary fringe.

The method of drainage is applicable to all types of road construction (whether rigid or flexible), and should be preferred wherever economically feasible.

5.2. Raising of the Embankment

Where it is too expensive to provide deep drainage channels as specified in para 5.1, it is recommended that subject to a careful examination of the economics of the case, an embankment, of such height may be provided that the bottom of the pavement remains at least 1.5 metres above the highest water-table.

5.3. Capillary Cutoff

As an alternative to the recommendations contained in paras 5.1 and 5.2, a capillary cutoff could be provided to arrest the capillary rise of water. Provision of capillary cutoffs could, however, prove to be expensive, and may be justified only in special circumstances.

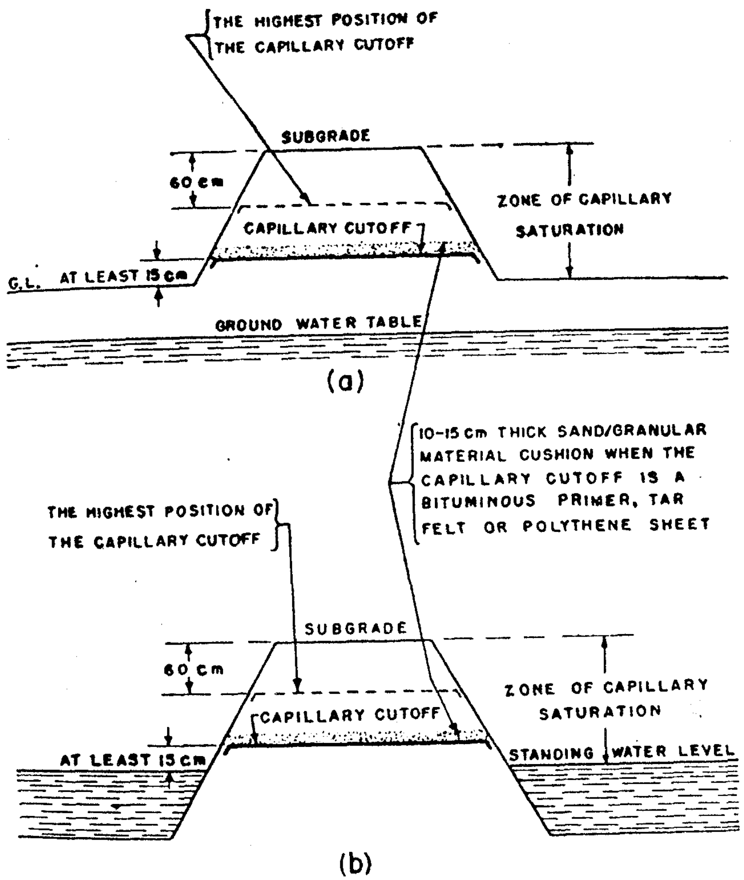
The cutoff should be placed at least 15 cm above the ground level or the standing water level, whichever be higher, as illustrated in figure on page 6. But in no case should it be positioned higher than 60 cm below the top of the subgrade. When provided, the cutoff medium should extend under the berms as well.

Suitable types of capillary cutoffs are listed under Section 8. When the cutoff medium selected is of the type bituminous primer, tar felt or polythene envelope, it will be advisable to cover it with a 10 to 15 cm thick layer of granular material like sand for the dual purpose of acting as a drainage course for water infiltrating from the top and of protecting the envelope during construction against rupture by sharp particles in the fill material.

5.4. Providing Sufficient Thickness of the Pavement to be Adequate for Saturated Subgrade Conditions

In case neither drains nor sufficiently high embankment nor capillary cutoff can be provided on an existing road, the thickness of the pavement should be determined on the basis of strength of the subgrade soil at saturation and strengthening carried out accordingly.

This measure can be adopted in the case of new constructions as well. In that event, at least 10 cm thickness of sub-base should be made up of such material as stabilised soil which will be stable when wet, so that it is not possible for the subgrade soil in saturated condition to work up into the void



SKETCHES ILLUSTRATING DESIRED POSITION OF THE CAPILLARY CUTOFF FOR PREVENTING THE RISE OF CAPILLARY MOISTURE

6. RECOMMENDATIONS FOR ROAD CONSTRUCTION IN AREAS WHERE IN ADDITION TO WATERLOGGING FLOODING FOR PROLONGED PERIODS IS ALSO EXPECTED

In the case of roads subject to flooding in addition to waterlogging, the following measures may have to be taken over and above those recommended in Section 5 against waterlogging. The three treatments suggested could be applied equally on new constructions as well as old.

6.1. Raising of the Embankment

In areas subject to frequent floods where the highest flood level is not too much above the natural ground level, it is recommended that the embankment should be raised so that the top of the subgrade is at least 30 cm above the highest recorded flood level.

6.2. Provision of Cement/Asphaltic Concrete Surfacing

Where for any reason whatsoever it is considered inevitable to let flood waters pass over the road, and also traffic is heavy and flooding expected for prolonged periods, cement/asphaltic concrete surfacing of appropriate thickness should be provided for at least two lanes of traffic. The cement concrete pavement, when provided, should have a cement/lime soil base 15 cm thick underneath the slab and permanent stakes on the carriageway edges for demarcating the travelled way. When asphaltic concrete is selected as the surfacing, the mix should be dense graded and resistant to flood conditions.

Where formation level of the road is well above the surrounding ground level, side drop walls and guide uprights must also be provided. In addition, the banks should be protected against erosion.

6.3. Provision of a Thin Bituminous Surfacing with Seal Coat

Where traffic is very light and the provision of a cement/asphaltic concrete surface is considered unjustified on economic grounds, a tolerable solution will be to provide a thin bituminous surfacing with seal coat over at least two lane width, using suitable anti-stripping agents and supplemented by guide uprights and side drop walls as necessary.

7. RECOMMENDATIONS FOR ROAD CONSTRUCTION IN AREAS WHERE IN ADDITION TO WATERLOGGING DETRIMENTAL SALTS ARE PRESENT IN THE SUBSOIL OR GROUND WATER

The following recommendations apply generally to construction of new roads. On existing roads, measures outlined in paras 5.1, 5.2 and 5.4 may be adopted for relief.

7.1. No special measures are considered necessary from the standpoint of physical/chemical action of injurious salts except those stated in Sections 5 and 6 if the concentration of sulphates in the subgrade soil is below 0.2 per cent (as sulphur trioxide) while also below 0.03 per cent (as sulphur trioxide) in ground water. Similarly, sodium carbonate concentrations of upto 0.2 per cent in subgrade soil and 0.02 per cent in the ground water are considered unharmful. Salt concentrations may be determined in accordance with the procedure laid down in relevant I.S.I. Standards—IS : 2720, part XXIII —1966 "Methods of Test for Soils : Determination of Calcium Carbonate", and IS : 2720, part XXVII—"Methods of Test for Soils : Determination of Total Sulphate".

No damage is expected from dissolved carbon dioxide or bicarbonate salt solutions (met with in certain marshy areas) provided the pH value of the solutions is higher than 5.

Where the concentration of these salts is in excess of the safe limits specified above, special measures as indicated below are recommended as a guide for road construction. These measures are in addition to those recommended in Section 5.

7.2. Flexible Pavements

7.2.1. **Water-bound macadam roads with or without bituminous surfacing:** Even if concentration of salts in the subgrade or ground water is higher than the safe limits prescribed in para 7.1, no special measures other than those set forth in Section 5 are considered necessary for water-bound macadam roads with or without bituminous surfacing except that the filler used in water-bound macadam and soling should be inert and free from injurious salts.

7.2.2. **Stabilised Soil Constructions:**

(a) **Mechanical stabilisation**

(b) **Cement and lime stabilisation**

(c) **Bituminous stabilisation**

If the above constructions are contemplated in waterlogged areas infested with salts, the soil used for stabilisation should not contain more than 0.2 per cent of total soluble sulphates and carbonates.

Besides this, to prevent the injurious salts in the subgrade or ground water from coming into contact with stabilised soil courses, a suitable capillary cutoff out of those described in Section 8 should be provided underneath the pavement extending across the full width of the roadway, treating it as an essential measure.

7.3. Rigid Pavements

When sulphates are in excess of the safe limits prescribed in para 7.1, the following additional measures are recommended during the construction of cement concrete pavements over and above the provisions of paras 5.1 and 5.2.

7.3.1. Since all types of concrete, irrespective of the type of cement used, are more vulnerable to salt attack during the initial period of hardening than when fully set, it is of importance to prevent contact between the ground water and concrete in the early stages. For this purpose, applying a light coat of bitumen to the underside of precast units, protecting cast-in-situ concrete by a thin bituminised coating on the base just below the slab, or provision of one of the capillary cutoffs mentioned in Section 8, are some of the measures recommended for adoption under relatively mild conditions of exposures to sulphate attack, viz., when sulphate concentration in soil is upto about 0.5 per cent.

Under more severe conditions, i.e. when sulphates are in excess of 0.5 per cent, the bituminous coatings used should be thicker* as they are known to possess higher durability.

7.3.2. Furthermore, the following measures are suggested as suitable for minimising adverse chemical effect of the sulphates on concrete:

(i) Designing a dense, well-compacted, high quality concrete which will have low permeability aginst ingress of sulphate solution. (This is recommended even when SO_3 in water is above 0.02 per cent).

(ii) Use of special sulphate resistant cement, puzzolanic cement or super-sulphated blast furnace slag cement, depending on availability and economy (when sulphate content is more than 0.3 per cent in soil and more than 0.03 per cent in ground water).

7.3.3. In areas where there is danger of damage from dissolved carbon dioxide or bicarbonate salts as evidenced by pH values of below 5, the provision of a waterproof layer below the concrete pavement, such as heavy duty bituminised paper or polythene sheet, and use of a dense, well-compacted, high quality concrete are the measures recommended for adoption.

*For the purposes of this specification, thin coats are considered to be those in which the rate of application of straight run bitumen is 12 kg per 10 m² and thick coats are those in which the rate of application is 20 kg per 10 m².

8. SUITABLE CAPILLARY CUTOFFS

8.1. Provision of Sand Blanket

Sand blanket of adequate thickness over the full width of embankment is recommended as an effective capillary cutoff. The thickness of the sand blanket needed to intercept capillarity depends on the particle size of the sand and may be determined from the following formula*:

$$t = \frac{(8)^{0.92}}{d}$$

where t = thickness of sand layer in cm

d = $\frac{2d_1 \times d_2}{d_1 + d_2}$

= mean particle diameter in mm

d_1 = aperture size of sieve (mm) through which the fraction passes

d_2 = aperture size of sieve (mm) on which the fraction is retained.

The sand shall be compacted after adding sufficient moisture to permit easy rolling. Alternatively, it might be compacted dry if the facility of vibratory roller was available.

8.2. Some of the Other Capillary Cutoffs**

8.2.1. **Bituminous impregnation using primer treatment:** 50 per cent straight-run bitumen (80-100) with 50 per cent high speed diesel oil or its equivalent in two applications of 10 kg per 10 m^2 each, allowing the first application to penetrate before applying the second one.

8.2.2. **Heavy duty tar felt:** Providing an envelope with heavy duty tar felt.

8.2.3. **Polythene envelope:** Providing an envelope with polythene sheets of at least 400 gauge.

8.2.4. **Bituminous stabilised soil:** Providing bituminous stabilised soil in a thickness of at least 4 cm.

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*This formula was proposed initially by the Public Roads Administration and is published in Highway Research Board Proceedings, Vol. 21, 1941, page 452.

**Experience on the successful performance of capillary cutoffs suggested in para 8.2 is, however, limited.

IRC:37-2

GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS

(Second Revision)

THE INDIAN ROADS CONGRESS
2001

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GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS

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ABBREVIATIONS

1.	AASHTO	-	American Association of State Highway and Transportation Officials
2.	BC	-	Bituminous Concrete
3.	BUSG	-	Built up Spray Grout
4.	BM	-	Bituminous Macadam
5.	CBR	-	California Bearing Ratio
6.	DBM	-	Dense Bituminous Macadam
7.	GB	-	Granular Base
8.	GSB	-	Granular Sub-Base
9.	IRC	-	Indian Roads Congress
10.	MORT&H	-	Ministry of Road Transport & Highways
11.	msa	-	Million Standard Axles
12.	SDBC	-	Semi-Dense Bituminous Concrete

GUIDELINES FOR THE DESIGN OF FLEXIBLE PAVEMENTS

1. INTRODUCTION

The design of flexible pavement involves the interplay of several variables, such as, the wheel loads, traffic, climate, terrain and sub-grade conditions. With a view to have a unified approach for working out the design of flexible pavement in the country, the IRC first brought out guidelines in 1970. These were based on California Bearing Ratio method. To handle large spectrum of axle load, these guidelines were revised in 1984 following the equivalent axle load concept. In this approach, the pavement thickness was related to the cumulative number of standard axles to be carried out for different sub-grade strengths. These guidelines were based on semi-empirical approach based on a large extent on past experience and judgement of highway agencies. Design curves were developed to cater upto 30 million standard axles.

With the rapid growth of traffic now, the pavements are required to be designed for heavy volume of traffic of the order of 150 million standard axles. In the meanwhile, an in-house software package was developed under MORT&H's Research Scheme R-56. This enabled mathematical modelling of the pavement structure using multiple layer elastic theory. With this background and the feed back on the performance of the existing designs, the Flexible Pavement Committee in 1997 set up a Sub-group consisting of the following personnel to review the existing "Guidelines for Design of Flexible Pavements". This Sub-group developed the design charts and catalogue of pavement designs for conditions prevailing in the country.

Dr. M.P. Dhir ... *(Convenor)*
S.C. Sharma
R.K. Pandey
Dr. L.R. Kadiyali
Dr. S.S. Jain
D.P. Gupta
V.K. Sood
Dr. Sunil Bose

The Sub-group discussed revision of IRC:37 in a number of meetings and finally on 10.8.98 submitted the revised draft to Flexible Pavement Committee (H-4). The draft was approved by H-4 Committee (Personnel given below) in its meeting held on 26.2.99 and the Convenor was authorised to incorporate the comments/suggestions made by the members and other experts appropriately and send the final draft to Highways Specifications & Standards Committee for consideration and approval.

Dr. A.K. Gupta	...	*Convenor* (upto 6.11.97)
S.C. Sharma	...	*Convenor* (w.e.f. 7.11.97)
Spl. Secy. & Chief Engineer R&B Deptt., Gandhinagar, (S.S. Rathore)	...	*Co-Convenor*
Dr. S.S. Jain	...	*Member-Secretary*

Members

Prof. C.G. Swaminathan	Engineer-in-Chief, Army HQ (Maj. Gen. C.T. Chari)
Dr. M.S. Srinivasan	Chief Engineer(R) S&R, MORT&H (Indu Prakash)
Prof. C.E.G. Justo	Head, Flexible Pavement Division, CRRI (I.R. Arya)
R.K. Jain	Rep. of DGBR (A.R. Aiyengar)
B.K. Dam	Engineer-in-Chief, UP PWD (Ravinder Kumar)
B.R. Tyagi	
I.C. Goel	
Dr. L.R. Kadiyali	
Prof. B.B. Pandey	
Dr. M.P. Dhir	
D.P. Gupta	

Ex-Officio Members

President, IRC (H.P. Jamdar)	DG (RD) & Addl. Secy., MORT&H (A.D. Narain)
Secretary, IRC (S.C. Sharma)	

Corresponding Members

Hari Om Prakash Sharma	I.K. Bhavsar
R.S. Shukla	Dr. P.D. Marathe
Prof. S.P. Jog	

The draft was further vetted by an Expert Group comprising of Shri S.C. Sharma, Prof. B.B. Pandey and Dr. S.S. Jain and forwarded to newly constituted Flexible Pavement Committee.

The newly constituted Flexible Pavement Committee (Personnel given below) in its meeting held on 23.9.2000 discussed and approved the revised draft of IRC:37 for placing before the Highways Specifications & Standards (HSS) Committee for approval.

S.C. Sharma	...	*Convenor*
Spl. Secy. & Chief Engr. R&B Deptt., Gandhinagar (S.S. Rathore)	...	*Co-convenor*
Dr. S.S. Jain	...	Member-Secretary

Members

D. Basu	R.K. Pandey
Dr. A.K. Bhatnagar	D. Sreerama Murthy
Dr. M.P. Dhir	Prof. C.G. Swaminathan
D.P. Gupta	Col. V.K.P. Singh
Dr. L.R. Kadiyali	Hargun Das
Prof. C.E.G. Justo	CE (R) S&R MORT&H (C.C. Bhattacharya)
H.L. Meena	
Prof. B.B. Pandey	

Ex-Officio Members

President, IRC (M.V. Patil)

DG(RD)&Addl. Secy. MORT&H (Prafulla Kumar)

Secretary, IRC (G. Sharan)

Corresponding Members

Sukomal Chakraborty

Smt. A.P. Joshi

R.S. Shukla

The HSS Committee in its meeting held on 30.9.2000 after detailed discussion approved the revised draft IRC:37 and authorised the Convenor, Flexible Pavement Committee to modify the same in light of the comments of members and submit to Convenor, HSS Committee for its approval. The draft of revised guidelines as modified by the Convenor, HSS Committee was approved by the Executive Committee in its meeting held at New Delhi on 5.10.2000 and by the Council in its meeting held at Kolkata on 4.11.2000. The draft as modified in light of comments of members of the Council was approved by the Convenor, HSS Committee on 12.2.2001 for printing.

2. SCOPE

2.1. These guidelines will apply to design of flexible pavements for Expressways, National Highways, State Highways, Major District Roads and other categories of roads predominantly carrying motorised vehicles.

2.2. For the purpose of the guidelines, flexible pavements are considered to include the pavements which have bituminous surfacing and granular base and sub-base courses conforming to IRC Standards or to Sections 500 and 400 of the Specifications

for Road and Bridge Works, Ministry of Road Transport and Highways.

2.3. These guidelines apply to new pavements.

2.4. For design of strengthening measures or overlays for existing pavements, the design procedure described in IRC:81 "Tentative Guidelines for Strengthening of Flexible Road Pavements Using Benkelman Beam Deflection Technique" shall apply.

2.5. The guidelines may require revision from time to time in the light of future experience and developments in the field. Towards this end, it is suggested that all the organisations intending to use the guidelines, should keep a detailed record of year of construction, subgrade CBR, soil characteristics, pavement composition and specifications, traffic, pavement performance, overlay history, climatic conditions, etc. and provide feedback to the Indian Roads Congress.

3. RECOMMENDED METHOD OF DESIGN

3.1. General

The pavement designs given in the previous edition IRC:37-1984 were applicable to design traffic upto 30 million standard axles (msa). With the increasing traffic and incidence of overloading, arterial roads need to be designed for traffic far greater than 30 msa. As empirical methods have limitations regarding their applicability and extrapolation, the analytical method of design has been used to reanalyse the existing designs and develop a new set of designs for design traffic upto 150 msa making use of the results of pavement research work done in the country and experience gained over the years on the performance of the existing designs.

3.2. Design Approach and Criteria

3.2.1. The flexible pavement has been modelled as a three layer structure and stresses and strains at critical locations *(Annexure-1)* have been computed using the linear elastic model FPAVE developed under the MORT&H Research Scheme, R-56 "Analytical Design of Flexible Pavements"[1].

3.2.2. To give proper consideration to the aspect of performance, the following three types of pavement distress resulting from repeated application of traffic loads are considered:

(i) Vertical compressive strain at the top of the subgrade. If the strain is excessive, the subgrade will deform resulting in permanent deformation at the pavement surface during the design life.

(ii) Horizontal tensile strain at the bottom of the bituminous layer. Large tensile strains cause fracture of the bituminous layer during the design life.

(iii) Pavement deformation within the bituminous layer.

While the permanent deformation within the bituminous layer can be controlled by meeting the mix design requirements as per the MORT&H Specification[2], thicknesses of granular and bituminous layers are selected using the analytical design approach so that strains at the critical points are within the allowable limits.

For calculating tensile strains at the bottom of the bituminous layer, the Elastic Modulus of Dense Bituminous

Macadam (DBM) layer with 60/70 bitumen has been used in the analysis. The relationships used for (i) allowable vertical subgrade strain; and (ii) allowable tensile strain at the bottom of the DBM layer along with elastic moduli of different pavement materials and the relationships for assessing the elastic moduli of subgrade, granular sub-base and base layers are given in *Annexure-1*.

3.2.3. Based on the performance of existing designs and using analytical approach, simple design charts (Figs. 1 and 2) and a catalogue of pavement designs (Plates 1 and 2) have been added for use of field Engineers. The pavement designs are given for subgrade CBR values ranging from 2 per cent to 10 per cent and design traffic ranging from 1 msa to 150 msa for an average annual pavement temperature of 35°C. The layer thicknesses obtained from the analysis have been slightly modified to adapt the designs to stage construction. Using the following simple input parameters, appropriate designs could be chosen for the given traffic and soil strength :

(i) Design traffic in terms of cumulative number of standard axles; and

(ii) CBR value of subgrade

The procedure for estimating design traffic and assessing the CBR value of the subgrade soil is described in paragraphs 3.3 and 3.4 respectively.

3.3. Traffic

3.3.1. General

3.3.1.1. The recommended method considers traffic in terms of the cumulative number of standard axles (8160 kg) to

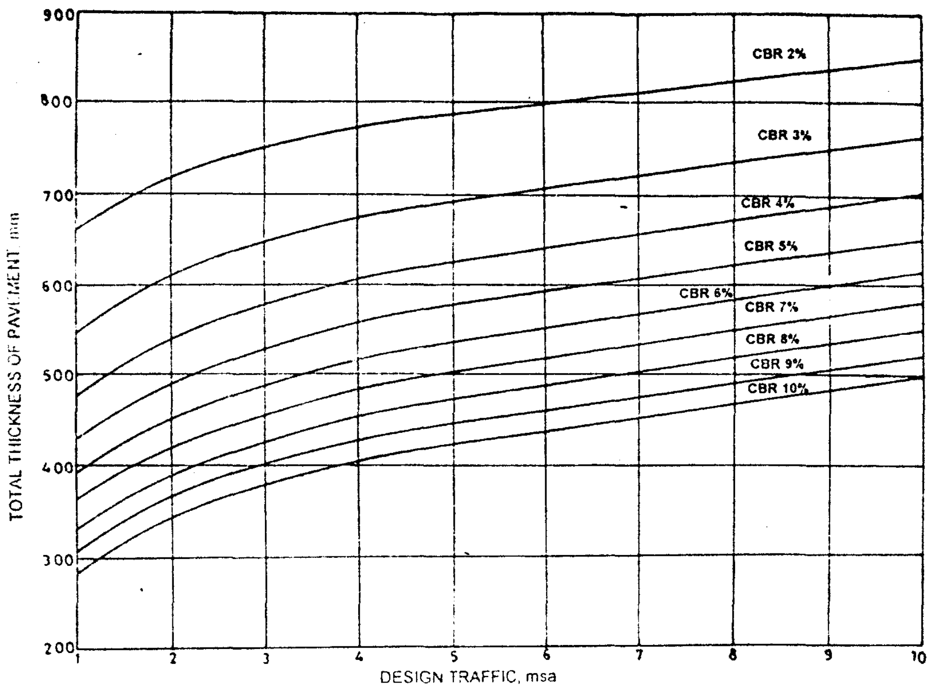


Fig. 1. Pavement Thickness Design Chart for Traffic 1-10 msa

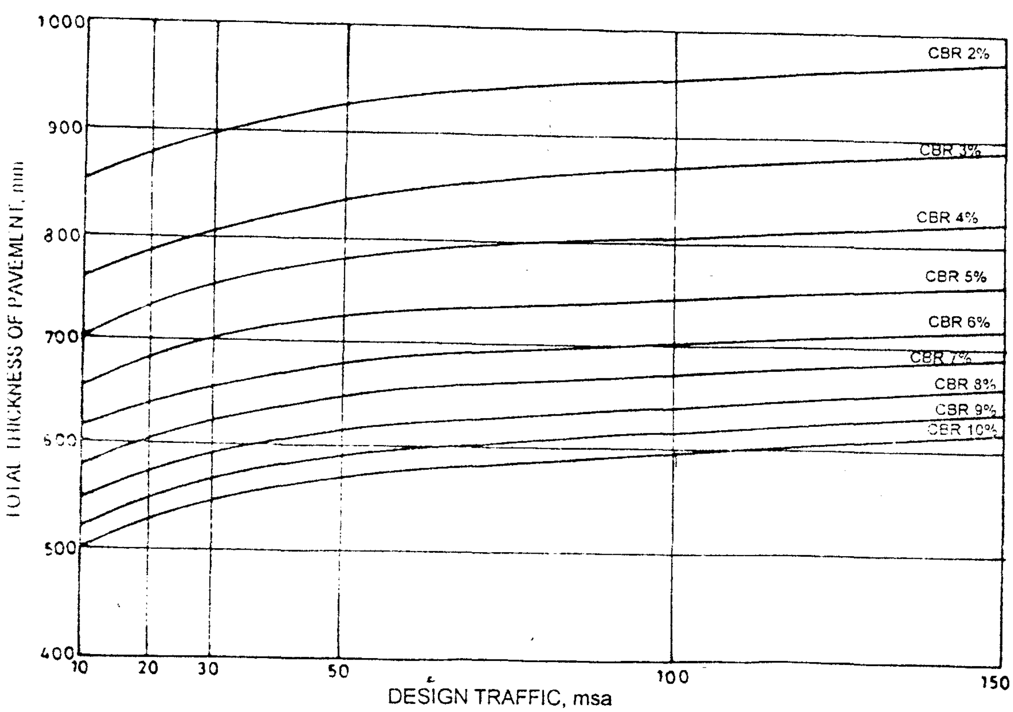


Fig. 2. Pavement Thickness Design Chart for Traffic 10-150 msa

be carried by the pavement during the design life. For estimating design traffic, the following information is needed :

(i) Initial traffic after construction in terms of number of commercial vehicles per day (CVPD)

(ii) Traffic growth rate during the design life in percentage

(iii) Design life in number of years

(iv) Vehicle damage factor (VDF)

(v) Distribution of commercial traffic over the carriageway.

3.3.1.2. For the purpose of structural design, only the number of commercial vehicles of gross vehicle weight of three tonnes or more and their axle-loading is considered.

3.3.1.3. To obtain a realistic estimate of design traffic, due consideration should be given to the existing traffic or that anticipated based on possible changes in the road network and land use of the area served, the probable growth of traffic and design life.

Estimate of the initial daily average traffic flow for any road should normally be based on atleast 7 days, 24 hour classified traffic counts. In cases of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.

3.3.2. Traffic growth rate

3.3.2.1. Traffic growth rates should be estimated :

(i) by studying the past trends of traffic growth, and

(ii) by establishing econometric models, as per the procedure outlined in IRC:108 "Guidelines for Traffic Prediction on Rural Highways".

3.3.2.2. If adequate data is not available, it is recommended that an average annual growth rate of 7.5 per cent may be adopted.

3.3.3. Design life

3.3.3.1. For the design of pavement, the design life is defined in terms of the cumulative number of standard axles that can be carried before strengthening of the pavement is necessary.

3.3.3.2. It is recommended that pavements for National Highways and State Highways should be designed
15 years. Expressways and urban roads may be des
longer life of 20 years. For other categories of roa
life of 10 to 15 years may be adopted.

3.3.3.3. Very often it is not possible to provide the full thickness of pavement right at the time of initial construction. Stage construction techniques should be resorted to in such cases.

3.3.4. Vehicle damage factor

3.3.4.1. The vehicle damage factor (VDF) is a multiplier to convert the number of commercial vehicles of different axle loads and axle configuration to the number of standard axle load repetitions. It is defined as equivalent number of standard axles per commercial vehicle. The VDF varies with the vehicle axle configuration, axle loading, terrain, type of road and from region to region. The VDF is arrived at from axle load surveys on typical road sections so as to cover various influencing factors, such as traffic mix, mode of transportation, commodities carried, time of the year, terrain, road conditions and degree of enforcement.

3.3.4.2. The axle load equivalency factors recommended in the AASHTO guide are given in *Annexure-2*. They are used for converting different axle load repetitions into equivalent standard axle load repetitions.

3.3.4.3. For designing a new pavement, the VDF should be arrived at carefully by carrying out specific axle load surveys on the existing roads. Some surveys have been carried out in the country on National Highways, State Highways and Major District Roads which reveal excessive overloading of commercial vehicles. Therefore, it is recommended that the designer should take the realistic values of VDF after conducting the axle load survey, particularly in the case of major projects. On some sections, there may be significant difference in axle loading in two directions of traffic. In such situations, the VDF should be evaluated direction wise to determine the lanes which are heavily loaded for the purpose of design.

3.3.4.4. Where sufficient information on axle loads is not available and the project size does not warrant conducting an axle load survey, the indicative values of vehicle damage factor as given in Table 1 may be used.

Table 1. Indicative VDF Values

Initial traffic volume in terms of number of commercial vehicles per day	Terrain: Rolling/Plain	Terrain: Hilly
0-150	1.5	0.5
150-1500	3.5	1.5
More than 1500	4.5	2.5

3.3.5. Distribution of commercial traffic over the carriageway

3.3.5.1. A realistic assessment of distribution of commercial traffic by direction and by lane is necessary as it directly affects the total equivalent standard axle load applications used in the design. In the absence of adequate and conclusive data for Indian conditions, it is recommended that for the time being the following distribution may be assumed for design until more reliable data on placement of commercial vehicles on the carriageway lanes are available :

(i) **Single-lane roads**

Traffic tends to be more channelised on single-lane roads than two-lane roads and to allow for this concentration of wheel load repetitions, the design should be based on total number of commercial vehicles in both directions.

(ii) **Two-lane single carriageway roads**

The design should be based on 75 per cent of the total number of commercial vehicles in both directions.

(iii) **Four-lane single carriageway roads**

The design should be based on 40 per cent of the total number of commercial vehicles in both directions.

(iv) **Dual carriageway roads**

The design of dual two-lane carriageway roads should be based on 75 per cent of the number of commercial vehicles in each direction. For dual three-lane carriageway and dual four-lane carriageway, the distribution factor will be 60 per cent and 45 per cent respectively.

3.3.5.2. The traffic in each direction may be assumed to be half of the sum in both directions when the latter only is known. Where significant difference between the two streams can occur, condition in the more heavily trafficked lane should be considered for design.

Where the distribution of traffic between the carriagway lanes and axle loads spectrum for the carriageway lanes are available, the design should be based on the traffic in the most heavily trafficked lane and the same design will normally be applied for the whole carriageway width.

3.3.6. **Computation of design traffic**

3.3.6.1. The design traffic is considered in terms of the cumulative number of standard axles (in the lane carrying maximum traffic) to be carried during the design life of the road. This can be computed using the following equation :

$$N = \frac{365 x [(1+r)^n - 1]}{r} x\ A\ x\ D\ x\ F$$

where,

N = The cumulative number of standard axles to be catered for in the design in terms of msa.

A = Initial traffic in the year of completion of construction in terms of the number of commercial vehicles per day.

D = Lane distribution factor (as explained in para 3.3.5)
F = Vehicle damage factor
n = Design life in years
r = Annual growth rate of commercial vehicles (for 7.5 per cent annual growth rate, r = 0.075)

The traffic in the year of completion is estimated using the following formula :

$$A = P(1 + r)^x$$

where,

P = Number of commercial vehicles as per last count.
x = Number of years between the last count and the year of completion of construction.

3.4. **Subgrade**

3.4.1. The subgrade whether in cut or fill should be well compacted to utilise its full strength and to economise thereby on the overall thickness of pavement required. For Expressways, National Highways, State Highways and Major District Roads, heavy compaction is recommended. Most of the specifications prescribe use of selected material and stiffer standards of compaction in the subgrade (top 500 mm portion of the roadway). The current MORT&H Specification for Road & Bridge Works (Third Revision 1995) recommend that the subgrade shall be compacted to 97 per cent of dry density achieved with heavy compaction (modified proctor density) as per IS:2720 (Part 8). This density requirement is recommended for subgrade compaction for Expressways, National Highways, State Highways, Major District Roads and other heavily trafficked roads. In other cases the subgrade should be compacted to atleast 97 per cent of the standard proctor density conforming to IS:2720 (Part 7). These requirements should be strictly enforced. IRC:36 "Recommended Practice for the Construction of Earth Embankments for Road Works" should be followed for guidance during planning and execution of work.

3.4.2. For high category roads, like, Expressways, National Highways and State Highways, the material used for subgrade construction should have the dry density of not less than 1.75 gm/cc.

3.4.3. For design, the subgrade strength is assessed in terms of the CBR of the subgrade soil in both fill and cut sections at the most critical moisture conditions likely to occur in-situ.

3.4.4. For determining the CBR value, the standard test procedure should be strictly adhered to. This is described in IS:2720 (Part 16) "Methods of Test for Soils; Laboratory Determination of CBR". The test must always be performed on remoulded samples of soils in the laboratory. Wherever possible the test specimens should be prepared by Static Compaction but if not so possible dynamic method may be used as an alternative. Both procedures are described in brief in *Annexure-3*. In-situ tests are not recommended for design purposes as it is not possible to satisfactorily simulate the critical conditions of dry density and moisture content in the field.

3.4.5. Selection of dry density and moisture content for test specimen

3.4.5.1. For a given soil, the CBR value and consequently the design, will depend largely on the density and moisture content of the test sample. Therefore, the test conditions should reproduce as closely as possible the weakest conditions likely to occur under the road after construction.

3.4.5.2. The samples of soil collected from selected borrow pits for fill sections or from subgrade level at cut sections

should be compacted to a dry density corresponding to the minimum state of compaction likely to be achieved in practice having regard to the compaction equipment used and the compaction limits specified.

3.4.5.3. The moisture condition of the subgrade which the test sample is expected to simulate is governed by local environmental factors, such as, the water table, precipitation, soil permeability, drainage conditions and the extent to which the pavement is waterproof. Thin surfacings do not always seal the pavement effectively against ingress of water. Further, the berms and verges are usually unsurfaced, and if not kept in well-maintained state to the requisite cross-fall, will enable surface water to percolate into the subgrade from near the edges of the pavement, leading to weak subgrade conditions.

Hence, it is recommended that as a general practice, the design for new construction should be based on the strength of the samples prepared at the values of prescribed dry density and moisture content obtained in accordance with IS:2720 (Part 8) or (Part 7) as the case may be and soaked in water for a period of four days prior to testing. Use of expansive clays is not allowed for subgrade construction particularly for heavily trafficked roads. As far as possible, a non-expansive soil should be used for the subgrade. Where use of expansive clays is unavoidable, the compaction requirements and additional measures as discussed in *Annexure-4* should be followed.

3.4.5.4. However, it should be realised that soaking for four days may be an unrealistically severe moisture condition in certain cases, where the climate is arid throughout the year, i.e., the annual rainfall is of the order of 500 mm or less and the water table is too deep to affect the subgrade adversely. It is anticipated that in this situation the most severe moisture condition in the field will be far below that of the sample at the end of four days soaking, resulting in unduly conservative designs if soaking procedure is adopted. In such cases, the specimens for finding the CBR value may be prepared at the natural moisture content of the soil at subgrade depth immediately after recession of the monsoon.

3.4.6. Use of test results for design and the minimum number of tests required

3.4.6.1. The design should be based on the CBR value of the weakest soil type proposed to be used for subgrade construction or encountered extensively at subgrade level over a given section of the road, as revealed by the soil surveys. Pavement thickness on new roads may be modified at intervals as dictated by the soil changes but generally it will be found inexpedient to do so frequently from practical considerations.

3.4.6.2. It is possible that in certain soil types or under abnormal conditions the measured CBR values may appear doubtful and not truly representative of the strength of soil. A more complete study of the soil may be warranted in such cases to arrive at a more reliable design.

3.4.6.3. The design evolved should be revised during construction phase if found necessary on account of the field compaction being lower than that considered in the initial design. In addition, the alternative of retaining local areas of soft soil or soil not meeting prescribed compaction level should also be considered.

3.4.6.4. As the reproducibility of the CBR results is dependent on a number of factors, wide variations in values can be expected. Therefore, atleast three samples should be tested on each type of soil at the same density and moisture content. This will enable a reliable average value to be obtained in most cases. To weed out erratic results, permissible maximum variation within the CBR values from the three specimens is indicated in Table 2.

TABLE 2. PERMISSIBLE VARIATION IN CBR VALUE

CBR (per cent)	Maximum variation in CBR value
5	±1
5-10	±2
11-30	±3
31 and above	±5

Where variation is more than the above, the design CBR value should be the average of test results from atleast six samples and not three.

4. PAVEMENT THICKNESS AND COMPOSITION

4.1. Pavement Thickness Design Charts

For the design of pavements to carry traffic in the range of 1 to 10 msa, the Pavement Thickness Chart is given in Fig. 1 and for traffic in the range of 10-150 msa, the Pavement Thickness Design Chart is given in Fig. 2. The design curves relate pavement thickness to the cumulative number of standard axles to be carried over the design life for CBR values of subgrade ranging from 2 per cent to 10 per cent. The thickness deduced from Fig. 1 or Fig. 2 for the given CBR value and design traffic is the total pavement thickness to be provided and consists of granular sub-base, granular base and bituminous surfacing. The requirements for the component layers are given in paragraph 4.2. Based on these, the recommended designs giving minimum thickness and compositions of pavement layers for new constructions are given in the Pavement Design Catalogue, Plates 1 and 2. The design procedure is illustrated with examples in paragraph 7.

4.2. Pavement Composition

4.2.1. Sub-base course

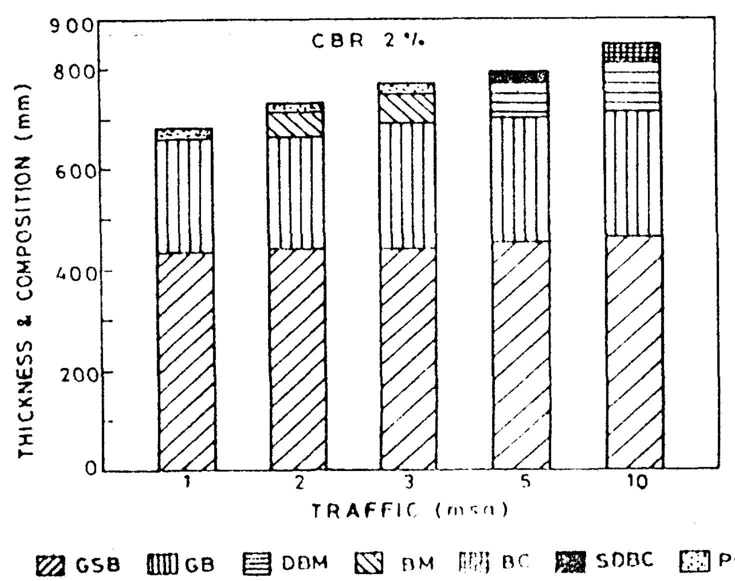
4.2.1.1. Sub-base materials comprise natural sand, moorum, gravel, laterite, kankar, brick metal, crushed stone, crushed slag, crushed concrete or combinations thereof meeting the prescribed grading and physical requirements. When the sub-base material consists of combination of materials, mixing shall be done mechanically either using a suitable mixer or adopting mix-in-place method.

Granular sub-base materials conforming to Clause 401 of MORT&H Specifications for Road and Bridge Works are recommended for use. These specifications suggest three gradings each for close and coarse graded granular sub-base materials and specify that the materials passing 425 micron sieve when tested in accordance with IS:2720 (Part 5) should have liquid limit and plasticity index of not more than 25 and 6 respectively. These requirements and the specified grain size distribution of the sub base material should be strictly enforced in order to meet stability and drainage requirements of the granular sub-base layer.

PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 2%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	660	20 PC		225	435
2	715	20 PC	50 BM	225	440
3	750	20 PC	60 BM	250	440
5	795	25 SDBC	70 DBM	250	450
10	850	40 BC	100 DBM	250	460



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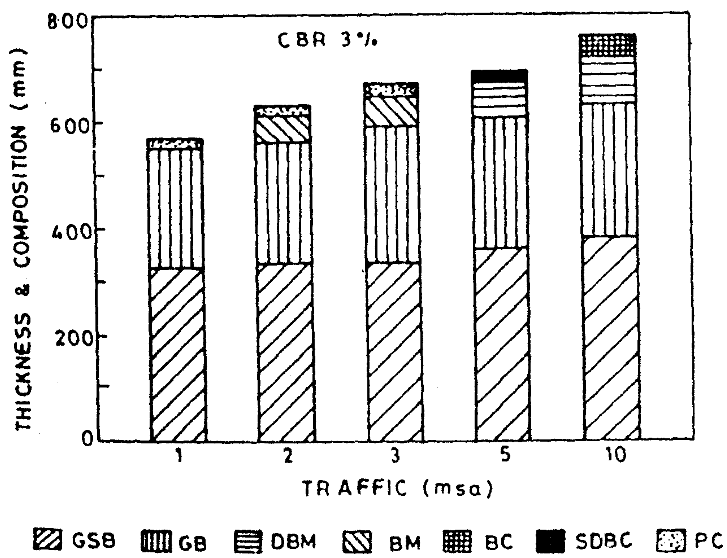
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 3%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	550	20 PC		225	435
2	610	20 PC	50 BM	225	335
3	645	20 PC	60 BM	250	335
5	690	25 SDBC	60 DBM	250	335
10	760	40 BC	90 DBM	250	380



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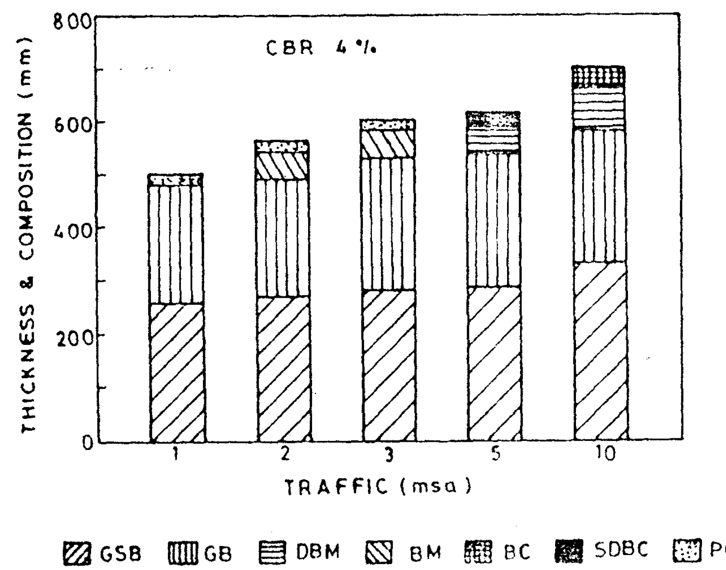
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 4%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	480	20 PC		225	255
2	540	20 PC	50 BM	225	265
3	580	20 PC	50 BM	250	280
5	620	25 SDBC	60 DBM	250	285
10	700	40 BC	80 DBM	250	330



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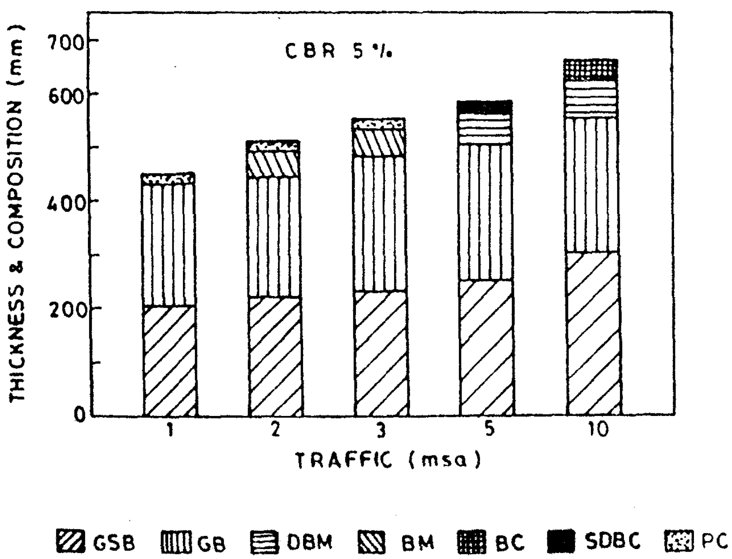
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 5%

Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION: Bituminous Surfacing – Wearing Course (mm)	Bituminous Surfacing – Binder Course (mm)	Granular Base (mm)	Granular Sub-base (mm)
1	430	20 PC		225	205
2	490	20 PC	50 BM	225	215
3	530	20 PC	50 BM	250	230
5	580	25 SDBC	55 DBM	250	250
10	660	40 BC	70 DBM	250	300



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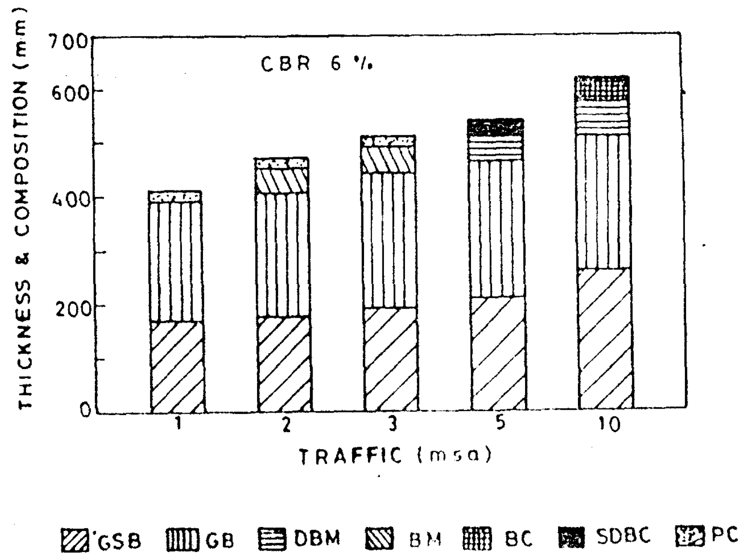
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 6%

Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION: Bituminous Surfacing – Wearing Course (mm)	Bituminous Surfacing – Binder Course (mm)	Granular Base (mm)	Granular Sub-base (mm)
1	390	20 PC		225	165
2	450	20 PC	50 BM	225	175
3	490	20 PC	50 BM	250	190
5	535	25 SDBC	50 DBM	250	210
10	615	40 BC	65 DBM	250	260



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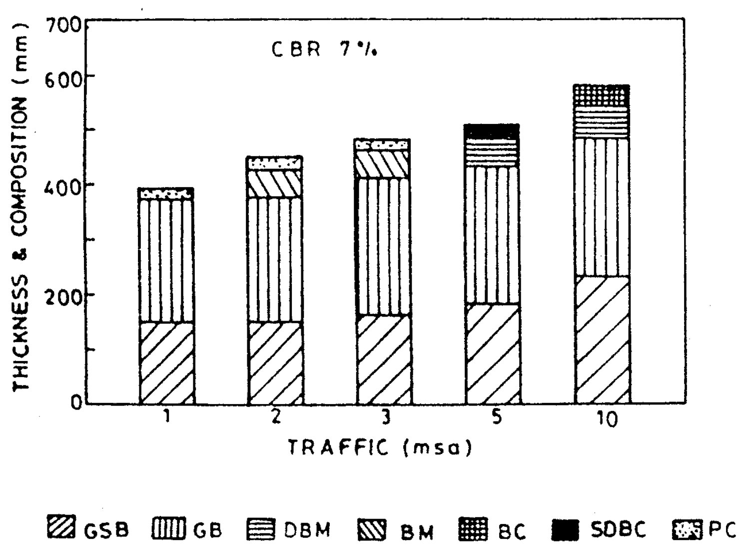
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 7%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	375	20 PC		225	150
2	425	20 PC	50 BM	225	150
3	460	20 PC	50 BM	250	160
5	505	25 SDBC	50 DBM	250	180
10	580	40 BC	60 DBM	250	230



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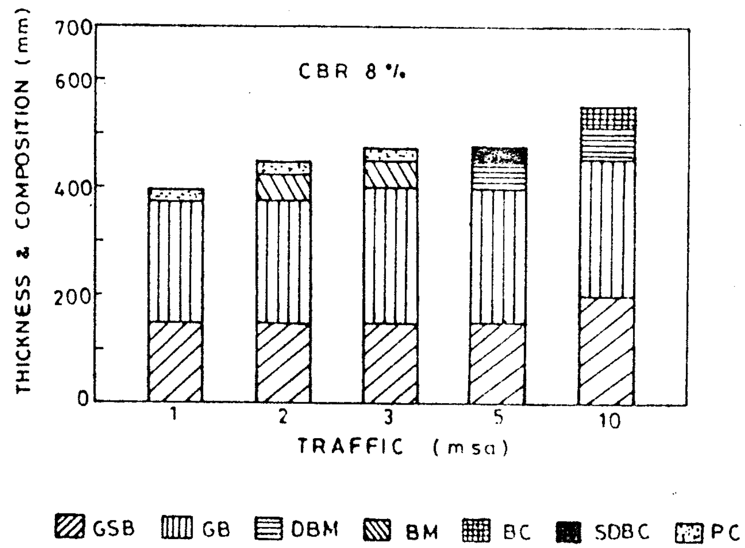
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

CBR 8%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	375	20 PC		225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	550	40 BC	60 DBM	250	200



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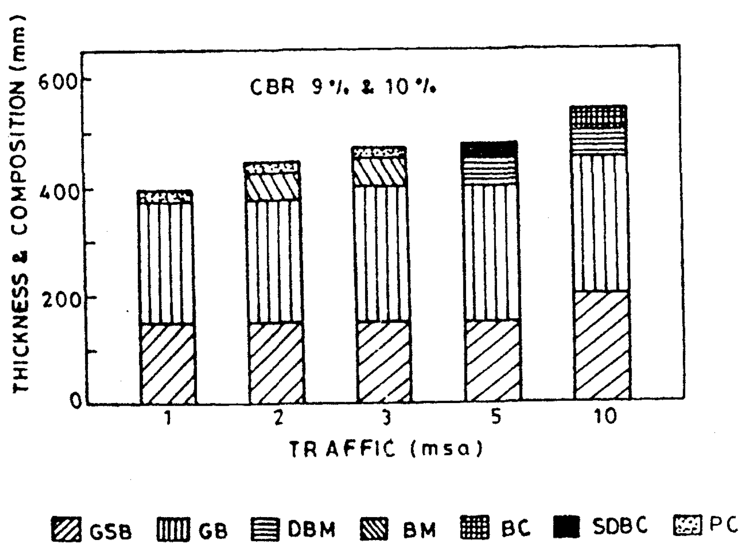
PAVEMENT DESIGN CATALOGUE

PLATE 1 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 1-10 msa

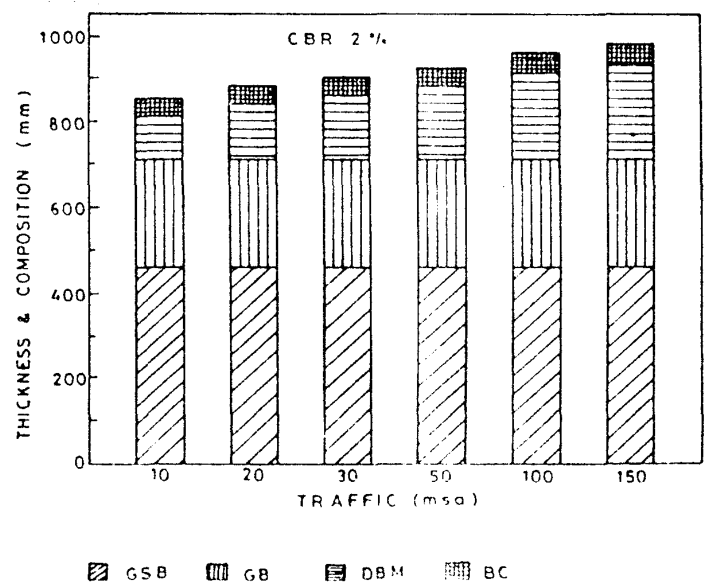
CBR 9% & 10%					
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION			
		Bituminous Surfacing		Granular Base (mm)	Granular Sub-base (mm)
		Wearing Course (mm)	Binder Course (mm)		
1	375	20 PC		225	150
2	425	20 PC	50 BM	225	150
3	450	20 PC	50 BM	250	150
5	475	25 SDBC	50 DBM	250	150
10	540	40 BC	50 DBM	250	200



PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 2%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	850	40	100	
20	880	40	130	
30	900	40	150	Base = 250
50	925	40	175	
100	955	50	195	Sub-base = 460
150	975	50	215	



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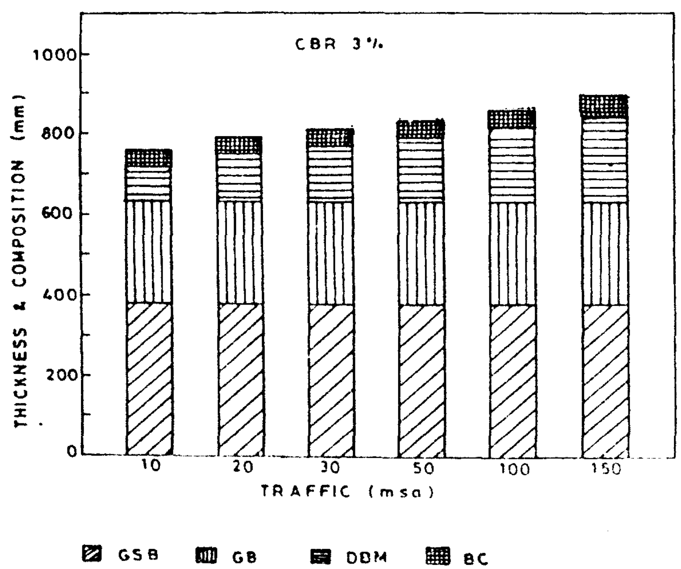
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 3%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	760	40	90	
20	790	40	120	
30	810	40	140	Base = 250
50	830	40	160	
100	860	50	180	Sub-base = 380
150	890	50	210	



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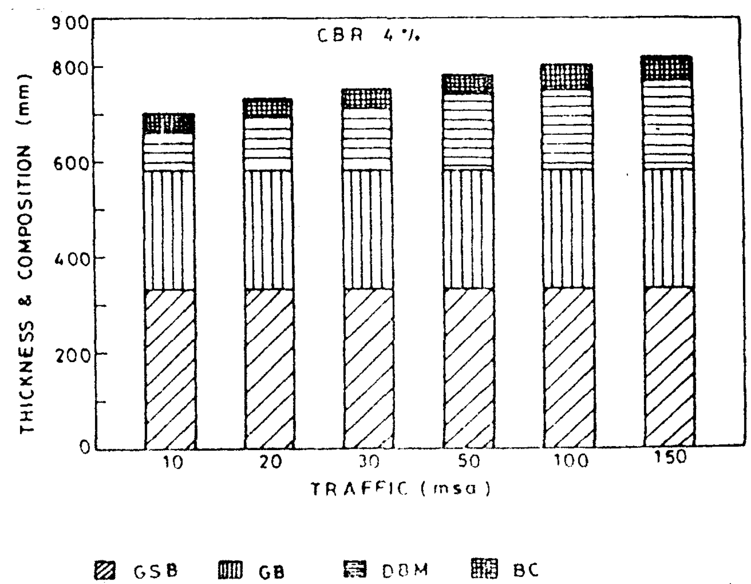
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 4%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	700	40	80	
20	730	40	110	
30	750	40	130	Base = 250
50	780	40	160	
100	800	50	170	Sub-base = 330
150	820	50	190	



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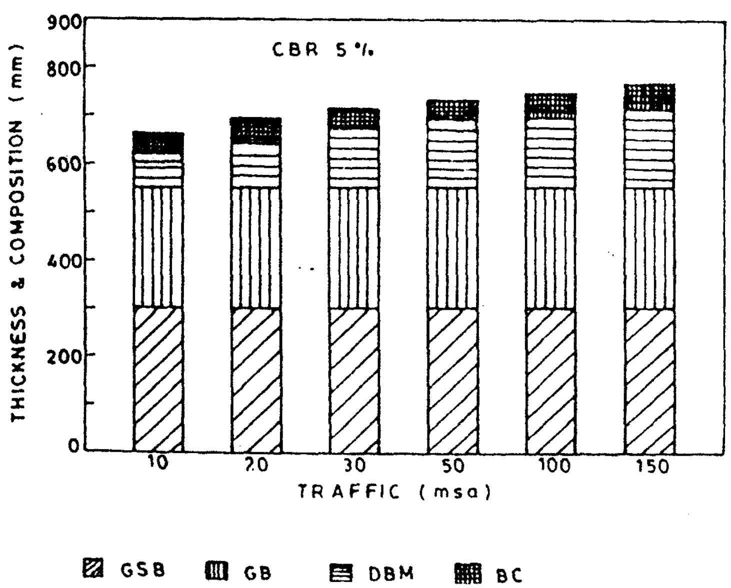
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 5%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	660	40	70	
20	690	40	100	
30	710	40	120	Base = 250
50	730	40	140	
100	750	50	150	Sub-base = 300
150	770	50	170	



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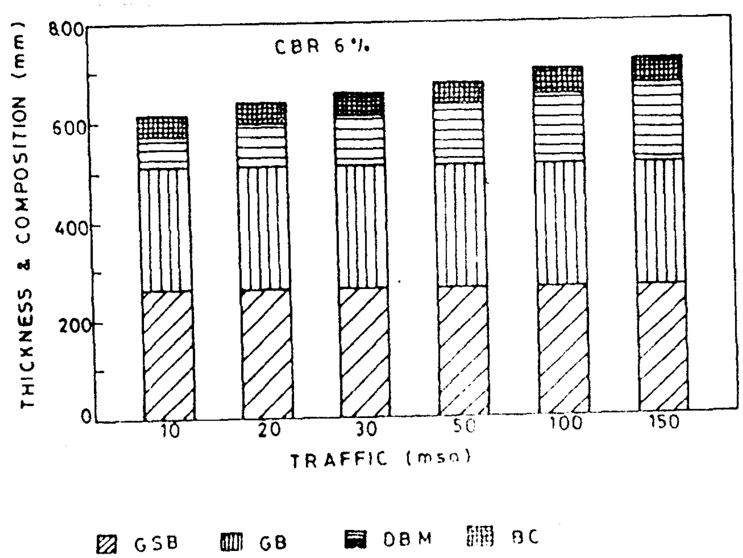
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 6%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	615	40	65	
20	640	40	90	
30	655	40	105	Base = 250
50	675	40	125	
100	700	50	140	Sub-base = 260
150	720	50	160	



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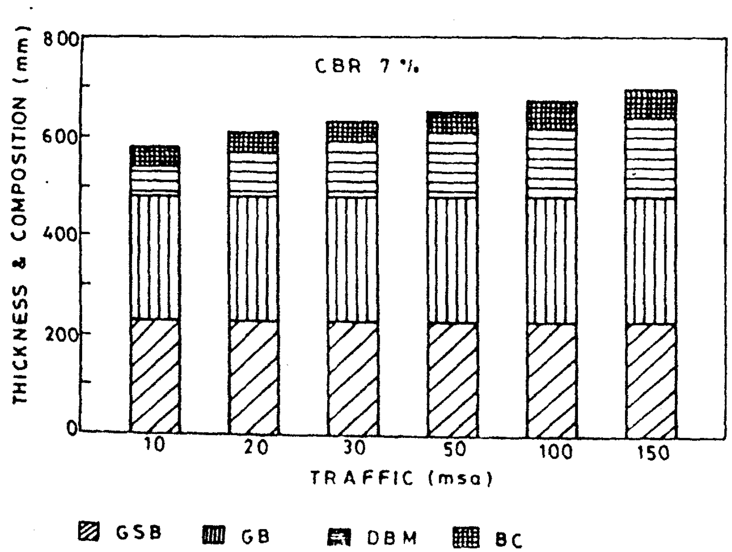
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 7%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	580	40	60	Base = 250
20	610	40	90	
30	630	40	110	
50	650	40	130	Sub-base = 230
100	675	50	145	
150	695	50	165	



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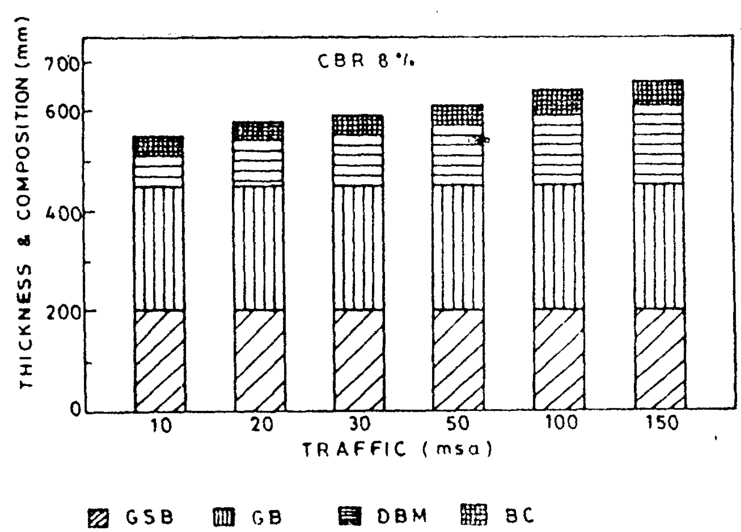
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 8%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	550	40	60	Base = 250
20	575	40	85	
30	590	40	100	
50	610	40	120	Sub-base = 200
100	640	50	140	
150	660	50	160	



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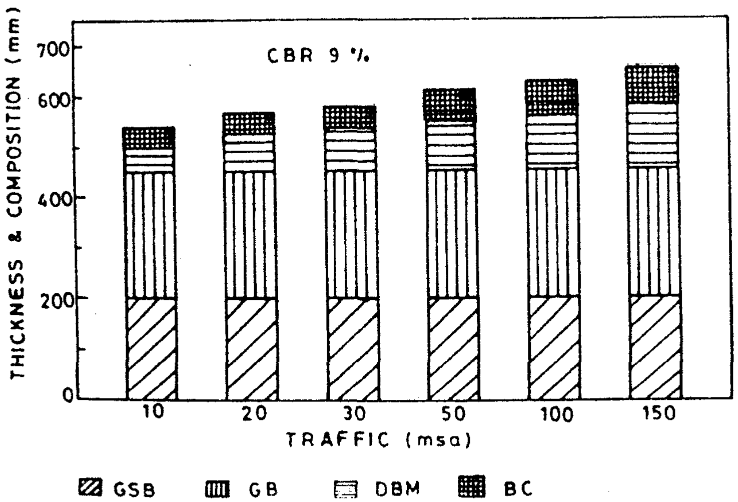
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 9%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	540	40	50	
20	570	40	80	
30	585	40	95	Base = 250
50	605	40	115	
100	635	50	135	Sub-base = 200
150	655	50	155	



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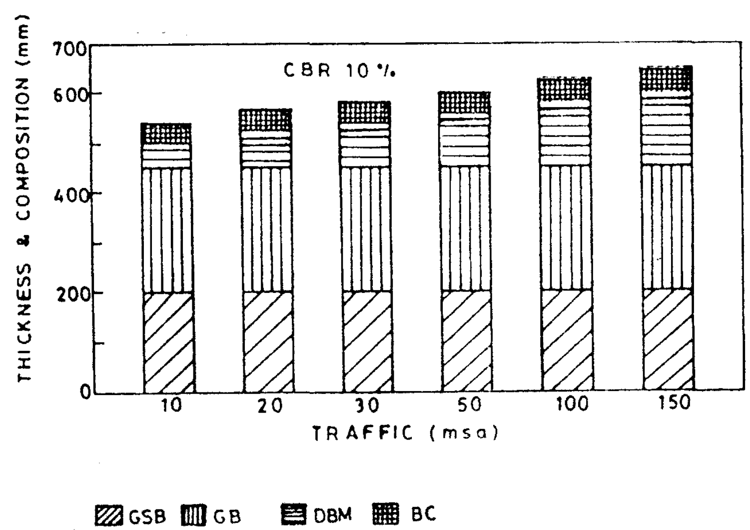
PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 10%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	540	40	50	
20	565	40	75	
30	580	40	90	Base = 250
50	600	40	110	
100	630	50	130	Sub-base = 200
150	650	50	150	



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The sub-base material should have minimum CBR of 20 per cent for cumulative traffic upto 2 msa and 30 per cent for traffic exceeding 2 msa. It should be ensured prior to actual execution that the material to be used in sub-base satisfies the CBR and other prescribed physical requirements. The material should be tested for CBR at the dry density and moisture content expected in the field. Where soaking conditions apply for design, the minimum strength of the sub-base material should be determined after soaking the test specimen in water for four days.

4.2.1.2. Where the granular sub-base material conforming to the above specifications is not available economically, other granular sub-bases, like, Water Bound or Wet Mix Macadam conforming to MORT&H Specifications are recommended.

4.2.1.3. From drainage considerations the granular sub-base should be extended over the entire formation width in case the subgrade soil is of relatively low permeability. The thickness of sub-base in the extended portions should not be less than the prescribed minimum as given in para 4.2.1.4.

4.2.1.4. The thickness of sub-base should not be less than 150 mm for design traffic less than 10 msa and 200 mm for design traffic of 10 msa and above.

4.2.1.5. Preferably the subgrade soil should have a CBR of 2 per cent. Where the CBR value of the subgrade is less than 2 per cent, the design should be based on subgrade CBR value of 2 per cent and a capping layer of 150 mm thickness of material with a minimum CBR of 10 per cent shall be provided in addition to the sub-base.

4.2.1.6. Where stage construction is adopted for pavements, the thickness of sub-base shall be provided for ultimate pavement section for the full design life.

4.2.1.7. In the areas affected by frost, care should be taken to avoid using frost susceptible materials in the sub-base.

4.2.2. Base course

4.2.2.1. Unbound granular bases which comprise conventional Water Bound Macadam (WBM), Wet Mix Macadam (WMM) or other equivalent granular construction conforming to IRC/MORT&H Specifications shall be adopted.

4.2.2.2. Materials for use in the base course must satisfy the grading and physical requirements prescribed in the IRC/MORT&H Specifications. The recommended minimum thickness of granular base is 225 mm for traffic upto 2 msa and 250 mm for traffic exceeding 2 msa.

4.2.2.3. For heavily trafficked roads, use of WMM base laid by paver finisher or motor grader is recommended.

4.2.2.4. Where WBM construction is adopted in the base course for roads carrying traffic more than 10 msa, the thickness of WBM base shall be increased from 250 mm to 300 mm (i.e., 4 layers of WBM grades II and III each of 75 mm compacted thickness) for ease of construction with corresponding reduction in the sub-base thickness keeping the overall pavement thickness unchanged as deduced from the design chart.

4.2.3. Bituminous surfacing

4.2.3.1. The bituminous surfacing shall consist of either [redacted] wearing course or a binder course with a wearing course depending upon the traffic to [redacted] The most commonly used wearing courses are surface dressing, open-graded premix carpet, mix seal surfacing, semi-dense bituminous concrete and bituminous concrete. For binder courses, the MORT&H Specification prescribes Bituminous Macadam and [redacted] Bituminous Macada[redacted] Bituminous Macadam has low binder content and high voids and is thus not impervious to water. Further the effect of high voids is reduced stiffness and increased stress concentrations. From fatigue considerations, the deterimental effect of voids is more apparent at low temperatures. On the other hand, during prolonged hot spells the average pavement temperatures are very high and consequently such a mix will operate over a very low stiffness range. Hence, the use of bituminous macadam binder course to IRC/MORT&H Specifications may desirably be restricted only to roads designed to carry traffic less than 5 msa. Dense Bituminous Macadam binder course is recommended for roads designed to carry more than 5 msa.

4.2.3.2. Recommended surfacing materials and thicknesses in terms of the cumulative standard axles to be carried during the design life are given in Plates 1 and 2. The suggested surfacings are a desirable minimum from functional and structural requirements.

4.2.3.3. However, in case the granular base is manually laid or if recommended by the Engineer, the Dense Bituminous Macadam (DBM) binder course may be preceeded by a 75 mm

thick Bituminous Macadam (BM) layer. Where this is done the thickness of the DBM layer will be suitably reduced. For practical purposes 10 mm BM can be taken as equivalent to 7 mm DBM for modifying the thickness of DBM layer.

4.2.3.4. Choice of the appropriate type of bituminous wearing course will depend on several factors, like, design traffic over the service life, the type of base/binder course provided, whether the pavement is to be built up in stages, rainfall and other related factors. The recommended types and thicknesses of wearing courses for traffic from 10 msa to 150 msa are given in Plate 2 and for traffic less than 10 msa in Plate 1, which may be read in conjunction with *Annexure-5* and may be modified if the environmental conditions and experience so justify.

4.2.3.5. The grade of bitumen will be selected keeping in view the traffic, rainfall and other environmental conditions. The selection criteria for the grade of bitumen to be used for bituminous courses are given in *Annexure-6*. Use of high performance mixes/modified binders are recommended in heavily trafficked situations.

4.2.3.6. For areas with heavy snow precipitation where mechanised snow clearance operations may be undertaken as well as at locations, like, bus stops and roundabouts, consideration ought to be given to the provision of bituminous concrete in single or multiple courses so as to render the surface more stable and waterproof. Mastic asphalt may be used at bus-stops and intersections.

4.2.3.7. Where the wearing surface adopted is open graded premix carpet of thickness upto 25 mm, the thickness of surfacing should not be counted towards the total thickness of the pavement as such surfacing will be purely for wearing and will not add to the structural capacity of the pavement.

4.3. Pavement Design Catalogue

4.3.1. Based on the results of analysis of pavement structures, practical requirements and specifications spelt out in paragraph 4.2, the recommended designs for traffic range 1 msa to 10 msa are given in Plate 1 and for traffic range 10 msa to 150 msa are given in Plate 2. In some cases, the total pavement thickness given in the recommended designs is slightly more than the thickness obtained from the design charts. This is in order to :

(a) provide the minimum prescribed thickness of sub-base

(b) adapt the design to stage construction which necessitated some adjustment and increase in sub-base thickness.

Dense Bituminous Macadam shall be constructed in two layers when the prescribed thickness is more than 100 mm.

4.3.2. The designs relate to CBR values ranging from 2 per cent to 10 per cent and ten levels of design traffic 1,2,3,5,10,20,30,50,100,150 msa. The pavement compositions given in the design catalogue are relevant to Indian conditions, materials and specifications. Where any change in layer thickness and specification is considered desirable from practical considerations, the composition can be suitably modified using analytical approach with in-service performance related information and appropriate design values.

4.3.3. For intermediate traffic ranges, the pavement layer thicknesses will be interpolated linearly.

4.3.4. For traffic exceeding 150 msa, the pavement design appropriate to 150 msa may be chosen and further strengthening carried out to extend the life at the appropriate time based on pavement deflection measurements as per IRC:81.

5. DRAINAGE MEASURES

5.1. The performance of a pavement can be seriously affected if adequate drainage measures to prevent accumulation of moisture in the pavement structure are not taken. Some of the measures to guard against poor drainage conditions are maintenance of transverse section in good shape to reasonable crossfall so as to facilitate quick run-off of surface water and provision of appropriate surface and sub-surface drains where necessary. Drainage measures are especially important when the road is in cutting or built on low permeability soils or situated in heavy rainfall/snow precipitation area.

5.2. On new roads, the aim should be to construct the pavement as far above the water table as economically practicable. The difference between the bottom of subgrade level and the level of water table/high flood level should, however, not be less than 0.6-1 m. In water logged areas, where the subgrade is within the zone of capillary saturation, consideration should be given to the installation of suitable capillary cut-off as per IRC:34 at appropriate level underneath the pavement.

5.3. When the traditional granular construction is provided on a relatively low permeability subgrade, the granular sub-base should be extended over the entire formation width

(Fig. 3) in order to drain the pavement structural section. Care should be exercised to ensure that its exposed ends do not get covered by the embankment soil. The trench type section should not be adopted in any case as it would lead to the entrapment of water in the pavement structure.

If the granular sub-base is of softer variety which may undergo crushing during rolling leading to denser gradation and low permeability, the top 100 to 150 mm thickness should be substituted by an open graded crushed stone layer to ensure proper drainage.

Drainage of the pavement structural section can be greatly improved by providing a high permeability drainage layer satisfying the following criteria :

$$\frac{D_{15} \text{ of drainage layer}}{D_{15} \text{ of subgrade}} \geq 5$$

To prevent entry of soil particles into the drainage layer

$$\frac{D_{15} \text{ of subgrade}}{D_{85} \text{ of subgrade}} \leq 5$$

$$\text{And} \quad \frac{D_{50} \text{ of drainage layer}}{D_{50} \text{ of subgrade}} \leq 5$$

Aggregates meeting the following criteria are regarded as very good drainage materials :

$$D_{85} < 4 D_{15}$$
$$D_2 > 2.5 \text{ mm}$$

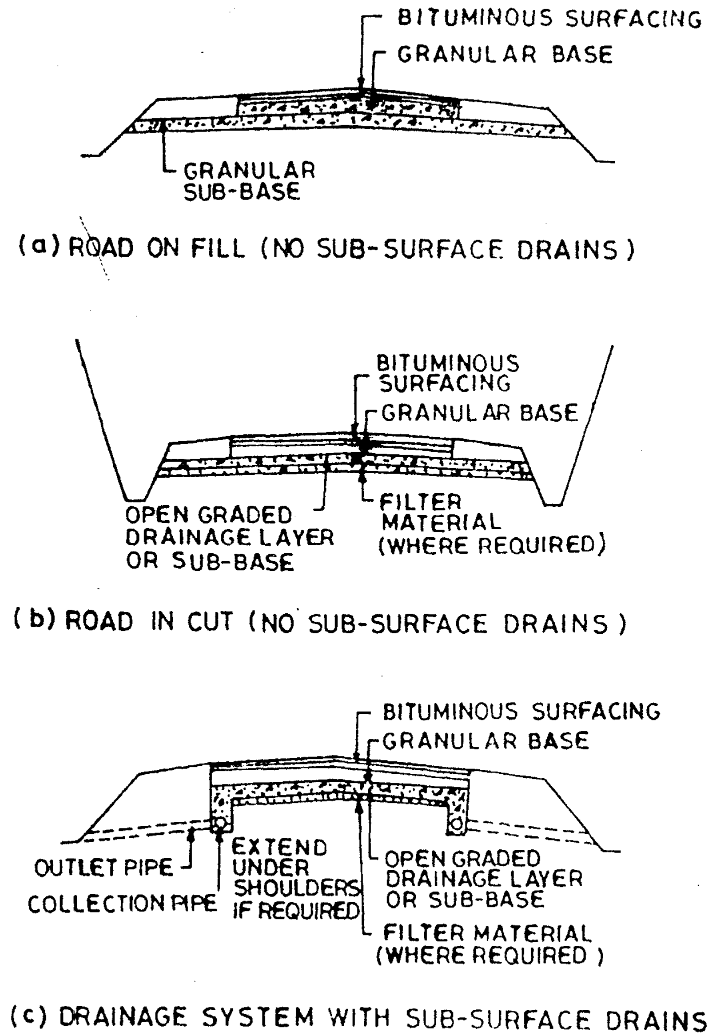


Fig. 3. Drainage of Pavements on Impermeable Subgrade

D_{85} means the size of sieve that allows 85 per cent by weight of the material to pass through it. Similar is the meaning of D_{50}, D_{15} and D_2.

The permeable sub-base when placed on soft erodible soils should be underlain by a layer of filter material to prevent the intrusion of soil fines into the drainage layer (Fig. 3). Non-woven geosynthetic can be provided to act as filter.

5.4. Where large inflows are to be taken care of, an adequately designed sub-surface drainage system consisting of an open graded drainage layer with collection and outlet pipes should be provided. The system should be designed on a rational basis using seepage principles to estimate the inflow quantities and the outflow conductivity of the drainage system. It should be ensured that the outflow capabilities of the system are atleast equal to the total inflow so that no free water accumulates in the pavement structural section. Sub-surface drains using geosynthetics can also be used. Sub-surface drains should be constructed to the requirements prescribed in Clause 309.3 of the MORT&H Specification for Road and Bridge Works.

5.5. Drainage of existing pavement of 'Trench Type' section on low permeability subgrades can be improved by providing a continuous drainage layer of 100-150 mm thickness under the shoulders at the subgrade level or by providing a combination of longitudinal and lateral drains, the latter spaced at 5 to 6 m intervals. The drains are cut through the shoulders upto the subgrade level and backfilled with coarse drainage material.

5.6. Very often, water enters the base, sub-base or the

subgrade at the junction of the verges and the bituminous surfacing. To counteract the harmful effects of this water, it is recommended that the shoulders should be well-shaped and if possible, constructed of impermeable material. With the same intent, it is suggested that as far as practicable, and in any case on major through roads, the base should be constructed 300-450 mm wider than the required bituminous surfacing so that the run-off water disperses harmlessly well clear off the main carriageway.

5.7. Shoulders should be accorded special attention during subsequent maintenance operation too. They should be dressed periodically so that they always conform to the requisite cross-fall and are not higher than the level of carriageway at any time.

6. DESIGN IN FROST AFFECTED AREAS

6.1. In areas susceptible to frost action, the design will have to be related to actual depth of penetration and severity of the frost. At the subgrade level, fine grained clayey and silty soils are more susceptible to ice formation, but freezing conditions could also develop within the pavement structure if water has a chance of ingress from above.

6.2. One remedy against frost attack is to increase the depth of construction to correspond to the depth of frost penetration, but this may not always be economically practicable. As a general rule, it would be inadvisable to provide total thickness less than 450 mm even when the CBR value of the subgrade warrants a smaller thickness. In addition, the materials used for building up the crust should be frost resistant.

6.3. Another precaution against frost attack is that water should not be allowed to collect at the subgrade level which may happen on account of infiltration through the pavement surface or verges or due to capillary rise from a high water table. Whereas capillary rise can be prevented by subsoil drainage measures and cut-offs, infiltrating water can be checked only by providing a suitable wearing surface.

7. WORKED EXAMPLES ILLUSTRATING THE DESIGN METHOD

Example - 1 : Design the pavement for construction of a new bypass with the following data :

DATA

(i)	Two-lane single carriageway	
(ii)	Initial traffic in the year of completion of construction	= 400 CV/day (sum of both directions)
(iii)	Traffic growth rate per annum	= 7.5 per cent
(iv)	Design life	= 15 years
(v)	Vehicle damage factor (based on axle load survey)	= 2.5 (standard axles per commercial vehicle)
(vi)	Design CBR of subgrade soil	= 4 per cent

DESIGN CALCULATIONS

(i) Distribution factor (para 3.3.5) = 0.75

(ii) Cumulative number of standard axles to be catered for in the design (Equation given in para 3.3.6.1)

$$N = \frac{365 \times [(1+0.075)^{15}-1]}{0.075} \times 400 \times 0.75 \times 2.5 = 7200000 = 7.2 \text{ msa}$$

(iii) Total pavement thickness for CBR 4% and Traffic 7.2 msa (from Fig. 1) = 660 mm

(iv) Pavement Composition interpolated from Plate 1, CBR 4%

(a) Bituminous surfacing = 25 mm SDBC + 70 mm DBM

(b) Road base = 250 mm WBM

(c) Sub-base = 315 mm granular material of CBR not less than 30 per cent

Example - 2 : It is proposed to widen an existing 2-lane National Highway section to 4-lane divided road. Design the pavement for new carriageway with the following data :

DATA

(i)	4-lane divided carriagway	
(ii)	Initial traffic in each directions in the year of completion of construction	= 5600 CV/day
(iii)	Design life	= 10 years/15 years
(iv)	Design CBR of subgrade soil	= 5 per cent
(v)	Traffic growth rate	= 8 per cent
(vi)	Vehicle damage factor (Found out from axle load survey on existing road)	= 4.5 (Standard axles per CV)

DESIGN CALCULATIONS

(i) Distribution factor (para 3.3.5) = 0.75

(ii) Vehicle damage factor = 4.5

(iii) Cumulative number of standard axles to be carried during

(a) Design life of 10 years

$$N = \frac{365 \times [(1+0.08)^{10}-1]}{0.08} \times 5600 \times 0.75 \times 4.5 = 99.2 \text{ msa or say } 100 \text{ msa}$$

(b) Design life of 15 years

$$N=\frac{365 \times [(1+0.08)^{15}-1]}{0.08} \times 5600 \times 0.75 \times 4.5 = 185.6 \text{ msa or say } 186 \text{ msa}$$

(iv) Pavement thickness and composition (from Fig. 2 and Plate 2) for CBR = 5 per cent and traffic = 100 msa/186 msa.

(a) For 10 years life

Total Pavement thickness for traffic 100 msa (from Fig. 2) CBR 5%	= 745 mm
Composition (from Plate 2, CBR 5%)	
Bituminous surfacing	= 50 mm BC + 150 mm DBM
Base	= 250 mm Wet Mix Macadam
Sub-base	= 300 mm Granular Sub-base of CBR not less than 30 per cent
Provide Total Pavement Thickness	= 750 mm

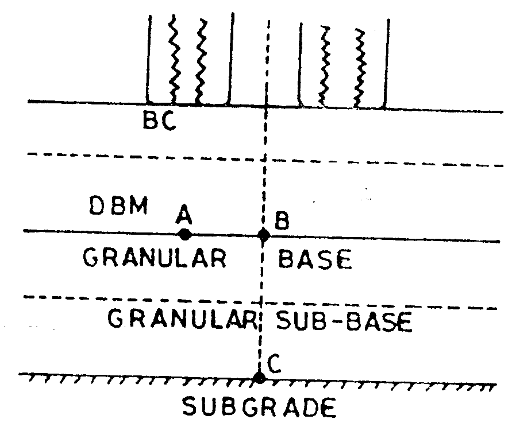
(b) For 15 years life (para 4.3.4) Accordingly provide pavement thickness and composition for 150 msa

Total pavement thickness (from Fig. 2)	= 760 mm
Composition, (from Plate 2, CBR 5%)	
Bituminous surfacing	= 50 mm BC + 170 mm DBM
Base	= 250 mm Wet Mix Macadam
Sub-base	= 300 mm GSB of CBR not less than 30 per cent
Provide Total Pavement Thickness	= 770 mm

Annexure-1

CRITICAL LOCATIONS, RELATIONSHIP BETWEEN NUMBER OF CUMULATIVE STANDARD AXLES, STRAIN VALUES AND ELASTIC MODULUS OF MATERIALS

Critical Locations in Pavement



A and B are the critical locations for tensile strains (ε_t). Maximum value of the strain is adopted for design.

C is the critical location for the vertical subgrade strain (ε_z) since the maximum value of the ε_z occurs mostly at C.

Fatigue Criteria :

Bituminous surfacings of pavements display flexural fatigue cracking if the tensile strain at the bottom of the bituminous layer is beyond certain limit. Based on large amount of field performance data of pavements of south, north, east and west zones in India collected under the Research Schemes R-6[4] and R-19[5] of Ministry of Surface Transport, Govt. of India, the relation between the fatigue life of the pavement and the tensile strain in the bottom of the bituminous layer was obtained[1] as

$$N_F = 2.21 * 10^{-4} \, [1/\varepsilon_t]^{3.89} \, [1/E]^{0.854}$$

Where,

N_F = Number of cumulative standard axles to produce 20 per cent cracked surface area

ε_t = Tensile strain at the bottom of BC layer (micro strain)

E = Elastic modulus of bituminous surfacing (MPa)

The above fatigue equation was calibrated at 35°C for Bituminous Concrete surfacing having 80/100 bitumen and the equation was generalised[1] by introducing the term containing the elastic modulus (E) of the bituminous layer so that pavement can be designed for temperatures from 20°C to 40°C using any grade of bitumen.

The values of the elastic moduli of Bituminous Concrete/ Dense Bituminous Macadam and Bituminous Macadam meeting the specifications of the MOST[2] are given below[1].

Elastic Modulus (MPa) Values of Bituminous Materials

Mix Type	Temperature °C				
	20	25	30	35	40
BC and DBM 80/100 bitumen	2300	1966	1455	975	797
BC and DBM 60/70 bitumen	3600	3126	2579	1695	1270
BC and DBM 30/40 bitumen (75 blow compaction and 4 per cent air void)	6000	4928	3809	2944	2276
BM 80/100 bitumen	-	-	-	500	-
BM 60/70 bitumen				700	

The Poisson's ratio of bituminous layer may be taken as 0.50 for pavement temperatures of 35°C and 40°C. For temperatures from 20°C to 30°C, a value of 0.35 may be adopted.

Fatigue equation at any pavement temperature from 20°C to 40°C can be evaluated by substituting the elastic modulus of the pavement temperature. Catalogue of designs has been worked out for temperature of 35°C.

Rutting Criteria :

As large number of data for rutting failure of pavements were obtained from the Research Scheme R-6[4] of the Ministry of Road Transport and Highways and other research investigations. Setting the allowable rut depth as 20 mm, the rutting equation was obtained[1] as

$$N_R = 4.1656 * 10^{-8} \, [1/\varepsilon_z]^{4.5337}$$

N_R = Number of cumulative standard axles to produce rutting of 20 mm

ε_z = Vertical subgrade strain (micro strain))

Modulus of Elasticity of Subgrade, Sub-base and Base Layers

Subgrade[8]

$E(\text{MPa}) = 10 * CBR$ for $CBR \le 5$ and

$= 176 * (CBR)^{0.64}$ for $CBR > 5$

Granular Sub-base and Base[7]

$E_2 = E_3 * 0.2 * h^{0.45}$

E_2 = Composite Elastic Modulus of granular Sub-base and Base (MPa)
E_3 = Elastic Modulus of Subgrade (MPa)
h = Thickness of granular layers (mm)

Poisson's ratio for both the granular layer as well as subgrade layer may be taken as 0.4.

Substitution of Dense Bituminous Macadam (DBM) :

Part of the DBM can be substituted for BM on the basis of equal flexural stiffness given as

$$E_1H_1^3/12(1-\mu_1^2) = E_2H_2^3/12(1-\mu_2^2)$$

where,

$E_1 H_1 \mu_1$ *and* $E_2 H_2 \mu_2$ are the parameters (Elastic Modulus, Thickness and Poisson's Ratio) of the DBM and BM respectively. Based on the above equation, following equivalent thicknesses may be used :

Example

180 mm DBM = 125 mm DBM + 75 mm BM

240 mm DBM = 185 mm DBM + 75 mm BM
75 mm of BM = 75* $(700/1695)^{1/3}$ = 55.85 mm of DBM

Annexure-2

EQUIVALENCE FACTORS AND DAMAGING POWER OF DIFFERENT AXLE LOADS

Gross Axle Weight Kg.	Load Equivalency Factors Single Axle	Tandem Axle
900	0.0002	0.0000
1810	0.002	0.0002
2720	0.009	0.001
3630	0.031	0.003
4540	0.08	0.006
5440	0.176	0.013
6350	0.35	0.024
7260	0.61	0.043
8160	1.00	0.070
9070	1.55	0.110
9980	2.30	0.166
10890	3.27	0.242
11790	4.48	0.342
12700	5.98	0.470
13610	7.8	0.633
14520	10.0	0.834
15420	12.5	1.08
16320	15.5	1.38
17230	19.0	1.73
18140	23.0	2.14
19051	27.7	2.61
19958	33.0	3.16
20865	39.3	3.79
21772	46.5	4.49
22680	55.0	5.28
23587	-	6.17
24494	-	7.15

Gross Axle Weight Kg.	Load Equivalency Factors Single Axle	Tandem Axle
25401	-	8.20
26308	-	9.4
27216	-	10.7
28123	-	12.1
29030	-	13.7
29937	-	15.4
30844	-	17.2
31752	-	19.2
32660	-	21.3
33566	-	23.6
34473	-	26.1
35380	-	28.8
36288	-	31.7

In case the class mark of the axle load survey does not match with the above axle loads, 4th power law may be used for converting axle loads into equivalent standard axle loads using the following formulae :

Single axle load

Equivalency factor = $(\text{axle load in kg}/8160)^4$

Tandem axle load

Equivalency factor = $(\text{axle load in kg}/14968)^4$

The above equations also give reasonably correct results for practical values of axle loads.

Annexure-3

PREPARATION OF LABORATORY TEST SPECIMENS

GENERAL

1. Wherever possible, the test specimens should be prepared by static compaction, but if not possible, dynamic method may be used as an alternative.

STATIC COMPACTION

2. The weight of wet soil at the required moisture content to give the intended density when occupying the standard test mould is calculated as follows :

Volume of mould = 2209 cc

Weight of dry soil = 2209 d gm

$$\text{Weight of wet soil} = \frac{100 + m}{100} \times 2209\ d \text{ gm}$$

where,

d = Required dry density in gm/cc

m = Required moisture content in per cent

3. The soil lumps are broken down and stones larger than 20 mm are removed. Sufficient quantity of the soil is mixed with water to give the required moisture content. The correct weight of wet soil is placed in the mould. After initial tamping with a steel rod, a filter paper is placed on top of the soil, followed by the 5 cm displacer disc, and the specimen compressed in the compression machine until the top of the displacer is flush with

the top of the collar. The load is held for about 30 seconds and then released. In some soil types where a certain amount of rebound occurs, it may be necessary to reapply load to force the displacer disc slightly below the top of the mould so that on rebound the right volume is obtained.

DYNAMIC COMPACTION

4. The soil is mixed with water to give the required moisture content, and then compacted into the mould in three layers using a standard soil rammer. After compaction, the soil is trimmed flush with the top of the mould with the help of a metal straight edge. The mould is weighed full and empty to enable determination of wet bulk density, and from it, knowing the moisture content, the dry density is calculated.

5. Further specimens, at the same moisture content, are then prepared to different dry densities by varying the number of blows applied to each layer of soil so that the amount of compaction that will fill the mould uniformly with calculated weight of wet soil (vide para 2 above) is known.

Annexure-4

SPECIAL POINTS RELATING TO DESIGN OF PAVEMENT ON EXPANSIVE SOILS

Potentially expansive soils, such as, black cotton soils are montmorillonite clays and are characterised by their extreme hardness and deep cracks when dry and with tendency for heaving during the process of wetting. Roadbeds made up of such soils when subjected to changes in moisture content due to seasonal wetting and drying or due to any other reason undergo volumetric changes leading to pavement distortion, cracking and general unevenness. In semi-arid climatic conditions, pronounced short wet and long dry conditions occur, which aggravate the problem of swelling and shrinkage. Due recognition of these problems at the design stage itself is required so that counter measures could be devised and incorporated in the pavement structure. A proper design incorporating the following measures may considerably minimise the problems associated with expansive soils.

SUBGRADE MOISTURE, DENSITY AND DESIGN CBR

The amount of volume change that occurs when an expansive soil road bed is exposed to additional moisture depends on the following :

(a) the dry density of the compacted soil
(b) the moisture content
(c) structure of soil and method of compaction

Expansive soils swell very little when compacted at low densities and high moisture but swell greatly when compacted at high densities and low moisture. Hence, where the probability of moisture variation in the subgrade is high, it is expedient to compact the soil slightly wet of the field optimum moisture content determined on the basis of a field trial. Experience shows that generally, it is not practicable to compact expansive soils at OMC determined by Laboratory Proctor Test. It is, therefore, necessary to study its field moisture density relationship through compacting the soil at different moisture contents and under the same number of roller passes. A minimum density corresponding to 95 per cent of the standard proctor density should be attained in the field and recommended moisture content should be 1-2 per cent wet of optimum moisture content.

1. Design CBR

The pavement thickness should be based on a 4-day soaked CBR value of the soil, remoulded at placement density and moisture content ascertained from the field compaction curve.

2. Buffer Layer

There is a definite gain in placing the pavement on a non-expansive cohesive soil cushion of 0.6-1.0 m thickness. It prevents ingress of water in the underlying expansive soil layer, counteracts swelling and secondly even if the underlying expansive soil heaves, the movement will be more uniform and consequently more tolerable. However, where provision of non-expansive buffer layer is not economically feasible, a blanket course of suitable material and thickness as discussed in para 3 below must be provided.

3. Blanket Course

A blanket course of atleast 225 mm thickness and composed of coarse/medium sand or non-plastic moorum having PI less than five should be provided on the expansive soil subgrade as a sub-base to serve as an effective intrusion barrier. The blanket course should extend over the entire formation width.

Alternatively, lime-stabilised black cotton sub-base extending over the entire formation width may be provided together with measures for efficient drainage of the pavement section.

4. Drainage

Improvement of drainage can significantly reduce the magnitude of seasonal heaves. Special attention should, therefore, be given to provision of good drainage measures as also discussed under Section 5 (Drainage Measures). The desirable requirements are :

(a) Provision must be made for the lateral drainage of the pavement structural section. The granular sub-base/base should accordingly be extended across the shoulders, refer to para 5.3 of section 5 (Drainage Measures).

(b) Normal camber of 1:40 for the black top surface and a cross slope of 1:20 for the berms should be provided to shed-off surface run-off quickly.

(c) No standing water should be allowed on either side of the road embankment

(d) A minimum height of 1 m between the subgrade level and the highest water level should be ensured.

5. Bituminous Surfacing

Desirably 40 mm thick bituminous surfacing should be provided to prevent ingress of water through surface.

6. Shoulders

Shoulders should be made up of impervious material so as not to allow water to permeate into the body of the pavement. Lime stabilised black cotton soil shoulder of 150-200 mm thickness may serve the purpose economically.

Annexure-5

RECOMMENDED TYPE AND THICKNESS OF BITUMINOUS WEARING COURSES FOR FLEXIBLE PAVEMENTS UNDER DIFFERENT SITUATIONS

Sl. No.	Type of Base/ Binder Course	Type of Bituminous Wearing Course	Annual Rainfall Low (L) less than 1500 mm; Medium (M) 1500-3000 mm; High (H) more than 3000 mm	Design Traffic (msa)
1.	Water Bound Macadam, Wet Mix Macadam, Crusher-run-Macadam, Built-up Spray Grout	(i) 20 mm Premix Carpet (PC) with sand seal coat	L and M	< 10.0
		(ii) 20 mm PC with liquid seal coat	L,M and H	<10.0
		(iii) Mix Seal Surfacing (MSS) (20 mm) Type 'A' or 'B'	L,M and H	< 10.0
2.	Bituminous Macadam base/binder course	(i) Semi-Dense Bituminous Concrete (25 mm)	L,M and H	<10.0
		(ii) 20 mm PC with liquid seal coat		
		(iii) MSS (20 mm) Type 'A' or 'B'		
3.	Dense Bituminous Macadam	Bituminous Concrete		
		(i) 25 mm	L,M and H	>5<10
		(ii) 40 mm	L,M and H	>10
		(iii) 50 mm	L,M and H	>100

In applying the above recommendations, the following points should be kept in view :

(i) In case where a pavement is decided to be developed in stages, the surfacing should correspond to that for the design stage.

(ii) As far as possible, wearing course amenable to laying with paver-finisher should be adopted over paver-finished base binder course.

(iii) Expensive surfacings, like, Bituminous Concrete (BC) should not be provided directly over manually laid granular bases.

Annexure-6

CRITERIA FOR THE SELECTION OF GRADE OF BITUMEN FOR BITUMINOUS COURSES

Climate	Traffic (CVD)	Bituminous Course	Grade of Bitumen to be used
Hot	Any	BM, BPM, BUSG	60/70
Moderate/Cold	Any	BM, BPM, BUSG	80/100
Any	Heavy Loads, Expressways, Urban Roads	DBM, SDBC, BC	60/70
Hot/Moderate	Any	Premix Carpet	50/60 or 60/70
Cold	Any	Premix Carpet	80/100
Hot/Moderate	Any	Mastic Asphalt	15±5
Cold	Any	Mastic Asphalt	30/40

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IRC : 39-1986

STANDARDS FOR ROAD-RAIL LEVEL CROSSINGS

(First Revision)

THE INDIAN ROADS CONGRESS

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STANDARDS FOR ROAD-RAIL LEVEL CROSSINGS

CONTENTS

STANDARDS FOR ROAD-RAIL LEVEL CROSSINGS

1. INTRODUCTION

1.1. Road-rail level crossings, however, adequately designed and constructed, are accident prone. However, where it is not possible from engineering and economic considerations to provide road over/under bridges, and level crossings have to be provided, the standards given here under should be followed in the interest of maximum safety.

1.2. These standards are intended primarily to be applied to new constructions or where an existing crossing is being reconstructed. Existing level crossings need not be altered merely to suit these standards.

1.3. The draft of this standard had been approved by the Specifications and Standards Committee at its meeting held at New Delhi in September 1961. Thereafter, as suggested by the Committee, it was forwarded to the Roads Wing of the Ministry of Shipping and Transport for finalisation in consultation with the Ministry of Railways. The Railways gave their concurrence to the standard in September 1970, after effecting slight changes in the original text. The draft standard was subsequently approved for publication by the Executive Committee and the Council at their meetings held respectively in November and December 1970. Provisions of this Standard have been separately circulated by the Railway Board among the different Zonal Railways.

1.4. On the request of the Indian Roads Congress, the first revision has been made by the Ministry of Transport, Department of Surface Transport (Roads Wing) after obtaining the comments from the Ministry of Transport, Department of Railways. Besides the minor editorial changes a fresh clause 21 "Safety Measures to Minimise Accidents" has been added in the revision.

2. LOCATIONS

As far as possible, road-rail level crossings should not be located near railway stations and marshalling yards. If this is unavoidable, they should be located beyond shunting limits.

3. CLASSIFICATION OF LEVEL CROSSINGS

3.1. Level crossings shall be classified as below :

Special A class B class C class	For vehicular traffic

D class For cattle crossings and footpaths

3.2. The classification of a rail-road level crossing shall be settled mutually by the Railway and Road Authorities keeping in view the class of the road, visibility conditions, the volume of road traffic and the number of trains passing over the level crossing.

4. CATEGORIZATION OF ROADS

For the purpose of this standard, the roads shall be categorized as under :

(*i*) **Class I Roads**

(*a*) National Highways ;

(*b*) State Highways ;

(*c*) Important roads within municipal towns ; and

(*d*) Roads in and around towns where road and rail traffic is heavy.

(*ii*) **Class II Roads**

(*a*) Major and Other District roads ;

(*b*) Unimportant roads within municipal towns ;

(*c*) Roads within non-municipal towns including those within shunting limits of its railway stations ; and

(*d*) Other surfaced roads.

(*iii*) **Class III Roads**

(*a*) Earth roads ; and

(*b*) Cart tracks.

(*iv*) **Class IV Roads**

Cattle crossings and footpaths.

5. WIDTH OF CARRIAGEWAY

(*i*) **Between gates**

Between gates the width of carriageway shall be the same as that of the gates (see Clause 7).

(*ii*) **Outside gates**

The minimum width of carriageway immediately outside the gates (but tapering off to the existing carriageway width within a distance of 30 m from the gate) shall be as below :

(*a*) **Class I Roads**

7 m or the width of the existing carriageway, whichever is greater.

(*b*) **Class II Roads**

5.5 m or the width of the existing carriageway, whichever is greater.

(*c*) **Class III Roads**

3.75 m or the width of the existing carriageway, whichever is greater.

(*d*) **Class IV Roads**

Suitable width, subject to 2 m being the minimum.

6. TYPE OF THE PAVEMENTS

(*i*) **Between gates**

The surface shall not be of a lower standard than the surface outside the railway boundary. In case the surface outside the gates be of cement-concrete, black-topped surface may be provided.

(ii) **Outside gates**

The surface should not be of a lower specification than that of the existing road. However, in the case of Class I and Class II roads, it will be desirable to have a black-topped surface for a distance of at least 30 m beyond each gate.

7. MINIMUM WIDTH OF GATES AT RIGHT ANGLES TO THE CENTRE LINE OF THE ROAD

(i) **For Class I Roads**

9 m or equal to the width of the carriageway, immediately outside gates plus 2.5 m whichever is more.

(ii) **For Class II Roads**

7.5 m or equal to the width of the carriageway immediately outside gates plus 2 m whichever is more.

(iii) **For Class III Roads**

5 m or equal to the width of the carriageway immediately outside the gates plus 1.25 m whichever is more.

(iv) **For Class IV Roads**

Suitable width, subject to 2 m being the minimum.

8. MINIMUM LENGTH OF GUARD-RAILS

This should be 2 m more than the width of the gates on square crossings, and proportionately longer on skew crossings.

9. POSITION OF GATES WITH RESPECT TO THE CARRIAGEWAY

9.1. The gates may be swing gates, lifting gates or movable barriers of approved design.

9.2. The gates should be at right angles to the centre line of the road.

9.3. On level crossings across Class IV roads, stakes shall be fixed between the gate posts to prevent passage of road vehicles.

10. MINIMUM DISTANCE OF GATES FROM THE CENTRE LINE OF THE NEAREST RAIL TRACK

This should be 3 m on broad gauge lines and 2.5 m on metre gauge and narrow gauge lines.

11. WIDTH OF ROAD FORMATION OUTSIDE THE GATES

The width of the road formation for a distance of 30 m beyond the gate should be as follows :

(i) **Class I and Class II Roads**

Width of carriageway immediately outside the gates (see Clause 5) plus 5 m.

(ii) **Class III Roads**

Width of the carriageway immediately outside the gates (see Clause 5) plus 2.5 m.

(iii) **Class IV Roads**

Suitable width subject to 3 m being the minimum.

12. LEVEL LENGTHS AND GRADIENTS

(i) **Between gates**

Level for all classes.

(ii) **Outside gates**

(a) **Class I Roads**

Same level as between the gates upto 15 m beyond gates and not steeper than 1 in 40 beyond.

(b) **Class II Roads**

Same level as between the gates upto 8 m beyond gates and not steeper than 1 in 30 beyond.

(c) **Class III Roads**

Same level as between the gates upto 8m beyond gates and not steeper than 1 in 20 beyond.

(*d*) **Class IV Roads :** Not steeper than 1 in 15.

Note : Shock-free vertical curves as per Indian Roads Congress Standards should be provided at all gradient changes. The level distances mentioned above are exclusive of the lengths required for the provision of vertical curves.

13. ANGLE OF CROSSING BETWEEN THE CENTRE LINES OF THE RAILWAY TRACK AND THE ROAD

The angle of crossing between the centre line of the road and that of the railway track should ordinarily not be lower than 45 degrees* in the case of Class I, Class II and Class III roads. For Class IV roads, the angle of crossing should be 90 degrees.

14. MINIMUM RADIUS OF THE CENTRE LINE OF THE ROAD ON CURVED APPROACHES

14.1. Minimum radius of the curve shall depend upon the design speed, coefficient of friction between the tyres and road surface and maximum allowable value of superelevation. The minimum radii for different design speeds for good surfaced roads may be provided as indictated in the table below :

Speed km/h	Radius of horizontal curve (metres)		
	Plain and rolling terrain	Hilly	
		Not affected by snow	Snow bound
20	—	14	15
25	—	20	23
30	—	30	33
35	45	40	45
40	60	50	60
50	90	80	90
60	130	—	—
65	155	—	—
80	230	—	—
100	360	—	—

* **An angle of crossing lower than 45 degrees can also be provided but only after special permission from the Railway Board which may be granted in exceptional cases.**

14.2. **In difficult terrain where it is not possible to adopt the above standard, the radius may be reduced with the concurrence of the Road Authority.**

14.3. For other categories of roads, the best possible radius having regard to safety of the road traffic, should be adopted.

15. SIGHT DISTANCES

15.1. The roads in the vicinity of the level crossings shall be provided with sight distances depending upon the design speed as per table No. 11 of IRC : 73-1980 reproduced below :

STOPPING SIGHT DISTANCE FOR VARIOUS SPEEDS

Speed	Perception and brake reaction		Braking		Safe stopping sight distance (metres)	
V (km/h)	Time, t (Sec.)	Distance (metres) $d_1 = 0.278$ Vt	Coefficient of longitudinal friction (f)	Distance (metres) $d_2 = \frac{V^2}{254 f}$	Calculated values $d_1 + d_2$	Rounded off values for design
20	2.5	14	0.40	4	18	20
25	2.5	18	0.40	6	24	25
30	2.5	21	0.40	9	30	30
40	2.5	28	0.38	17	45	45
50	2.5	35	0.37	27	62	60
60	2.5	42	0.36	39	81	80
65	2.5	45	0.36	46	91	90
80	2.5	56	0.35	72	118	120
100	2.5	70	0.35	112	182	180

15.2. To further improve visibility, gate lodges should be so sited that a clear and unobstructed view is obtained by the road traffic of all approaching trains. While doing so, care should be taken to make allowance for all possible future extensions, *e.g.*, additions to the railway track(s) or widening of the road.

15.3. On unmanned level crossings, efforts should be made to keep the sight triangles demarcated in the four corners on the basis of speeds of trains and the road vehicles, clear of any obstruction to sight.

16. MINIMUM STRAIGHT LENGTH OF ROAD OUTSIDE THE GATES

This shall normally be 30 m, 22.5 m and 15 m for level crossings of Class I, Class II and Class III roads respectively. The straight length may, however, be reduced depending on sight conditions if difficult to attain. The reduction should, however, not go below the minimum straight lengths of 15 m, 9 m and 4.5 m for the three classes of roads respectively.

17. WARNING TO ROAD TRAFFIC OF THE PROXIMITY OF LEVEL CROSSING

17.1. Unguarded Railway Crossing

The sign should be used on the approaches of level crossings where there are no gates or other barriers. A pair of signs shall be used for the purpose : (i) an advance warning sign located at 200 metre away from the crossing, and (ii) a second sign to be erected near the crossing. The distance of the second sign from the crossing may be 50-100 metre in plain and rolling terrain and 30-60 metre in hilly terrain.

17.2. Guarded Railway Crossing

The sign should be used to warn traffic on the approaches of guarded railway crossings. A pair of signs shall be used for the purpose : (i) an advance warning sign located at 200 metre away from the crossing, and (ii) a second sign to be erected near the crossing. The distance of the second sign from the crossing may be 50-100 metre in plain and rolling terrain and 30-60 metre in hilly terrain.

17.3. Gates should be painted white, with a red disc not less than 60 cm in diameter in the centre. The gate posts also must be painted white. Where gates or chains are not provided, posts must still be provided at the position prescribed for gate posts and these should be painted white.

18. MINIMUM DISTANCE OF GATE LODGE

18.1. The minimum distance of gate lodge shall be as given below :

	Class I Roads	*Class II Roads*	*Class III Roads*	*Class IV Roads*
(*a*) From the centre line of the nearest rail track	6 m	6 m	6 m	6 m
(*b*) From the edge of the carriageway	6 m	6 m	6 m	6 m

18.2. The recommendation in Clause 15 regarding sight distances should also be kept in view.

19. PROVISION OF WICKET GATES FOR FOOT-PASSENGERS

19.1. In the case of level crossings on Class I and Class II roads, wicket gates for pedestrians shall be provided except where there are foot overbridges.

19.2. In the case of level crossings on Class III and Class IV roads, wicket gates need not be provided.

19.3. Wicket gates should be of such a design that cattle cannot easily and readily pass through them.

20. PROVISION OF LIGHT ON GATES AT NIGHT

(*i*) **Light as observed by road users**

(*a*) **Class I and Class II Roads**

Red when either gate is closed to the road. White, when the gates are opened to the road.

(*b*) **Class III Roads**

Same as above, but reflectors may be used as an alternative to lamps.

(*ii*) ***Light as observed by drivers of approaching trains***

(*a*) **Class I Roads :** Red, when gates are closed across the railway track.

(*b*) **Other cases :** Nil.

21. SAFETY MEASURES TO MINIMISE ACCIDENTS

21.1. Latest IRC road signs indicating whether the railway crossing is manned or unmanned shall be installed on either end of the crossing at the prescribed distance as per IRC : 67

21.2. Speed limit road signs for the imposed Speed Signs of limit on speed of approaching traffic shall be installed on either end of the crossing at the prescribed distance.

21.3. Rumble strips on both sides of the Railway crossing shall be provided as per following specifications. A common application of rumble strips is the placement of intermittent, raised bituminous overlays across the roadway. Raised sections can be 15-25 mm high, 200-300 mm wide, and spaced about one metre centre to centre. A series of such strips, roughly 15-20 at one location shall be provided. The raised sections shall consist of premix carpet/semi-dense carpet/asphaltic concrete.

21.4. Speed breakers shall not be permitted.

21.5. Flashing signals shall be installed on both sides of the crossing after assessing their requirement for each case.

IRC : 49-1973

RECOMMENDED PRACTICE FOR THE PULVERIZATION OF BLACK COTTON SOILS FOR LIME STABILISATION

THE INDIAN ROADS CONGRESS

IRC : 49-1973

RECOMMENDED PRACTICE
FOR
THE PULVERIZATION
OF
BLACK COTTON SOILS
FOR
LIME STABILISATION

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RECOMMENDED PRACTICE FOR THE PULVERIZATION OF BLACK COTTON SOILS FOR LIME STABILISATION

1. INTRODUCTION

1.1. Stabilising black cotton soils with lime has been found to be an effective method to improve their engineering properties. This technique is finding increasing application in black cotton soil areas for the construction of sub-bases of road pavements.

1.2. Black cotton soil is characterised by high swelling when wet and excessive shrinkage when dry. This poses problems as regards subsequent performance of the road. Moreover, the softened subgrade has a tendency to work up into the upper layers of the pavement, especially when the sub-base consists of stone soling with lot of voids. Gradual intrusion of soil invariably leads to failure of the road. Such difficulties could be avoided by providing a sub-base layer of granular material or of lime-stabilised soil which would serve as a blanket course between the subgrade soil and the upper layers of the pavement.

1.3. A pre-requisite to successful lime stabilisation is that at the time of addition of stabiliser, the soil should be in a reasonably pulverised state. Whereas light textured soils are generally friable, and therefore easy to pulverise, this is not so in the case of expansive soils like black cotton soils, which are soft and sticky when wet but very hard when dry. Besides affecting the uniform mixing of lime, the degree of pulverisation influences the amount of lime necessary to achieve the desired gain in strength.

1.4. Realising the importance of pulverisation in the case of black cotton soils, this Standard was initially prepared by the Soil Engineering Committee (personnel given below) for general guidance in this respect. It was processed and approved by the Specifications and Standards Committee in their meeting held on the 29th and 30th September 1972. Later, this was finally approved by the Executive Committee in their meeting held on the 11th March 1973 and by the Council in their 81st meeting held at Cochin on the 26th April 1973.

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2. SCOPE

2.1. The Recommended Practice applies both to construction of new roads and widening of existing roads. It covers pulverisation by manual means as well as with mechanical equipment.

2.2. Pulverisation may be carried out either at borrow source, or at embankment site after deposition of the soil, depending on feasibility and convenience.

3. DEGREE OF PULVERISATION

3.1. Degree of pulverisation has a profound effect on the properties of lime-mixed black cotton soil. This would be clear from the illustrative example given in *Annexure 1*.

3.2. Pending further research, pulverisation to the following degree should be aimed at in the field :

Sieve designation (IS : 460)	*Per cent by weight of soil passing the sieve after pulverisation*
25 mm	100
4.75 mm	50

3.3. Method of sieving to determine the degree of pulverisation should be as given in *Annexure 2*.

4. METHODS OF PULVERISATION

4.1. Using Manual Labour

4.1.1. This method can be best used in summer months when the soil is dry. Top vegetation is removed and dry soil crust loosened to a depth of about 20 cm with the help of crow-bars. Bigger sized clods can be further broken down with pick-axes or rammers. Application of a country plough driven by a bullock can also be tried, which proves sometimes effective.

4.1.2. Alternatively, the soil dug from the field can be stacked and water sprinkled on it. After allowing some time for the soil to dry, water should be sprinkled again. This procedure is repeated sufficiently so as to lead to a gradual disintegration of the soil clods at the surface thereby reducing their size. But the process is slow and workable only in areas where water is within economic reach.

4.2. Using Power Rollers

4.2.1. This method can be used only when the soil is dry. The soil is dug from the field and clods broken with pick-axes so as to reduce them to a maximum size of 5 cm. Soil clods are spread over the subgrade and a smooth wheeled 8 tonne power roller passed over them a number of times, accompanied by frequent raking of the crushed material. About 8 passes of the roller combined with raking should normally be able to achieve the required degree of pulverisation.

4.3. Using Heavy Agricultural Machinery

4.3.1. The following mix-in-place plant can be effectively utilised for pulverisation :

(i) Tractors (on chain tracks) of about 110 H.P.;

(ii) Mould Board Ploughs (consisting of four furrows which can plough upto about 40 cm depth);

(iii) Disc Harrows (consisting of five discs of about 70 cm dia, with a working width of 3 metres);

(iv) Off-set Harrows (consisting of 18 discs of about 50 cm dia, arranged in two rows with a working width of 3 metres).

4.3.2. The natural ground may be ploughed using a Mould Board Plough attached to a heavy tractor. This should loosen the soil to a depth of about 40 cm. The Mould Board Plough is then replaced by Disc Harrow which is passed over the ploughed soil about four times in order to break down the clods further. The soil is then processed about five times with an Off-set Harrow attached to a tractor. Any big clods still left could be picked out by manual labour.

4.3.3. A moisture content in the range of 15-30 per cent is most appropriate for the use of heavy agricultural machinery. At higher moisture contents the soil tends to be sticky and the implements get bogged down. At lower moisture contents, the soil clods are too dry for effective pulverisation. It follows from this that it would be most advantageous to pulverise soil after the rainy season when appropriate moisture conditions are likely to prevail in the field.

4.4. Using Light Agricultural Machinery

4.4.1. The light machinery could comprise the following :

(i) Tractors of about 50 H.P.

(ii) Mould Board Ploughs (consisting of three furrows)

(iii) Disc Harrows (consisting of 20 saucer shaped discs having a dia of about 25 cm); and

(iv) Off-set Harrows (consisting of 10 discs arranged in 2 rows with a dia of about 50 cm).

4.4.2. The operations are to be carried out in the same way as with the use of heavy machinery. Effective pulverisation is possible in the moisture content range of 18 to 22 per cent. About 6 passes of Disc Harrow and 10 passes of Off-set Harrow would be sufficient to yield an acceptable degree of pulverisation.

4.4.3. The cost with this method works out to be generally more than with heavy machinery. The higher cost with light machinery is due to the fact that the Mould Board Plough fitted to a light tractor cannot plough to a depth of more than 20 cm.

4.4.4. Even though light machinery may prove less economical, its use has the advantage that as compared to heavy machinery, the equipment can be procured more readily either by hiring out from agriculturists or buying it outright from the market.

4.5. Other Variations

4.5.1. As a modification of the methods described in the preceding paragraphs, the soil may be excavated and stacked on the berms, allowed to dry to a moisture content of about 15-20 per cent, relaid on the subgrade in a layer of about 25 cm thickness and its clods broken by dragging a heavy wooden log of 30 cm × 30 cm × 3 m size pulled by a tractor. The soil is then turned upside down with the help of an off-set plough so that no clods remain at the bottom of the layer. Thereafter lime is spread on the pulverised soil layer in required quantities and mixed with the soil by means of an off-set plough resulting in further pulverisation of the soil.

5. PRIOR ADDITION OF LIME TO AID PULVERISATION

5.1. Adding a small quantity of lime to soil, which is already partially broken down to small clods, and allowing it to work in for 3-4 days before final pulverisation, is known to help in achieving a better degree of pulverisation. The moisture content should be sufficient to assist the migration of lime. Detrimental effect of this procedure on soil-lime reaction, due to exposure to air containing carbon dioxide, is very slight inasmuch as the time of exposure is small. Presumably lime reacts with carbon dioxide to form calcium carbonate, with consequent loss in lime content and strength. It is likely that if this procedure is resorted to, a somewhat greater concentration of lime might be required than if the entire quantity of lime were to be added in a single stage.

5.2. Tentatively, it is recommended that about 50 per cent of the required concentration of lime may be added prior to pulverisation, and the balance after pulverisation. The exact amount of lime to be added initially could be finalised on the basis of tests for the soaked CBR value attainable by this procedure.

5.3. It is suggested that in the field, the clay clods should all be broken down to 5 cm size or lower before lime is added in the pre-pulverisation stage.

Annexure 1

ILLUSTRATIVE EXAMPLE OF THE EFFECT OF DEGREE OF PULVERISATION AND PERCENTAGE ADDITION OF LIME ON PROPERTIES OF BLACK COTTON SOIL

1. Table below shows the effect of degree of pulverisation on soaked CBR of a typical black cotton soil when treated with 3 per cent hydrated commercial lime (of 40 per cent purity) and compacted to a density of 1.5 gm/cc.

TABLE

Sample No.	Per cent passing sieve no.			Soaked CBR	Moisture absorption (per cent)
	IS sieve 25 mm	IS sieve 4.75 mm	IS sieve 2 mm		
1	100	—	—	2.4	27.5
2	100	50	15	14.2	26.3
3	100	100	30	14.3	26.9
4	100	100	100	14.7	25.3

Notes : (a) The values of CBR given above should be regarded as illustrative only.

(b) Curing period for the samples was 10 days.

(c) The soil had LL=75, PI=35 and per cent free swell of 100.5 per cent.

2. It may be inferred from the results above that if soil is so pulverised that at least 50 per cent of it is finer than 4.75 mm sieve, then the strength attained is practically the same regardless of the fraction passing 2 mm sieve. This may be attributed to the fact that in the process of mixing soil with lime, and during its subsequent compaction, the degree of pulverisation increases.

Annexure 2

METHOD OF SIEVING FOR COHESIVE SOILS TO DETERMINE THE DEGREE OF PULVERISATION

1. A sample of pulverised soil, approximately 1 kg in weight, should be taken and weighed (W_1).

2. It should be spread on the sieve and shaken gently, care being taken to break the lumps of soil as little as possible. Weight of soil retained on the sieve should be recorded (W_2). Lumps of finer soils in the retained material should be broken until all the individual particles finer than the aperture size of the sieve are separated.

3. The soil should be again replaced on the sieve and shaken until sieving is complete. The retained material should be weighed (W_3).

4. Per cent weight of soil passing the sieve can then be calculated from the expression :

$$\frac{(W_1 - W_2)\,100}{(W_1 - W_3)}$$

GUIDELINES FOR THE USE OF SOIL-LIME MIXES IN ROAD CONSTRUCTION

(First Revision)

THE INDIAN ROADS CONGRESS
1992

MEMBERS OF THE HIGHWAYS SPECIFICATIONS AND STANDARDS COMMITTEE

GUIDELINES FOR THE USE OF SOIL-LIME MIXES IN ROAD CONSTRUCTION

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GUIDELINES FOR THE USE OF SOIL-LIME MIXES IN ROAD CONSTRUCTION

1. INTRODUCTION

1.1. Addition of lime to soils is known to improve their strength and reduce plasticity which in turn controls volume change. Because of these advantages, this technique has been used extensively for improving quality of sub-grade and producing a high strength material for sub-bases.

1.2. Lime reacts with most of the medium, moderately fine, and fine grained soils to decrease plasticity, increase workability, reduce swell, and increase strength. In general terms, the soils that are most reactive to lime include clayey gravels, silty clays and clays. Soils classified according to the Indian Standard system as CH, CL, MH, ML, CL-ML, SC, SM, GC and GM should be considered as potentially capable of being stabilised with lime.

1.3. In the case of soil-lime construction, best results are obtained when mixing is homogeneous throughout. This is best achieved by mechanical methods of construction. Use of mechanised method is, therefore, very desirable. If for any unavoidable reason, manual method is to be adopted, care must be taken to ensure proper pulverisation of soil and uniform mixing of lime.

1.4. The guidelines were originally published as IRC: 51-1973 "Recommended Design Criteria for the Use of Soil-Lime Mixes in Road Construction". The revised document in the present form was prepared by the Geotechnical Engineering Committee (personnel given below), and considered by the Highways Specifications and Standards Committee in its meeting held on the 16th April, 1990. The Committee requested a Sub-Group consisting of Shri R.P. Sikka, Dr. P.J. Rao, Shri T.K. Natarajan and Dr. L.R. Kadiyali to modify the draft.

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1.5. The modified draft was subsequently considered and approved by the Highways Specifications & Standards Committee, the Executive Committee and the Council of the Indian Roads Congress in their meetings held on 30th October 1990, 18th November 1990 and 8th December 1990 respectively.

2. SCOPE

The guidelines relate to soil-lime mixes intended for use as improved subgrade or as a sub-base in highway pavements. It is presumed that the work will be carried out in accordance with appropriate construction specifications and adequate site supervision would be exercised as regards the quality of constituent materials and process of construction.

3. MATERIALS

3.1. Properties of lime-soil mixes are dependent on many variables viz. soil type, lime type,lime percentage, fineness of lime and lime purity in addition to the conditions of temperature and moisture. Some of the variables related to materials are discussed below.

3.2. Soil

3.2.1. **Samples:** Before taking recourse to lime stabilisation, it is desirable to examine the suitability of soil for this treatment. This is done by taking samples of the soil at appropriate locations. Disturbed samples are normally adequate for this purpose. The normal practice followed in the field is to take at least three samples in a stretch of one kilometre along the alignment of the road if the same type of soil is found throughout. In case the soil type changes earlier, at least one sample is taken from each new stretch of soil. Also, when the soil type changes with depth within a borrow pit, at least one sample should be collected from each strata.

3.2.2. **Suitability:** Clayey soils including heavy clays, moorums and other soils met within the alluvial plains can be effectively treated with lime. Since the gain in strength is based on pozollanic reactions between lime and clay minerals, it is essential that any soil selected for stabilisation should have enough clay minerals. For effective stabilisation, a soil must have a fraction passing 425 micron mesh not less than 15 per cent and its PI should be at least 10 per cent. For effective stabilisation, it is desirable that the per cent retained on 425 micron mesh should be well graded with uniformity coefficient not less than 5. Besides, clay minerals should belong to illitic, montmorillonitic or kaolinitic group. Generally clayey soils including black cotton soils, moorum and other alluvial types of soils, can be economically stabilised with lime. Organic matter in the soil selected for lime stabilisation should not be more than 2.0 per cent and sulphate content should not exceed 0.2 per cent. A pH value of 10 or 11 is desired for the pozollanic reaction to take place between clay minerals and lime for the formation of cementitious compounds. Where organic matter and soluble carbonate/sulphate contents in the soil are in excess of 2.0 and 0.2 per cent respectively, special studies would be necessary to determine whether lime stabilisation would prove to be practicable and economical.

3.3. Lime

3.3.1. **Type of lime:** Lime for lime-soil stabilization work shall be calcitic dry lime, commercially available, slaked at site, or pre-slaked lime delivered at site in suitable packing. Use of dolomitic lime is not considered suitable.

3.3.2. **Purity:** Purity of lime affects the strength of lime soil stabilisation. The effectiveness of lime in its reaction with clay minerals is dependent to a good extent on its chemical composition i.e. the amount of calcium oxide present in the lime. The purity of lime is expressed as the percentage of calcium oxide present in the lime. It is generally recommended that lime used for soil stabilisation should have a purity of 50 per cent. The addition of lime should be correspondingly increased whenever the field tests show a lesser purity.

Calcium oxide content in lime should be determined as specified in IS: 1514-1959 "Indian Standard Method of Sampling and Test for Quick Lime and Hydrated Lime" or IS: 712-1984 "Indian Standard Specification for Building Limes."

3.3.3. **Fineness:** For effective stabilisation with lime, uniform mixing is a pre-requisite and the degree of mixing depends on the fineness of lime. With fine lime,there will be quick and effective reaction with clay minerals to form cementitious compounds. Lime for stabilization shall conform to the fineness requirement of Class C hydrated lime as specified in IS: 1514 and IS: 712, which is as under:

Sieve size (in micron)	% Passing
850	100
300	95 (Minimum)
212	99 (Minimum)

4. DESIGN CONSIDERATIONS

4.1. Evaluation of Lime Requirement

4.1.1. Before construction of lime stabilised road is undertaken, survey

of soil deposits existing on the aligned route is done (para 3.2.1.). A small addition of lime to clayey soil results in considerable increase in the workability of the soil, possibly due to base exchange or flocculation phenomenon or a combination of the two. With such small dosages of lime, no appreciable gain in strength of the mixture results. As the dose of lime is increased, the strength of the mixture increases. This suggests that the lime added initially in small dosages is utilised in satisfying the affinity of the soil for lime. The quantity of lime needed for satisfying this affinity is termed as lime fixation point or lime retention point. Lime needed for this purpose is usually 1 to 3 per cent.

4.1.2. The strength of lime soil mixture is dependent to a great extent on the quantity of lime added above the lime fixation point. The tangible effect of lime soil stabilisation in increasing the strength of the mixture begins to be felt as the lime content is increased above lime fixation point. This is due to pozzolanic reactions resulting in the production of cementitious compounds. It is generally found that beyond a certain percentage of lime, the increase in strength ceases and in fact a lowering in strength may result due to the presence of unreacted free lime indicating that there exists an optimum lime content for maximum strength gain. Some tentative values of optimum lime content for soils of different mineralogical composition are indicated below:

Kaolinitic soil	- 4 per cent
Illitic soil	- 8 per cent
Montmorillonitic soil	- 10 per cent

The optimum lime content as explained above may be determined by one of the following methods:

(i) **pH Method**

The optimum lime requirement of soil is also determined by finding out the pH value of the treated soil with a given percentage of lime. Percentage of lime which gives soil-lime mix a pH of 12.4, is taken to be the right percentage of lime required for stabilisation of the given soil. To find the optimum lime requirement of soil 20g of the soil sample passing 425 micron mesh is taken in a bottle. A given percentage of lime is added by weight and mixed thoroughly. To this is added 100 cc of distilled water and mixed thoroughly. The slurry so formed is kept at room temperature (25°C) and after an hour, it is shaken again to ensure a uniform mixing of ingredients and its pH is determined by a pH meter. The minimum percentage of lime which gives a pH of 12.4 when mixed with soil in the form of slurry, is the optimum percentage of lime for stabilisation.

(ii) Moisture Absorption Method

Capillary absorption of water is an intrinsic property of every soil. Besides other factors, this depends mainly on the mineralogical composition and particle size distribution. The addition of lime to soils brings about a change in the particle size distribution due to chemical reactions and the moisture absorption characteristics of soils change with the addition of lime. The change in moisture absorption continues upto a certain lime content depending upon the soil. With further addition of lime, the change in moisture absorption is negligible. The lime content at which the soil attains a steady moisture absorption is termed as the optimum requirement of lime for a particular soil. The method to carry moisture absorption test is given in *Appendix - 1*.

4.1.3. After ascertaining optimum lime content the strength of soil-lime mix is determined either by California Bearing Ratio or Unconfined Compressive Strength (see para 4.2.). Samples of CBR/UCS should be prepared at maximum dry density and optimum moisture content as per IS: 2720 (Part VII) or IS: 2720 (Part VIII) as specified. The specimen thus prepared should be cured for 3 days followed by 4 days soaking in water. At least three specimens should be tested. UCS design procedure should be preferred since it is more realistic as compared to CBR design criteria.

4.1.4. If the strength values of the soil-lime mix obtained at optimum lime-content are much more than those required from design consideration, strength tests may be conducted with reduced lime content to fix the design mix. However, in no case should the lime content be less than the minimum value of lime required from durability (Para 4.5.) and leaching (Para 4.6.) considerations.

4.2. Strength Criteria

For testing the strength of stabilised soils, the CBR test is widely used in

the case of gravelly soils whereas UCS test is often used with fine textured soils. Both tests are done at an age of seven days comprising three days moist curing followed by four days immersion in water. Guiding strength criteria are as follows:

(1) **CBR Test :** Minimum CBR value for the lime-stabilised sub-base should be 15 per cent for low trafficked rural roads, 20 per cent for cumulative traffic upto 2 million standard axles (MSA) and 30 per cent for traffic exceeding 2 MSA. The CBR test should be carried out on samples compacted to the density specified and moulded at optimum moisture content.

(2) **Unconfined Compression Test :** In terms of the unconfined strength, the lime stabilised soil used for sub-base should have a strength of 700 KN/Sq. m.

For purposes of design, the CBR value in the field should be assumed as 60 to 70 per cent of the one obtained in the laboratory in consideration of the anticipated efficiency of mixing, placing, curing and other aspects of construction in the field.

4.3. Degree of Pulverisation

4.3.1. For effective stabilisation, the soil must be in a well pulverised state before lime is added. Yet, it may not be economical to pulverise heavy clays like black cotton soil to an appreciable degree. With this in view, the requirements of pulverisation for different soils are setforth in Table-1.

TABLE 1. SOIL PULVERISATION REQUIREMENT FOR LIME STABILISATION

Sieve designation	Minimum per cent by weight passing the sieve	
	For black-cotton soil	For other soils
22.4 mm	100	100
4.00 mm	50	60

4.3.2. To determine the degree of pulverisation, method of sieving for wet cohesive soils as given in *Appendix - 2* shall be used.

4.4. Evaluation of Field Strength

Whenever a large number of strength tests are done on lime-stabilised layer, a significant variation in results is normally witnessed. This happens because of certain factors like heterogeneous character of the soil-lime mass, non-uniformity in mixing etc. Besides, a small deviation in the procedure of preparation of a sample and the testing techniques inevitably takes place. Variation in curing temperature and duration of curing also affect the strength results. In the presence of these uncertainties, test results can be interpreted statistically in two possible methods, either by taking mean of the test values or by computing standard deviation of values. From these measures, other useful quantities like coefficient of variation and confidence limit for true mean can be found. Results are taken to be fairly consistent if the coefficient of variation is upto 10 per cent. If the test results lie on either side of the design value and coefficient of variation is less than 10 per cent, the test results are taken to be in conformity with the design value. In case the mean happens to be either less or more than 10 per cent, then allowance is made in the design thickness either by effecting a corresponding increase or decrease in its value.

4.5. Durability

Durable soil-lime mixtures can be obtained when reactive soils are stabilised with lime. Although some strength reduction and volume change may occur due to alternate wetting and drying, the residual strength of stabilised material is generally adequate to meet the field service requirements. For the climatic conditions prevailing in India, durability under wetting and drying would have to be taken into consideration and durability under freeze-thaw conditions does not generally apply. The minimum seven day strength of 700 kN/Sq. m. recommended for subbase is expected to meet the durability requirements of wetting and drying under normal circumstances. However, in situations where waterlogging conditions prevail, the adoption of soil-lime stabilisation may not be a satisfactory solution from durability considerations. It would be desirable to consider such alternatives as composite stabilisation involving the use of lime and cement, which is not within the scope of these guidelines.

4.6. Prevention of Leaching

The amount of lime leached out from the lime-soil mix exposed to continual flow of water depends on the period of curing and to some extent, on the quantity of lime added. Test results have shown that if the lime stabilised sample is cured for 28 days before it is exposed to leaching process, the loss of lime due to leaching is not appreciable. The loss of lime ranges from 5 to 10 per cent of lime added. On consideration of gain in strength, it

is believed that percentage of lime to be added should be about 4.0 per cent or more. If this is done, the leaching problem is taken care of automatically.

5. MIX DESIGN

5.1. The proportion of lime for stabilisation should be determined in the laboratory in accordance with the following procedure:

(1) The soil should be tested for PI, fraction passing 425 micron sieve, sulphate content and organic content in order to evaluate its suitability for stabilisation (vide para 3.2.2.).

(2) Moisture density relationship for the soil should be established as per IS: 2720 (Part VII) - 1980 or IS: 2720 (Part VIII) - 1983.

(3) Lime to be used for stabilisation should be tested for available calcium oxide content in accordance with IS: 1514-1959 or IS: 712-1984.

(4) Evaluation of lime requirement is carried out as per para 4.1.

(5) After pulverising the soil to the degree stated in para 3.5. specimens for CBR/UCS tests are prepared with optimum percentage of lime, compacted at maximum dry density and optimum moisture content corresponding to IS: 2720 (Part VII) or IS: 2720 (Part VIII). The specimens thus prepared should be cured initially for 3 days and then soaked in water for 4 days before testing them for CBR/UCS as per IS: 2720 (Part XVI)-1979/IS: 2720 (Part X)-1973. At least 3 specimens should be tested for each lime concentration.

(6) On the basis of these results, the mix design should be selected using criteria set forth in para 4.1. and 4.2.

6. CONSTRUCTION OPERATIONS

6.1. Pulverisation and Mixing

6.1.1. Full benefit of lime stabilisation can be had only if the lime added to the soil is thoroughly mixed. From this consideration mechanical method should be preferred. Manual method of mixing may be accepted on only small or less important works. The method of mixing, whether mechanical or manual, should in any case ensure that lime is thoroughly and uniformly mixed with the soil. The compacted thickness of soil-lime layer or lift shall be in the range of 75-200 mm depending on efficiency of mixing and laying equipment.

6.1.2. **Mechanical methods:** Stabilised soil course shall preferably be constructed by mechanical method of construction. Mechanical methods of mixing lime-soil may be broadly grouped into two categories e.g. mix-in-place method and stationary plant method. Because of better controls, the stationary plant method should generally give better results.

(i) Mix-in-place method

The mix-in-place method permits rapid construction with a small labour force and at relatively low cost. The equipment required is simple and a large daily output can be maintained. Its disadvantages are the difficulty of obtaining uniformity of mix and difficulty of ensuring a uniform thickness of the processed soil. In the mix-in-place method, the following operations are involved:

(a) Pulverisation of soil
(b) Spreading lime
(c) Mixing
(d) Addition of water
(e) Final grading
(f) Compaction, and
(g) Curing

Pulverisation comprises of two stages, first scarifying the soil to the required depth of treatment and second pulverising the scarified soil until it is broken down to a size suitable for mixing with lime. Suitable plant for cutting up the soil to the required depth comprises a plough or robust tiller with a positive depth control. While scarifying the area the plough should move the soil towards the centre of the road; this leaves a vertical face of the soil at the shoulders and prevents processing being carried outside the limits of the road. Rotary tillers are used for pulverisation, but disc harrows can also be advantageously employed for some soils. When the pulverisation is completed, the loose material is shaped with a grader to give even distribution along the length and width of the road. Lime can be spread either mechanically or by hand. The mixing of soil with lime is usually done with the same plant as is used for pulverising the soil. Rotavators can also be used advantageously for performing the twin jobs of pulverisation and mixing. Water is added to the soil-lime mix by means of water distributors. The water distributor is followed by the mixing machine and the processing is continued till optimum moisture content is obtained. When mixing is complete, it is necessary to shape the surface again with a blade grader before compaction.

In the mix-in-place construction by using stabiliser, the plant shall be capable of pulverising the soil to the specified degree to the full thickness of the layer being processed, thereby, achieving the desired degree of mixing and uniformity of the stabilised material. The plant shall be either of single pass or multiple pass type. With single pass equipment, the forward speed of the machine will be so selected that the required degree of mixing, pulverisation and depth of processing is obtained.

In the multiple pass processing, the soil on the prepared subgrade shall be pulverised to the required depth with successive passes of the plant and moisture content adjusted to be within the prescribed limits. Lime shall then be spread uniformly and mixing continued with successive passes until the required depth and uniformity of processing has been obtained.

(ii) Stationary plant method

Stationary plants are essentially of two types, continuous and batch mix.

In the case of continuous mix type plant an elevating loader supplies soil to a hopper with a measuring gate. A belt conveyor discharges it to a pug mill, where water or fluid stabilizer may be added through spray nozzles and mixed into the soil. Sometime lime is added on the conveyor belt. The mixed material is then discharged into lorries.

On small jobs, concrete batch mixers can be used for mixing soil, lime and water. However, best results are achieved by double-paddle mixers, pug mills or roller pan type machines, in which soil lumps are easily broken up.

Usually mixtures produced by stationary plant cost more than those formed by the mix-in-place method, but certain advantages accrue from the use of this method. Advantages claimed for stationary plant method include greater uniformity of the mix, greater ease of control of the proportions of the mixture, greater ease in supplying water to the mix and fewer delays due to bad weather.

The sequence of operations involved in the plant mix method is as follows:

(a) Site preparation
(b) Collection and pulverisation

(c) Mixing
(d) Transporting and spreading
(e) Compaction, and
(f) Curing

After preparing the formation, the subgrade is thoroughly compacted by rolling before soil-lime layer is superimposed. The soil which has been tested and found suitable, is excavated from the road site or from borrow areas as the case may be, by means of suitable implements. All clods and lumps in the soil are broken up and when pulverisation is completed, it is stokpiled near the mixing plant. Mixing of soil, lime and water is done in the mixer. The mixed material is then transported to the site, tipped, spread and compacted in the normal manner.

6.1.3. **Manual mixing:** The manual method is more labour intensive, but because of its limitations in pulverising and mixing, it should be used for small jobs only. The success of manual method depends on strict quality control at site during pulverisation, mixing and compaction. Where manual mixing is permitted, the soil is excavated from the borrow areas after it is cleaned of all vegetation and placed on prepared subgrade. The soil shall then be pulverised with crowbars, pick axes etc. A sprinkling of water is given if the clods are dry and the soil is a heavy clay. The calculated quantity of lime stored in bags is placed on the track with equal spacings. If the soil is wet with excessive moisture, it is allowed to dry so that its moisture is reduced to the optimum value. If the soil is on the drier side, required quantity of water is added to bring it to the optimum moisture content. Lime is spread in requisite quantity on the excavated soil. To account for construction losses, uneven distribution of lime etc. add one per cent extra lime in the mix to get the designed field percentage. With the help of a shovel, the soil is turned upside down. This process is started from one end and is continued upto the other end cross-sectionally. This makes one turn and the same is repeated along the whole stretch of the road. Experience has shown that 4-5 turns are enough to obtain a thorough mixing of the material. Alternatively, the soil is excavated and is stacked along the track of the road pavement. Big clods are broken down to the desired size. Calculated amount of lime is sprinkled and water is added to bring it to the optimum moisture content. Mixing is done with shovel in the same way as stated earlier. Lime-soil mix is spread on the track and shaped with a blade grader before it is compacted.

6.2. Addition of Lime

Lime may be added to the prepared material either in slurry form or in dry state. No traffic other than mixing equipment will be allowed to pass over the spread material until the completion of mixing. Mixing shall continue till the material is free from any pocket where lime is deficient. The moisture content shall be equal to the optimum moisture content and shall not exceed OMC by more than 2.0 per cent. The lime content of samples taken from the completed mixture shall not vary above or below the designated lime content, by more than 1 per cent by weight.

6.3. Time Between Mixing and Compaction

6.3.1. Time interval between the mixing of lime and soil, and compaction of mix, has a definite effect on the gain achieved in strength. Long delay between the timings of these two operations results in less density and hence reduced strength of the soil-lime mix for the same compactive effort. If the same density and strength are to be achieved, extra compactive effort is required which involves extra cost and thus escalates the expenditure of the project. Both laboratory and field observations have shown that time gap between mixing and compaction should not exceed three to four hours.

6.4. Rolling

Immediately after spreading, grading and levelling of the mixed material, compaction shall be carried out with 8-10 tonne smooth wheel roller or vibratory roller. Rolling shall start from the edges and progress towards the centre. During rolling, the surface shall be frequently checked for grade and camber. Irregularities during rolling shall be corrected by loosening the material and removing or adding fresh material. Compaction shall continue till at least 100 per cent of the maximum dry density determined in accordance with IS: 2720 (Part VII) or IS: 2720 (Part VIII), as decided by the designer, is achieved in the field.

6.5. Curing

Duration of the curing period and the temperature at which curing takes place, have a significant influence on the strength achieved. Longer period of curing is conducive to attaining higher strengths since chemical reactions continue to take place for a long time. Normal period of curing varies from 7 to 28 days at normal temperature under wet conditions. Curing progresses faster in warm conditions. As a compromise between the objective of achieving high strength and maintaining a statisfactory schedule of progress, it is suggested that a minimum 7 days curing should be given to the lime-soil mix under moist conditions.

Two common methods employed for curing are asphaltic membrane curing and moist curing. In membrane curing, a prime coat of cut-back bitumen is applied at a rate of about 0.45 to 1.1 Litre/Sq.m within one day of rolling. The prime coat is meant to assist curing of stabilised soil by inhibiting the evaporation of moisture. No equipment or traffic should be permitted on lime stabilised surface during first 3 days after applying the prime coat. If the compacted stabilised layer is not rutted or distorted by the equipment, the immediate placement of overlay is permitted. This overlay maintains the moisture content of the compacted layer and is an adequate medium for curing.

The other method of curing is moist curing. In this method, the surface is kept damp by light sprinkling of water at frequent intervals to prevent drying with light rollers being used to keep the surface knitted together. Light vehicles may be permitted on the finished surface but heavy traffic should not be allowed for 10 to 15 days.

7. QUALITY CONTROL

7.1. Depth of the Treatment and Uniformity of Mixing

Since lime elevates the pH value of the soil, phenolphthalein alcohol indicator solution can be sprayed on the soil to determine the presence of lime. If lime is present, a reddish pink colour develops and indicates the depth of mixing.

Non uniformity of colour reaction, when the treated material is tested with the standard phenolphthalein alcohol indicator, will be considered evidence for inadequate mixing. One test per 500 Sq. m should be carried out for determination of depth and uniformity of mixing.

7.2. Purity of Lime

One test should be carried out for each consignment corresponding to IS: 1514-1959 or IS: 712-1964, subject to a minimum of one test per 5 tonnes of lime. The test shall be done just before lime is used in the stabilisation work.

7.3. Lime Content Immediately After Mixing

One test per 250 Sq.m should be carried out corresponding to IS: 4332 (Part VIII)-1969.

7.4. Degree of Pulverization

Degree of pulverization shall be checked regularly and the same shall conform to the specification given in Table 1.

7.5. Moisture Content Prior to Compaction

Moisture content should be determined as per IS: 2720 (Part II)-1973 at a rate of one test per 250 Sq. m.

7.6. Dry Density of Compacted Layer

Density should be checked according to IS: 2720 (Part XXVIII)-1974. One test per 500 Sq. m should be carried out.

7.7. Layer Thickness and Longitudinal Profile

Layer thickness should be measured regularly by thickness blocks or cores to conform to specified thickness.

Actual finished levels of different courses shall not vary from the design levels beyond the tolerances mentioned below:

Sub-grade ± 25 mm
Sub-base ± 20 mm

7.8. Surface Regularity

The surface regularity of stabilised subgrade and sub-base in longitudinal and transverse directions shall be within the tolerances indicated below:

Layer	Longitudinal Profile with 3 m straight edge			Cross profile
	Max. permissible undulations	Max. Number of Permissible undulations in any 300 m. length exceeding		Max. permissible variation from specified profile under camber template
		18 mm	12 mm	mm
Subgrade	24	30	—	15
Sub-base	15	—	30	12

7.9. CBR/UCS Tests on Mixed Materials

One test per 3000 Sq. m should be carried out corresponding to IS: 2720 (Part-XVI)-1979 for CBR and IS: 2720 (Part X)-1973 for UCS.

7.10. Deleterious Constituents of Soil

Soil should be tested regularly for deleterious constituent corresponding to IS: 2720 (Part XXII)-1972.

7.11. Curing

A record should be maintained for the curing period to ensure proper curing for 7 days or any other period as specified.

8. LIMITATIONS

Apart from organic soils and some rare soils which react unfavourably with lime because of chemical contamination, the only other physical factors limiting the use of soils for stabilisation are:

(i) That it must be possible to break the soil to a fine tilth in order to mix in a stabiliser,

(ii) That the soil should have an adequately stable grading,

(iii) That necessary clay minerals are there in the soil for the stabilisation reaction to proceed. A useful working limit is that soil should contain at least 15 per cent of fines passing 425 micron sieve and have a plasticity index of at least 10 per cent.

(iv) Lime-soil stabilisation should not be done when the air temperature in the shade is less than 10°C.

(v) Lime stabilization should not be carried out during rains.

Appendix-1

PROCEDURE FOR THE DETERMINATION OF MOISTURE ABSORPTION

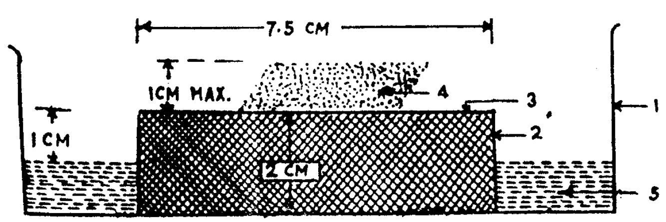
Apparatus

1. Trough
2. Porous Plate (thickness 2 cm and 7.5 cm diameter)
3. Filter paper (Whatman 42)
4. Distilled water
5. Spatula
6. Physical balance, sensitivity 0.01 gm
7. Weight box
8. Oven for drying
9. Petri dish

The porous plate is cleaned by boiling in water for about 15 minutes. The plate is taken out and placed in the trough. Distilled water is poured in the trough so that its level reaches about 1 cm below the top of the porous plate. A dry filter paper cut to the same diameter as that of the plate is carefully placed on the top of the porous plate.

15-20 gm of soil or soil-lime mix is carefully stacked loosely to a height of about 1 cm. (Fig. 1) and is allowed to absorb moisture from the trough for 1 hour. The porous plate is taken out and the excess moisture is allowed to drain off for 5 minutes. Moisture content of the sample is determined according to Indian Standards No. IS: 2720 (Part-II)-1973. This moisture content is termed as moisture absorption.

Average of 3 such determinations should be taken.



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1 Trough
2 Porous Plate
3 Filter Paper
4 Soil Sample
5 Distilled Water

Fig. 1. Assemblage for moisture absorption

Appendix-2

METHOD OF SIEVING FOR WET SOILS TO DETERMINE THE DEGREE OF PULVERISATION

1. A sample of pulverised soil approximately 1 kg by weight should be taken and weighed (W1).

2. It should be spread on the sieve and shaken gently, care being taken to break the lumps of soil as little as possible. Weight of soil retained on the sieve should be recorded (W2). Lumps of finer soils in the retained material should be broken until all the individual particles finer than the aperture size of the sieve are separated.

3. The soil should be again placed on the sieve and shaken until sieving is completed. The retained material should be weighed (W3).

4. Weight of soil by per cent passing the sieve can then be calculated from the expression:

$$\frac{(W1\text{-}W2)\ 100}{(W1\text{-}W3)}$$

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24.	G.S. Palnitkar	Engineer-in-Chief, MPPWD
25.	M.M. Patnaik	Engineer-in-Chief-cum-Secretary to the Govt.
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38.	The Director	Highways Research Station, Madras
39.	The Engineer-in-Chief	Haryana P.W.D., B&R.
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41.	The Director General	(Road Development) & Addl. Secretary to the Govt. of India (K.K. Sarin) — *(Ex-officio)*
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ROAD ACCIDENT FORMS A-1 AND 4

THE INDIAN ROADS CONGRESS

ROAD ACCIDENT FORMS A-1 AND 4

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ROAD ACCIDENT FORMS A-1 AND 4

1. INTRODUCTION

Accurate and comprehensive acc[redacted]ds are the foundation of the accident analysis. The effective use of accident records, however, depends upon three factors, viz. accuracy of data, maintenance of records and analysis of data. Need for a high standard of accident reporting is the principal pre-requisite for the use of accident records in working out road safety measures. If the original accident report themselves are poor, the analysis and the results from their use will also be poor. Inaccurate and incomplete accident data make the results vague, misleading and not fruitful.

In India, the details of day to day road accidents are generally being collected by the Traffic Police in various States in Road Accident Form A-1. This form was suggested at the fourth meeting of the Transport Advisory Council held in Delhi in July, 1939. Besides this Form A-1, the Traffic Police also prepare summary of road accidents in a State during a year in the Road Accident Form 4. From these statistics, summaries are prepared by various authorities.

There were many deficiencies in the existing Form A-1 with the result that the details supplied by the Traffic Police were insufficient for making scientific investigation of road accidents. Although this form covered the details of road accidents, like severity, location, day, month, time, vehicles involved, age and sex of the driver primarily involved in accident and also of persons killed and injured, light condition, road condition, etc., there was no provision for a number of [redacted]etry of the [redacted], (iii) type [redacted]er, if any, (v) [redacted] behaviour. More[redacted]alysing the data [redacted]at the same time, be accurate.

Keeping these requirements in view, the Traffic Engineering Committee in their meeting held on the 3rd October, 1967 set up a Panel consisting of the following to go into this question and suggest suitable forms workable on an all-India basis :

1. Dr. V.G. Bhatia ... Convenor
2. K.C. Saxena ... Member-Secretary
3. T.R. Sehgal
4. Dr. N.S. Srinivasan
5. R.P. Sikka
6. P.G. Mukherjee, Dy. Commissioner of Police (Traffic), Calcutta
7. Deputy Commissioner of Police, Bombay
8. Deputy Commissioner of Police, Madras
9. Superintendent of Police (Traffic), New Delhi
10. Miss D.V. Hingorani

This Panel formulated revised Form A-1. Later the Traffic Engineering Committee in their meeting held on the 14th & 15th May, 1971 and 2nd & 3rd November, 1972 respectively approved revised Form A-1 and formulated revised Form 4 and recommended them for uniform adoption in the country by the police and other traffic authorities collecting and maintaining road accident statistics. These were then approved by the Specifications and Standards Committee in their meetings held on the 29th and 30th September, 1972. Later, these were approved by the Executive Committee in their meeting held on the 25th April, 1973 and the Council in their 81st meeting held on the 26th April, 1973. This document was published by the I.R.C. during 1973 and was sent to all concerned for adoption.

The Chief Engineers, Transport Commissioners and Inspector General of Police of all States and also the various concerned Central Government Departments were addressed in August 1978 to send their comments in regard to the use of these forms with a view to consider whether any changes are necessary in these forms in the light of their experiences. [redacted] comments received from various authorities we[redacted] placed before the Traffic Engineering Committee [redacted] the Indian Roads Congress which in turn set up a Sub-committee (names mentioned below) to review these comments and revise the Road Accident Forms A-1 and 4 wherever considered necessary:

Dr. N.S. Srinivasan
Dr. (Mrs.) L.R. Barthakur
P.S. Bawa
R.P. Sikka
D.P. Gupta
H.A. Bindra

Articulated Vehicles : A unit made up of a road motor vehicle and a semi-trailer.

Nature of Collision : Positions and directions of vehicles in each case are as indicated in the diagrams below :

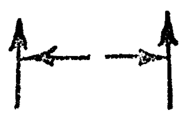
(i) Head on collision
(ii) Rear end collision
(iii) Brush/side swipe
(iv) Right angled collision
(v) Right turn collision

(i)

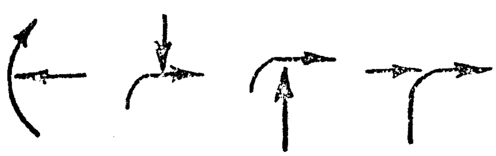
(ii)

(iii)

(iv)



(v)



Types of Insurance :

Comprehensive : Covers all risks.

Third Party : For the purpose of reporting in this form, this category includes all types of insurance, other than comprehensive insurance.

3. CLASSIFICATION OF HIGHWAYS

Non-urban roads in the country have been classified into five categories as detailed below :

National Highways : These are main highways running through the length and breadth of the country connecting major ports, foreign highways, State capitals, large industrial and tourist centres etc.

State Highways : These are arterial routes of a State linking district headquarters and important cities within the State and connecting them with National Highways or highways of the neighbouring States.

Major District Roads : These are important roads within a district serving areas of production and markets, and connecting these with each other or with the main highways.

Other District Roads : These are roads serving rural areas of production and providing them with outlet to market centres, taluka/tehsil headquarters, block development headquarters, or other main roads.

Village Roads : These are roads connecting villages or groups of villages with each other and to the nearest road of a higher category.

ROAD ACCIDENT REPORTING FORM A1

1. IDENTIFICATION PARTICULARS

(i) State
(ii) District/Taluk
(iii) City/Town/Village
(iv) Police Zone/Station
(v) Accident number alloted by Police Station
FIR number
MVA accident No.

2. LOCATION

(i) Name of road
(ii) Classification of road

☐ 1. National Highway
☐ 2. State Highway
☐ 3. Other road

(iii) Location of area ☐ 1. Urban
☐ 2. Rural

(iv) If intersection, specify the names of roads

(v) Details of location (the details under this should be such as to give precise location)

(vi) Type of area

☐ 1. Near school or college
☐ 2. Near or inside a village
☐ 3. Near a factory/industrial area
☐ 4. Near a religious place
☐ 5. Near a recreation place/cinema
☐ 6. In bazaar
☐ 7. Near office complex
☐ 8. Near hospital
☐ 9. Residential area
☐ 10. Open area
☐ 11. Near bus stop
☐ 12. Near petrol pump
☐ 13. At pedestrian crossing
☐ 14. Affected by encroachments

(vii) Narrow bridge or culverts ☐ 1. Yes
☐ 2. No

3. DATE, DAY AND TIME

(i) Date Month Year
(ii) Day of the week
(iii) Time A.M./ P.M.
(iv) Holiday 1. ☐ Yes 2. ☐ No.

4. LIGHT CONDITIONS

☐ 1. Daylight
☐ 2. Twilight
☐ 3. Dark hours with good street light
☐ 4. Dark hours with poor street light
☐ 5. Dark hours with no street light

5. WEATHER CONDITIONS

☐ 1. Fine
☐ 2. Mist/Fog
☐ 3. Cloudy
☐ 4. Light rain
☐ 5. Heavy rain
☐ 6. Hail/sleet
☐ 7. Snow
☐ 8. Strong wind
☐ 9. Dust Storm
☐ 10. Very hot
☐ 11. Very cold
☐ 12. Other extraordinary weather conditions (specify)

6. CLASSIFICATION OF ACCIDENT

☐ 1. Fatal
☐ 2. Grievous injury
☐ 3. Minor injury
☐ 4. Non injury

7. TYPE OF VEHICLES WITH REGISTRATION NUMBERS AND OBJECTS INVOLVED

7.1. Vehicles involved

	Type of vehicle	Registration number	Year of manufacture
Vehicle 1	______	______	______
Vehicle 2	______	______	______
Vehicle 3	______	______	______

7.2. Pedestrian, Animal and other Objects involved (specify type)

Sl.No.	
1	______
2	______
3	______

8. NATURE OF THE ACCIDENT

8.1. Type

☐ 1. Overturning
☐ 2. Head on collision
☐ 3. Rear end collision
☐ 4. Collision brush/side swipe
☐ 5. Right angled collision
☐ 6. Skidding
☐ 7. Right turn collision
☐ 8. Others (describe)

8.2. Hit and run 1. Yes 2. No

9. DETAILS OF DRIVERS OF VEHICLES INVOLVED

9.1. Name, Sex, Age, Education and Address of the Driver(s)

	Name of Driver	Sex M/F	Age	Highest class passed	Address
Veh.1	______	___	___	______	______
Veh.2	______	___	___	______	______
Veh.3	______	___	___	______	______

9.2. Person driving the Vehicle

		Veh. 1	Veh. 2	Veh. 3
Owner of private vehicle	1	☐	☐	☐
Owner of public/commercial vehicle	2	☐	☐	☐
Paid driver	3	☐	☐	☐
Other	4	☐	☐	☐

12. DETAILS OF CYCLISTS INVOLVED

- ☐ 1. Double riding
- ☐ 2. Overloading
- ☐ 3. Not keeping to the left
- ☐ 4. Without light at night
- ☐ 5. Going on carriageway - cycle track available
- ☐ 6. Cycling in the lane of fast moving vehicle
- ☐ 7. Cutting in the flow of traffic or zigzag moving
- ☐ 8. Turning right carelessly/without giving signal
- ☐ 9. Towing himself with other vehicle
- ☐ 10. Not observing traffic rules
- ☐ 11. Confused by traffic
- ☐ 12. Rider inexperienced
- ☐ 13. Loss of control
- ☐ 14. Skidding on wet/slippery road
- ☐ 15. Others (specify)

13. TYPE OF PERSONS AND ANIMALS KILLED OR INJURED

13.1. Type and Number of Persons Killed or Injured

Type	Number Killed		Number Injured			
			grievously		slightly	
	Male	Female	Male	Female	Male	Female
1. Pedestrian	____	____	____	____	____	____
2. Bicycles						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
3. Motor Cycles						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
4. Scooters						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
5. Mopeds						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
6. Autorickshaws						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
7. Cars, taxis, vans, and other light and medium motor vehicles						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
8. Trucks						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
9. Buses						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
10. Other motor vehicles						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
11. Animal driven vehicles						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
12. Cycle rickshaws						
(i) Drivers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
13. Hand carts and rickshaws						
(i) Pullers	____	____	____	____	____	____
(ii) Passengers	____	____	____	____	____	____
14. Other persons	____	____	____	____	____	____

13.2. Number of Animals Killed/Injured

Killed ____________

Injured ____________

14. TYPE OF DAMAGE TO VEHICLES AND PROPERTY

Vehicle/Property	Type of damage (specify)
1. Vehicle 1	____________
2. Vehicle 2	____________
3. Vehicle 3	____________
4. Other property	____________

15. ROAD CONDITION

15.1. Geometry of the Road

(i) Horizontal features of the road

- ☐ 1. Straight road
- ☐ 2. Slight curve
- ☐ 3. Sharp curve

(ii) Vertical features of the road

- ☐ 1. Flat road
- ☐ 2. Gentle incline
- ☐ 3. Steep incline
- ☐ 4. Hump
- ☐ 5. Dip

15.2. Type of Surface

- ☐ 1. Surfaced (black topped/concrete)
- ☐ 2. Metalled
- ☐ 3. Kutcha

15.3. Condition of Surface

- ☐ 1. Dry
- ☐ 2. Wet

15.4. Nature of Surface

- ☐ 1. Good surface
- ☐ 2. Loose surface
- ☐ 3. Rutted and/or potholed
- ☐ 4. Road under repair/construction
- ☐ 5. Corrugated or wavy road
- ☐ 6. Slippery surface
- ☐ 7. Snowy
- ☐ 8. Muddy
- ☐ 9. Oily
- ☐ 10. Speed breaker
- ☐ 11. Others (specify)

16. ROAD FEATURES

(i) Carriageway

- ☐ 1. Single lane
- ☐ 2. Two lanes
- ☐ 3. Three lanes or more without central divider (median)
- ☐ 4. Four lanes or more with central divider

(ii) Cycle track provided 1.☐ Yes 2.☐ No

(iii) Foot path provided 1.☐ Yes 2.☐ No

(iv) Pucca shoulder provided 1.☐ Yes 2.☐ No

ROAD ACCIDENT FORM-4

Statement containing particulars of road accidents in
the State .... for the year ending December....................

1. TOTAL NUMBER OF ACCIDENTS CLASSIFIED ACCORDING TO MONTH OF THE YEAR

Month	Number of accidents				
	Fatal (F)	Grievous injury (GI)	Minor injury (MI)	Non-injury (NI)	Total
1. January					
2. February					
3. March					
4. April					
5. May					
6. June					
7. July					
8. August					
9. September					
10. October					
11. November					
12. December					
Total					

2. ACCIDENTS CLASSIFIED ACCORDING TO TYPE OF AREA AND TIME

Time	Number of accidents									
	URBAN					RURAL				
	F	GI	MI	NI	Total	F	GI	MI	NI	Total
0600—0700										
0700—0800										
0800—0900										
0900—1000										
1000—1100										
1100—1200										
1200—1300										
1300—1400										
1400—1500										
1500—1600										
1600—1700										
1700—1800										
1800—1900										
1900—2000										
2000—2100										
2100—2200										
2200—2300										
2300—2400										
2400—0100										
0100—0200										
0200—0300										
0300—0400										
0400—0500										
0500—0600										

3. ACCIDENTS CLASSIFIED ACCORDING TO CLASSIFICATION OF ROAD

Classification of Road	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
1. National Highway					
2. State Highway					
3. Other Road					

Type of vehicle	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
(B) Other Vehicles & Objects :					
1. Cycle					
2. Cycle rickshaw					
3. Hand drawn vehicle					
4. Animal drawn vehicle					
5. Pedestrian					
6. Animal					
7. Tree					
8. Level crossing					
9. Other fixed objects					

7. ACCIDENTS CLASSIFIED ACCORDING TO AGE OF VEHICLE

Age of vehicle (in year)	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
Less than 1 year					
1—2					
2—4					
4—6					
6—8					
8—10					
10 and above					

8. ACCIDENTS CLASSIFIED ACCORDING TO NATURE OF ACCIDENT

Nature of accident	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
1. Overturning					
2. Head on collision					
3. Rear end collision					
4. Collision brush/side swipe					
5. Right angled collision					
6. Skidding					
7. Right turn collision					
8. Hit and run					
9. Others					

9. ACCIDENTS CLASSIFIED ACCORDING TO CAUSE

Cause of accident	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
1. Fault of driver of motor vehicle					
2. Fault of cyclist					
3. Fault of driver of other vehicle					
4. Fault of pedestrian					
5. Fault of passenger					
6. Defect in mechanical condition of motor vehicle					
7. Poor light condition					
8. Defect in road condition					
9. Result of weather conditions					
10. Other causes					
11. Cause not known					

12. ACCIDENTS CLASSIFIED ACCORDING TO RESPONSIBILITY OF DRIVER

Fault of driver	Number of accidents				
	Fatal	Grievous Injury	Minor Injury	Non-Injury	Total
1. Consumption of alcohol or drugged					
2. Exceeded lawful speed					
3. Did not give right of way to vehicle					
4. Did not give right of way to pedestrian					
5. Followed too closely					
6. Passed on hill					
7. Passed on curve					
8. Cut in sharply after passing					
9. Other improper passing					
10. On wrong side of the road					
11. Failed to give signal					
12. Gave improper signal					
13. Improper turn					
14. Disregarded Police Officer					
15. Disregarded traffic light signal					
16. Disregarded stop sign					
17. Improper starting from parked position					
18. Wrong or improper parking location					
19. Asleep or fatigued or sick					
20. Inattentive or attention diverted at the moment					
21. Improper use of headlights					
22. Other improper actions					

13. ACCIDENTS CLASSIFIED ACCORDING TO PARTICULARS OF VEHICLES INVOLVED

Particulars of vehicles	Number of accidents				
	Fatal	Grievous Injury	Minor Injury	Non-Injury	Total
(A) Load					
(i) Overloaded/overcrowded					
(ii) Load protruding					
(B) Left Hand Drive					
(C) Vehicular Defect					
(i) Defective brakes					
(ii) Defective steering					
(iii) Punctured or burst tyres					
(iv) Bald tyre					
(v) Other serious mechanical defect					
(D) Certificate of Fitness in the case of Commercial Vehicles					
(i) In force					
(ii) Not in force (expired)					

16 ACCIDENTS CLASSIFIED ACCORDING TO ROAD CONDITION

Road condition	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
(A) Type of Surface					
1. Surfaced					
2. Metalled					
3. Kutcha					
(B) Condition of Surface					
1. Dry					
2. Wet					
(C) Nature of Surface					
1. Good surface					
2. Loose Surface					
3. Rutted and/or pot holed					
4. Road under repair/construction					
5. Corrugated or wavy road					
6. Slippery surface					
7. Snowy					
8. Muddy					
9. Oily					
10. Speed breaker					
11. Others					
(D) Horizontal Features					
1. Straight road					
2. Slight curve					
3. Sharp curve					
(E) Vertical Feature					
1. Flat road					
2. Gentle incline					
3. Steep incline					
4. Hump					
5. Dip					

17. ACCIDENTS CLASSIFIED ACCORDING TO ROAD FEATURES

Carriageway width	Number of accidents				
	Fatal	Grievous injury	Minor injury	Non-injury	Total
1. Single lane					
2. Two lanes					
3. Three lanes or more without central divider (median)					
4. Four lanes or more with central divider					

LATERAL AND VERTICAL CLEARANCES AT UNDERPASSES FOR VEHICULAR TRAFFIC

THE INDIAN ROADS CONGRESS
1987

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LATERAL AND VERTICAL CLEARANCES AT UNDERPASSES FOR VEHICULAR TRAFFIC

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STANDARD FOR LATERAL AND VERTICAL CLEARANCES AT UNDERPASSES FOR VEHICULAR TRAFFIC

1. INTRODUCTION

This Standard was first discussed by the Specifications & Standards Committee in their meeting held at Gandhinagar on the 30th November 1972. Later, it was approved by that Committee in their meeting held at New Delhi on the 31st January and 1st February 1974 and then by the Executive Committee in their meeting held on the 1st May 1974. Finally, it was approved by the Council in their 82nd meeting held on the 2nd May 1974.

2. GENERAL

2.1. Many times a road has to be taken through an underpass below another road, railway line, pipeline or irrigation facility like aquaduct. In order that capacity, speed and safety of travel are not affected, the lateral and vertical clearances at underpasses must be adequate.

2.2. Desirable practices in this regard are indicated herein. It is recommended that these may be followed uniformly on all roads throughout the country.

3. SCOPE

3.1. The Standard covers both rural and urban roads. Specific cases of subways meant for the exclusive use of cyclists or pedestrians are, however, not dealt with. Guidance about clearances on cycle subways is contained in IRC: 11-1962 "Recommended Practice for the Design and Layout of Cycle Tracks". For pedestrian subways, another standard is proposd to be issued in due course.

4. DEFINITIONS

The following definitions will be applicable for the purpose of this standard :

4.1. **Underpass** implies a short passage beneath a grade-separated structure to carry one or more streams of traffic.

4.2. **Lateral clerance** is the distance between the extreme edge of the carriageway to the face of the nearest support whether it is a solid abutment, pier or column.

4.3. **Vertical clearance** stands for the height above the highest point of the travelled way, i.e., the carriageway and part of the shoulders meant for vehicular use, to the lowest point of the overhead structure.

4.4. Rural roads stand for roads of non-urban character.

5. OVERALL CONSIDERATIONS

5.1. Conscious effort must be made to create a sense of freedom for the drivers travelling through the underpass. As far as possible, the underpass roadway should conform to the natural lines of the highway at the approaches as regards alignment, profile and cross-section. Road profile should not dip too sharply under the structure as that will produce a considerably enhanced sense of restriction when compared with a profile that proceeds smoothly through.

5.2. To promote a feeling of openness and unrestrained lateral clearance, preferably structures with open-end spans should be employed, Fig. 1. Where it becomes inescapable to have structures with solid abutments, these should be set back from the roadway edge as much as possible, Fig. 2. From considerations of cost, these treatments are meant for higher categories of roads, especially with divided carriageways.

5.3. Since width at an existing underpass cannot be easily increased later on, initial construction should be sufficient for the standards to which the underpass roadway would need to be improved within the near future. This is essential especially for important routes like National and State Highways to be widened soon from single-lane to two-lane standards, as also busy two-lane roads which are in the planning stage for being upgraded to a four-lane divided cross-section.

5.4. Protect vehicles from accidents with abutments or piers. Guard-rails must be provided at a sutitable height. These should be of a robust design to effectively resist disturbance of the support in the event of a collision. In addition, ends of the guard-rails should be turned away from the line of approaching traffic, as shown in Fig. 3, so as to deflect runaway vehicles which may otherwise hit the underpass structure. As a general rule, guard-rails must be provided on both sides of the central piers or columns, though these

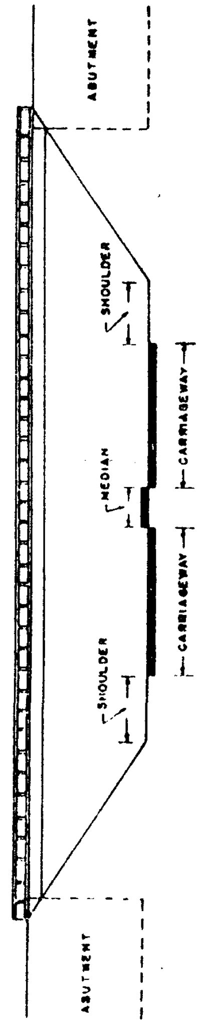


Fig. 1. Underpass with open end spans

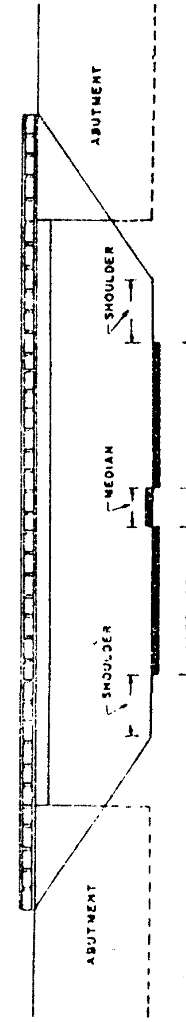


Fig. 2. Underpass with solid abutments offset from the shoulders

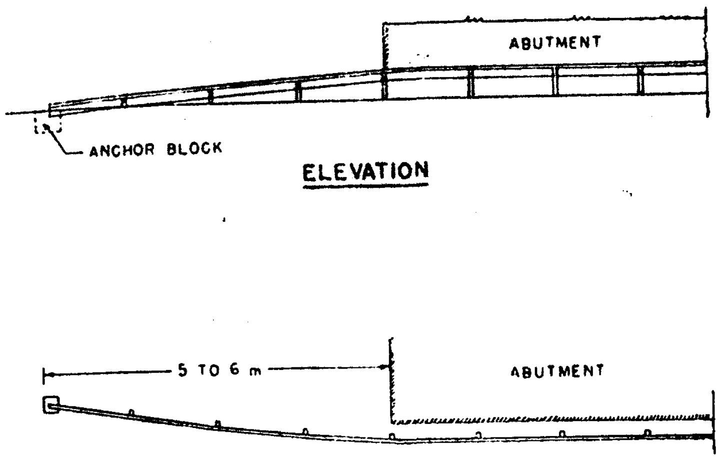


Fig. 3. Guard-rail end treatment
(not to scale)

could be dispensed with on the abutment side when a raised footpath forms part of the cross-section.

6. LATERAL CLEARANCE ON RURAL ROADS

6.1. Single Carriageway

6.1.1. Desirably the full roadway width at the approaches should be carried through the underpass. This implies that the minimum lateral clearance on either side must equal the shoulder width. This rule should be relaxed only in exceptional circumstances. Normal and exceptional values of lateral clearance for different classes of highways are given below (see Fig. 4a) :

(i) National and State Highways	Normal 2.5 metres; exceptional 2.0 metres
(ii) Major District and Other District Roads	Normal 2.0 metres exceptional 1.5 metres
(iii) Village Roads	Normal 1.5 metres : exceptional 1.0 metre

6.1.2. If a footpath is needed on a rural road, lateral clearance in the underpass portion should be the width of the footpath plus one metre, Fig.4 (b). Footpath width depends upon the expected pedestrian traffic and might be fixed with the help of following capacity guidelines, subject to not being less than 1.5 metres :

Anticipated capacity Number of persons per hour		Required footpath width
All in one direction	In both directions	
1200	800	1.5 m
2400	1600	2.0 m
3600	2400	2.5 m

6.2. Divided Carriageways

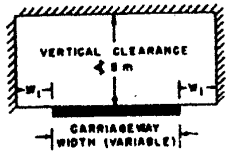
6.2.1. When an underpass is built for a divided highway, left hand side clearance shall be in accordance with para 6.1.1. If footpaths are provided in addition, para 6.1.2. should be applied.

6.2.2. Lateral clearance on the right to a pier or column in the central median shall be 2 metres desirably, and 1.5 metres at the minimum. Where the central median is kerbed, the carriageway width should be increased by the side safety margin of 0.5 metre as shown in Fig. 4 (c). Lateral clearance in that event could be reduced to 1.5 metres (desirable value) or I metre (exceptional). If the median is not wide enough to permit these clearances, either it should be widened gradually at the approaches or a single span structure provided across the full cross-section thereby avoiding a central pier.

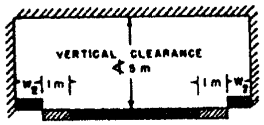
7. LATERAL CLEARANCE ON URBAN ROADS

7.1. Single Carriageways

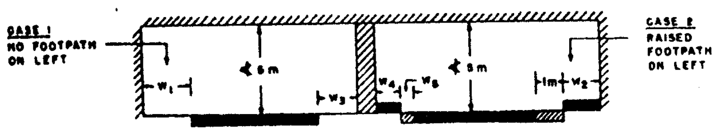
7.1.1. Usually roads in urban areas are bordered by kerbs on both sides. If so, these should be extended across the underpass. However, to offset the effect of kerb shyness, the carriageway in the underpass area should be widened on both sides by the side safety margin of 0.25 metre in the case of lower category urban roads



(a) SINGLE CARRIAGEWAYS WITHOUT FOOTPATHS



(b) SINGLE CARRIAGEWAYS WITH FOOTPATHS



(c) DIVIDED CARRIAGEWAYS

Notes :

W_1=lateral clearance vide para 6.1.1.

W_2=footpath width vide para 6.1.2.

W_3=right lateral clearance without kerbs; 2 m-desirable, 1 m-exceptional

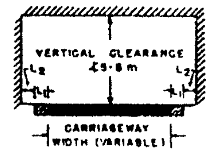
W_4=right lateral clearance with kerbs;
1.5 m desirable, 1 m-exceptional

W_5=side safety margin, i.e. extra carriageway width to offset kerb shyness, 0.5 m.

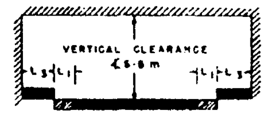
Fig. 4. Lateral and vertical clearances for rural roads
(not to scale)

and 0.5 metre in the case of higher category urban roads, Fig. 5(a).

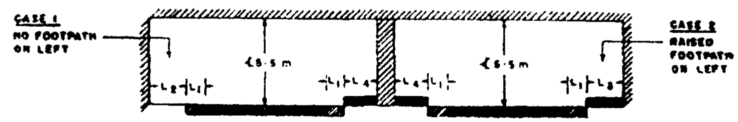
7.1.2. If a footpath does not form part of the cross-section of the urban road, the minimum lateral clearance in addition to the side safety margin mentioned in para 7.1.1. shall be 0.5 metre for lower category urban roads and 1 metre for higher category roads, Fig. 5 (a).



(a) SINGLE CARRIAGEWAYS WITHOUT FOOTPATH



(b) SINGLE CARRIAGEWAYS WITH FOOTPATH



(c) DIVIDED CARRIAGEWAYS

Notes :

L_1 = side safety margin, i.e. extra width to offset kerb shyness; **0.25 m** for lower category roads; 0.5 m for higher category roads.

L_2 = 0.5 m for lower category roads; 0.1m higher category roads

L_3 = footpath width, vide para 6.1.2.

L_4 = 0.5 m for lower category roads; 1.0 m for higher category roads,

Fig. 5. Lateral and vertical clearances for urban roads
(not to scale)

7.1.3. Where a raised footpath is provided, it will not be necessary to have additional clearance beyond the width of the footpath, Fig. 5 (b). Footpath width could be fixed in accordance with para 6.1.2.

7.2. **Divided Carriageways**

7.2.1. Where the underpass serves a divided facility, the width of the carriageway should be increased on either side by the side safety margin stated in para 7.1.1.

7.2.2. Lateral clearances on the left hand side should conform to paras 7.1.2. and 7.1.3. Right lateral clearances to the face of any structure in the central median over and above the side safety margin shall be at least 1 metre in the case of higher category urban roads and 0.5 metre in the case of lower category urban roads, Fig. 5 (c). A single span structure will no doubt be preferable as brought out in para 6.2.2.

8. VERTICAL CLEARANCE

Vertical clearance at underpasses shall be at least 5 metres. However, in urban areas, this should be increased to 5.50 metres so that double-decker buses could be accommodated.

IRC : 58-1988

GUIDELINES FOR THE DESIGN OF RIGID PAVEMENTS FOR HIGHWAYS

(First Revision)

THE INDIAN ROADS CONGRESS
1991

GUIDELINES
FOR
THE DESIGN OF RIGID PAVEMENTS
FOR
HIGHWAYS

(First Revision)

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CONTENTS

GUIDELINES FOR THE DESIGN OF RIGID PAVEMENTS FOR HIGHWAYS

1. INTRODUCTION

Guidelines for the Design of Rigid Pavements for Highways were approved by the Cement Concrete Road Surfacing Committee in their meeting held at Chandigarh on the 11th March, 1973. These were approved by the Specifications & Standards Committee in their meeting held on 31st January and Ist February, 1974. The Guidelines were then approved by the Executive Committee and the Council in their meetings held on the Ist May and 2nd May, 1974 respectively.

In view of the recent upward revision of the legal limit on the maximum laden axle-loads of commercial vehicles from 8160 kg. to 10200 kg. (new legal maximum wheel load 5100 kg.), appropriate modifications have become necesary in some sections of the Guidelines.

Accordingly, the Cement Concrete Road Surfacing Committee of the Indian Roads Congress in their 17th meeting held at Nagpur on 8th January, 1984 (personnel given below) considered and approved certain changes :

K.K. Nambiar	...	*Convenor*
Y.R. Phull	...	*Member-Secretary*

H.S. Bhatia	L. Shivalingaiah
D.C. Chaturvedi	N. Sivaguru
N.C. Duggal	K. Suryanarayana Rao
O.P. Gupta	Director (Civil) I.S.I. (G. Raman)
P.K. Issac	D.G.B.R. (Maj. Gen. J.M. Rai)
P.J. Jagus	B.T. Unwalla
D.P. Jain	Director General Cement Research Institute of India (Dr. H C. Visvesvaraya)
R.S. Jindal	Director, U.P. P.W.D. Research Institute (P.D. Agarwal)
Maj. Gen. R.K. Kalra	City Engineer (Roads), Municipal Corporation of Bombay
P.V. Kamat	A Rep. of C.P.W.D.
Dr. S.K. Khanna	D G. (R.D.) —*Ex-officio*
P.J. Mehta	
C.B. Mathur	
D.C. Panda	

The amendments were considered by the Specifications & Standards Committee in their meeting held at New Delhi on the 21st August, 1985 and were returned back to Cement Concrete Road Surfacing Committee for further consideration. The draft was then finalised by Dr. M.P. Dhir Convenor and Shri S.S. Seehra Member- Secretary of the reconstituted Committee. The draft received from the Cement Concrete Road Surfacing Committee was reconsidered by the Highways Specifications & Standards Committee in their meeting held on 25th April, 1988 at New Delhi and approved. These amendments received the approval of the Executive Committee and the Council in their meetings held on 26th April and 7th May, 1988 respectively.

2. GENERAL

Rigid pavement design commenced with the classical analysis of Westergaard in 1926. Most of the subsequent work, till recently, aimed at modifications and adaptations of Westergaard's work either with a view to match better with the actual performance and test data, or simplify the analysis for easier design. Of late, there is a noticeable trend towards the development of ultimate load analysis in this field, and consequent upon the AASHO Road Test, attempts have been made also to apply the serviceability-performance criteria to rigid pavement design. They are, however, still in a developing stage.

Some of these methods take only traffic loads into account, ignoring such environmental factors as temperature changes in the pavement, which may substantially limit its load-carrying capacity. There are other factors, like impact, load repetitions, etc., the effects of which though understood qualitatively, are not yet conclusively established quantitatively. Nevertheless, for a rational design, their effects should be incorporated to the extent possible with the existing knowledge.

It is with the objective of simplifying this task and to promote the scientific design of rigid pavements that these guidelines have been drawn.

3. DESIGN PARAMETERS AND ASSESSMENT OF THEIR DESIGN VALUE

3.1. Traffic Parameters

3.1.1. **Design wheel load :** The design wheel load should be the maximum wheel load of the predominant heavy vehicle

likely to use the pavement in the normal course. In case of public highways, it will obviously be governed by the prevailing legal limits on the maximum laden weight of commercial vehicles. It is currently taken at 5100 kg.

In addition to the design wheel load, the maximum tyre inflation pressures for the vehicles should also be ascertained, so as to enable determination of tyre contact area through which the load is transmitted to the pavement. For most commercial highway vehicles, this ranges from about 5.3 to 7.3 kg/cm^2.

3.1.2. **Traffic intensity :** Passage of traffic results in repetitive loading of the pavement, thereby inducing fatigue effects in the concrete. These effects, while not of much consequence in case of low traffic intensities because of considerable time lag between successive passes, assume greater importance in case of heavily trafficked pavements, as the fatigue strength of concrete reduces with increase in the number of load repetitions it is required to sustain.

While a rigorous approach would require the assessment of total number of design load repetitions during intended design life of a pavement including due allowance for lighter and heavier loads through the use of appropriate equivalency factors, a more practical approach is to classify the pavements, for the purpose of making fatigue allowance, according to traffic intensity range expected.

Since traffic intensity is a growing phenomenon, the heaviest intensity will occur at the end of the design life of a pavement. However, it is generally considered adequate if the traffic is projected to a period of 20 years after construction, since in the initial stages the traffic intensity will be much less than that at the end of the design life.

For traffic prediction on main highways, the following correlation may be adopted :

$$T=P\,(1+r)^{n+20} \qquad \dots \quad (1)$$

with T = design traffic intensity in terms of number of commercial vehicles (laden weight $>$ 3 tonnes) per day,

P = traffic intensity at last traffic count,

r = annual rate of increase of traffic intensity, and

n = number of years since last traffic count and commissioning the new concrete pavement.

The traffic intensity P for assessment of design traffic intensity T should normally be a seven day average based on 24-hour counts, in accordance with IRC : 9-1972 : Traffic Census on Non-Urban Roads (First Revision). However, in exceptional cases, where such data are not available, an average of three day counts may be used as an approximation. Based on growth rate of traffic over the past few years, a value of 7.5 per cent is suggested for 'r' for rural roads for the time being, wherever actual data are not available.

In case of new highway links, where no traffic count data will be available, data from highways of similar classification and importance may be used to predict the design traffic intensity.

The pavement classification based on design traffic intensity, suggested for adoption for rigid pavement design, is given in Table 1.

TABLE 1. TRAFFIC CLASSIFICATION FOR RIGID PAVEMENT DESIGN

Traffic classification	Design Traffic Intensity : Vehicles (laden weight $>$3 tonnes) per day at the end of design life
A	0-15
B	15-45
C	45-150
D	150-450
E	450-1500
F	1500-4500
G	Above 4500 and all expressway

3.2. Environmental Parameters

3.2.1. **Temperature differential :** Temperature differential between top and bottom of concrete pavements is a function of solar radiation received by the pavement surface at the location, losses due to wind velocity, etc., and thermal diffusivity, of concrete, and is thus affected by geographical features of the pavement location.

As far as possible, values of actually anticipated temperature differentials at the location of the pavement should be

adopted for pavement design. For this purpose, guidance may be had from Table 2.

TABLE 2. RECOMMENDED TEMPERATURE DIFFERENTIALS IN CONCRETE ROADS

Zone	States	Temp. differential in °C in slabs of thickness				
		10 cm	15 cm	20 cm	25 cm	30 cm
I.	Punjab, U.P., Rajasthan, Gujarat, Haryana and North M.P., excluding hilly regions and coastal areas	10.2	12.5	13.1	14.3	15.8
II.	Bihar, West Bengal, Assam and Eastern Orissa, excluding hilly regions and coastal areas	14.4	15.6	16.4	16.6	16.8
III.	Maharashtra, Karnataka South M.P., Andhra Pradesh, Western Orissa and North Madras excluding hilly regions and coastal areas	14.75	17.3	19.0	20.3	21.0
IV.	Kerala and South Madras, excluding hilly regions and coastal areas	13.2	15.0	16.4	17.6	18.1
V.	Coastal areas bounded by hills	12.8	14.6	15.8	16.2	17.0
VI.	Coastal areas unbounded by hills	13.6	15.5	17.0	19.0	19.2

Note : The above mentioned table has been prepared on the basis of actual observations by Central Road Research Institute, New Delhi.

3.2.2. **Mean temperature cycles :** Mean temperature cycles-daily and annual of concrete pavements affect the maximum spacing of contraction and expansion joints in the pavement and design values for these factors would be required if it is desired to adopt the maximum safe spacing of expansion joints. However, these factors are dependent upon the geographical location of the pavement, and data thereon are generally not available readily. Somewhat conservative recommendations for maximum expansion joint spacing have, therefore, been framed as guidelines on the basis of actual temperature data collected at selected locations in different parts of the country.

3.3. Foundation Strength and Surface Characteristics

3.3.1. **Strength :** The foundation strength, in case of rigid pavements, is expressed in terms of modulus of subgrade reaction, K, which is defined as pressure per unit deflection of the foundation as determined by plate bearing tests. As the limiting design deflection for concrete pavements is taken at 1.25 mm, the K-value is determined from the pressure sustained at this deflection. As K-value is influenced by test plate diameter, the standard test is run with a 75 cm dia. plate, beyond which the effect of diameter has been found to be negligible. A frequency of one test per km per lane is recommended for assessment of K value, unless foundation changes with respect to subgrade soil type, of sub-base or the nature of formation (i.e. cut or fill) when additional tests may be conducted.

In case of homogeneous foundation, test values obtained with plates of smaller diameter may by converted to the standard 75 cm plate value by experimentally obtained correlations, e.g,

$$K_{75}=0.5\ K_{30} \qquad \ldots(2)$$

with K_{75} and K_{30} as the K values obtained on 75 cm and 30 cm dia. plates respectively. However, in case of layered construction, as in the case of sub-base, the tests with smaller plates give greater weightage to the stronger top layer, and direct conversion to 75 cm plate values by the above correlations somewhat overestimates the foundation strength, and such conversion must be regarded as very approximate only.

The subgrade soil strength, and consequently the strength of the foundation as a whole, is affected by its moisture content. The design strength obviously must be the minimum that will be available under the worst moisture conditions encountered. The ideal period for testing the foundation strength would thus be after the monsoons when the subgrade would have attained its highest moisture content.

In case the tests have to be conducted at some other time, especially during the dry part of the year, allowance for loss in strength due to increase in moisture must be made. For this purpose, an idea of the expected reduction in strength on saturation of the subgrade may be had from laboratory CBR tests on subgrade soil samples compacted at field density and moisture content and tested before and after saturation. An approximate idea of K value of a homogeneous soil subgrade may be had directly from its CBR value using Table 3. A more elaborate procedure involves correlation through consolidation tests on unsoaked and soaked samples.

TABLE 3 APPROXIMATE K VALUE CORRESPONDING TO CBR VALUES FOR HOMOGENEOUS SOIL SUBGRADES

CBR value (%)	2	3	4	5	7	10	20	50	100
K-valve (kg/cm³)	2.08	2.77	3.46	4.16	4.84	5.54	6.92	13.85	22.16

The recommendations of IRC : 15-1981 shall be followed and a K-value of less than 5.5 kg/cm³ tested on the subgrade shall not be permitted. In case of rocky subgrades with a K-value of 5.5 kg/cm³ and higher, the pavement may be laid directly thereon or after providing a levelling course, if required. In case of problematic subgrades such as clayey and expansive soils, etc., appropriate provisions shall be made for a blanket course in addition to the sub-base as per the relevant stipulations of IRC: 15-1981, reproduced in *Appendix I*.

3.3.2. **Foundation surface characteristics :** The foundation surface characteristics, viz., its smoothness or roughness, determine the extent of resistance to slab movement during expansion and contraction on account of foundation restraint, and affect joint spacings. The maximum safe spacing increases with increase in surface roughness of the foundation in case of expansion joints, and decreases in case of contraction joints.

For the purpose of determination of joint spacings, different types of foundations generally adopted may be classified into three categories, viz., very smooth, smooth and rough, according to their surface characteristics, as given in Table 4. As the foundations normally adopted in the country fall within the last two categories, only these two categories have been considered in formulating recommendations for expansion joint spacings.

TABLE 4. CLASSIFICATION OF DIFFERENT TYPES OF RIGID PAVEMENT FOUNDATIONS ACCORDING TO THEIR SURFACE CHARACTERISTICS

Surface roughness characteristics	Type of foundation
Very Smooth	Compacted sand and gravel. Smooth foundation covered with waterproof paper
Smooth	Compacted sand, gravel and clinker, stabilised soil. Rough foundation covered with waterproof paper
Rough	Water-bound macadam, soil-gravel mix, rolled lean concrete, lime-pozzolana concrete, etc.

3.4. Concrete Characteristics

3.4.1. **Design strength :** As stresses induced in concrete pavements are due either to bending or its prevention, their design is necessarily based on the flexural strength of concrete. For economical design, the design value adopted for flexural strength of pavement concrete should not be less than 40 kg/cm^2. This strength value, however, should not be confused with the mix design strength. The mix has to be so designed as to ensure the minimum structural strength requirements in the field with the desired confidence level. Thus if :

s = structural design value for concrete strength,

s = mix design value for concrete strength,

t = tolerance factor for the desired confidence level,

σ = expected standard deviation of field test samples, based on a knowledge of the type of control, viz. very good, good or fair, feasible at site.

then $s = s + t.\sigma$ .. (3)

so that to achieve the desired minimum structural strength s in the field, the mix design in the laboratory has to be made for somewhat higher strength, s making due allowance for the type and extent of quality control feasible in the field.

For pavement construction, the concrete mix should preferably be designed and controlled on the basis of flexural strength. If that is not possible, correlation between flexural and compressive strengths should be established on the basis of actual tests on additional samples made for the purpose at the time of mix design. Quality control can then be exercised on the basis of compressive strength, so long as the mix materials and proportions remain substantially unaltered. Even though it is customary to assume 280 kg/cm^2 as compressive strength corresponding to 40 kg/cm^2 flexural strength, such general assumptions should be avoided as far as possible in view of the variety of factors which influence the correlation between the two strengths.

For general guidance, the value of t and σ for concrete compressive strength value of 280 kg/cm^2 are given in Table 5 for different degrees of quality control. For design of cement concrete mix, IRC: 44-1972 Tentative Guidelines for Cement Concrete Mix Design for Road Pavements (for non-air entrained and continuously graded concrete) may be followed.

TABLE 5. CONCRETE MIX DESIGN STRENGTH FOR DIFFERENT DEGREES OF QUALITY CONTROL, FOR STRUCTURAL DESIGN VALUE OF 280 kg/sq. cm FOR CONCRETE COMPRESSIVE STRENGTH

Degree of quality control	Tolerance level	Tolerance factor, t	Coefficient of variation	Mix design strength kg./sq.cm.	Standard deviation kg/sq.cm.
Very good	1 in 15	1.50	7%	315	22
Good	1 in 15	1.50	10%	330	33
Fair	1 in 10	1.20	15%	350	52

Notes : *Very good quality control :* Control with weigh batching, use of graded aggregates, moisture determination of aggregates, etc. Rigid and constant supervision by the quality control team.

Good quality control : Control with weigh-batching, use of graded aggregates, moisture determination of aggregates, etc. Constant supervision by the quality control team.

Fair quality control : Control with volume-batching for aggregates. Occasional checking of aggregate moisture. Occasional supervision by the quality control team.

3.4.2. **Modulus of elasticity and Poisson's ratio:** The modulus of elasticity, E, and Poisson's ratio, μ of concrete are known to vary with concrete materials and strength. The elastic modulus increases with increase in strength, and Poisson's ratio decreases with increase in the modulus of elasticity. While it is desirable that the values of these parameters are ascertained experimentally for the concrete mix and materials actually to be used at the construction, this information may not always be available at the design stage. In such cases, it is suggested that for design purposes, the following values may be adopted for concrete in the 38-42 kg/cm^2 flexural strength range :

Modulus of elasticity of concrete, $E = 3 \times 10^5$ kg/cm^2

Poisson's ratio, μ $= 0.15$

3.4.3. **Coefficient of thermal expansion :** The coefficient of thermal expansion, α, of concretes of the same mix proportions varies with the type of aggregate, being in general high for siliceous aggregates, medium for igneous rocks and low for calcareous ones. However, for design purposes, a value $\alpha = 10 \times 10^{-6}/°C$ may be adopted in all cases.

4. DESIGN OF SLAB THICKNESS

4.1. Critical Stress Condition

Concrete pavements in service are subjected to stresses due to a variety of factors, acting simultaneously, the severest combination of which inducing the highest stress in the pavement will give the critical stress condition. The factors commonly considered for design of pavement thickness are traffic loads and temperature variations, as the two are additive. The effects of moisture changes and shrinkage, being generally opposed to those of temperature and of smaller magnitude, would ordinarily relieve the temperature effects to some extent, and are not normally considered critical to thickness design.

For purposes of analysis, three different regions are recognised in a pavement slab—corner, edge and interior—which react differently from one another to the effect of temperature differentials, as well as load application.

The concrete pavements undergo a daily cyclic change of temperature differentials, the top being hotter than the bottom during day, and cooler during night. The consequent tendency of the pavement slabs to warp upwards (top convex) during the day and downwards (top concave) during the night, and restraint offered to this warping tendency by self-weight of the pavement induces stressses in the pavement, referred to commonly as temperature stresses. These stresses are flexural in nature, being tensile at bottom during the day and at top during night. As the restraint offered to warping at any section of the slab would be a function of weight of the slab upto that section, it is obvious that corners have very little such restraint. The restraint is maximum in the slab interior, and somewhat less at the edge. Consequently the temperature stresses induced in the pavement are negligible in the corner region, and maximum at the interior.

Under the action of load application, maximum stress is induced in the corner region, as the corner is discontinuous in two directions. The edge being discontinuous in one direction only, has lower stress, while the least stress is induced in the interior where the slab is continuous in all directions. Furthermore, the corner tends to bend like a cantilever, giving tension at the top, interior like a beam giving tension at bottom. At edge, main bending is along the edge like a beam giving maximum tension at bottom.

The maximum combined tensile stresses in the three regions of the slab will thus be caused when effects of temperature differentials are such as to be additive to the load effects. This would occur during the day in case of interior and edge regions, at the

time of maximum temperature differential in the slab. In the corner region, the temperature stress is negligible, but the load stress is maximum at night when the slab corners have a tendency to lift up due to warping and lose partly the foundation support. Considering the total combined stress for the three regions, viz., corner, edge and interior, for which the load stress decreases in that order while the temperature stress increases, the critical stress condition is reached in the edge region where neither of the load and temperature stresses are the minimum. It is, therefore, felt that both the corner and the edge regions should be checked for total stresses and design of slab thickness based on the more critical condition of the two.

4.2. Calculation of Stresses

4.2.1. Edge stresses

(a) **Due to load :** The load stress in the critical edge region may be obtained as per Westergaard analysis and modified by Teller and Sutherland from the following correlation (metric units) :

$$\sigma l_e = 0.529 \frac{P}{h^2}(1+0.54\,\mu)\left(4\log_{10}\frac{l}{b} - \log_{10} b - 0.4048\right) \quad \ldots 4$$

with σl_e = load stress in the edge region, kg/cm²,

P = design wheel load, kg,

h = pavement slab thickness, cm,

μ = Poisson's ratio for concrete,

E = modulus of elasticity for concrete, kg/cm²,

K = reaction modulus of the pavement foundation, **kg/cm³**,

l = radius of relative stiffness, cm

$$= \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)K}} \quad \ldots(5)$$

b = radius of equiv. distribution of pressure

$$= a \text{ for } \frac{a}{h} > 1.724$$

$$= \sqrt{1.6a^2+h^2} - 0.675\,h \text{ for } \frac{a}{h} < 1.724 \quad \ldots(6)$$

and a = radius of load contact, cm, assumed circular.

The values of l and b can be ascertained directly from Tables 6 and 7. For ready reference, 4-figure log tables are included in *Appendix 2*

TABLE 6. RADIUS OF RELATIVE STIFFNESS, l, FOR DIFFERENT VALUES OF PAVEMENT SLAB THICKNESS, h, AND FOUNDATION REACTION MODULUS, K, FOR CONCRETE $E = 3.0 \times 10^5$ kg/cm²

h (cm)	Radius of relative stiffness l (cm) for different values of K (kg/cm)³				
	$K=6$	$K=8$	$K=10$	$K=15$	$K=30$
15	61.44	57.18	54.08	48.86	41.09
16	64.49	60.02	56.76	51.29	43.31
17	67.49	62.81	59.40	53.67	45.14
18	70.44	65.56	62.01	56.03	47.07
19	73.36	68.28	64.57	58.35	49.06
20	76.24	70.95	67.10	60.63	50.99
21	79.08	73.59	69.60	63.89	52.89
22	81.89	76.20	72.08	65.13	54.77
23	84.66	78.80	74.52	67.33	56.62
24	87.41	81.35	76.94	69.31	58.45
25	90.13	83.88	79.32	71.68	60.28

TABLE 7. RADIUS OF EQUIV. DISTRIBUTION OF PRESSURE SECTION, b, IN TERMS OF RADIUS OF CONTACT, a, AND SLAB THICKNESS, h

a/h	b/h	a/h	b/h
0.0	0.325	1.0	0.937
0.1	0.333	1.1	1.039
0.2	0.357	1.2	1.143
0.3	0.387	1.3	1.250
0.4	0.446	1.4	1.358
0.5	0.508	1.5	1.470
0.6	0.580	1.6	1.582
0.7	0.661	1.7	1.695
0.8	0.747	1.724	1.724
0.9	0.840	>1.724	a/h

(b) **Due to temperature :** The temperature stress at the critical edge region may be obtained as per Westergaard analysis, using Bradbury's coefficient, from the following correlation :

$$\sigma t_e = \frac{E \alpha \Delta t}{2} . C \qquad \dots (7)$$

with σt_e = temperature stress in the edge region,

Δt = maximum temperature differential during day between top and bottom of the slab,

α = coefficient of thermal expansion of concrete,

C = Bradbury's coefficient, which can be ascertained directly from Bradbury's chart against values of L/l and W/l,

L = slab length, or spacing between consecutive contraction joints,

W = slab width, and

l = radius of relative stiffness.

Values of the coefficient, C, based on the curves given in Bradbury's chart, are given in Table 8.

TABLE 8. VALUES OF CO-EFFICIENT 'C' BASED ON BRADBURY'S CHART

L/l or W/l	C
1	0.000
2	0.040
3	0.175
4	0.440
5	0.720
6	0.920
7	1.030
8	1.075
9	1.080
10	1.075
11	1.050
12 and above	1.000

4.2.2. **Corner stresses:** The load stress in the corner region may be obtained as per Westergaard's analysis, as modified by Kelley, from the following correlation :

$$\sigma/c = \frac{3P}{h^2}\left[1 - \frac{(a\sqrt{2})^{1.2}}{1}\right]$$

with σl_c = load stress in the corner region, other notations remaining the same as in the case of the edge load stress formula.

The temperature stress in the corner region is negligible as the corners are relatively free to warp, and may be ignored.

4.3. Design Charts

Figs. 1 and 2 give ready-to-use design charts for calculation of load stresses in the edge and corner regions of rigid pavement

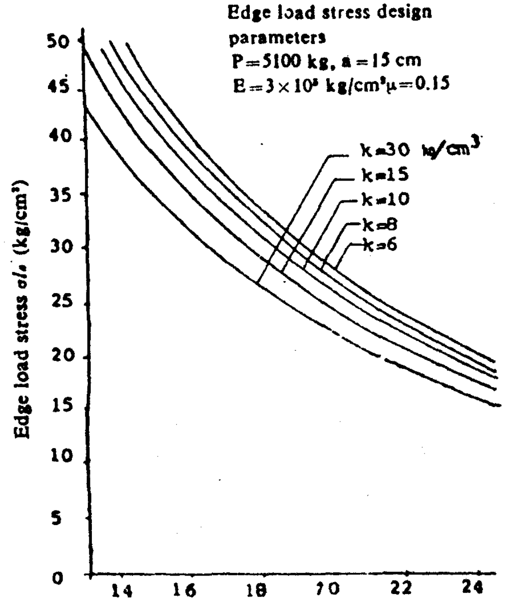

Fig. 1. Design chart for calculation of edge load stress

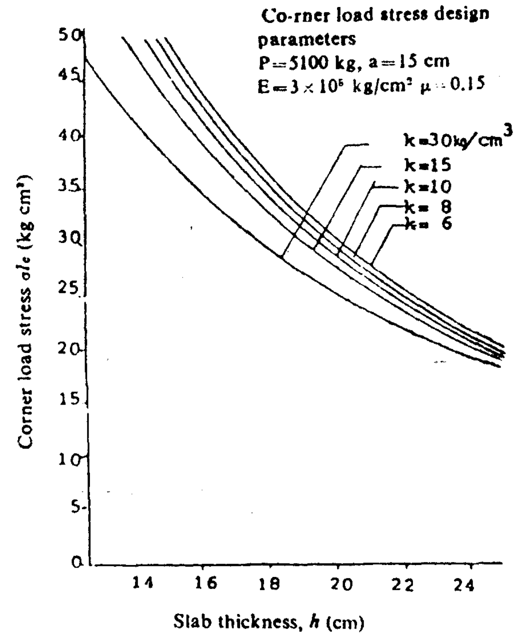

Fig. 2. Design chart for calculation of corner load stress

slabs for the design wheel load of 5100 kg. Fig. 3 gives a design chart for calculation of temperature stresses in the edge region.

4.4. Recommended Design Procedure

Step 1 : Stipulate design values for the various parameters.

Step 2 : Decide joint spacing and land-widths (vide para 5.1).

Step 3 : Select tentative design thickness of pavement slab.

Step 4 : Ascertain maximum temperature stress for the critical edge region from Equation (7) or Fig. 3.

Step 5 : Calculate the residual available strength of concrete for supporting traffic loads.

Step 6 : Ascertain edge load stress from Equation (4) or Fig. 1, and calculate factor of safety thereon.

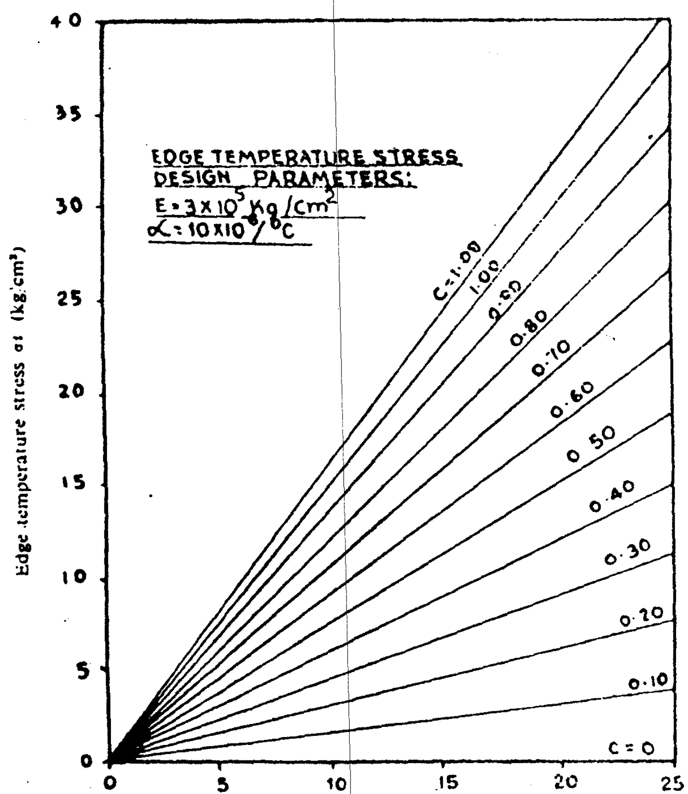

Chart for determination of coefficient C

L/l or W/l	C	L/l or W/l	C
1	0.000	7	1.030
2	0.040	8	1.077
3	0.175	9	1.080
4	0.440	10	1.075
5	0.720	11	1.050
6	0.920	12	1.000

Fig. 3. Design chart for calculation of edge temperature stress

Step 7 : In case the available factor of safety is less than or far in excess of 1, adjust the tentative slab thickness and repeat steps 3 to 6 till the factor of safety is 1 or slightly more. Denote the corresponding slab thickness as h_s.

Step 8 : Check for adequacy of thickness in the corner region by ascertaining corner load stress from Equation (8) of Fig. 1 and readjust the thickness h_s, if inadequate;

Step 9 : Adjust h_s for traffic intensity. The adjusted design thickness, h, may be obtined from

$$h = h_s + h_t \quad ...(9)$$

The values of h_t may be taken from Table 9.

TABLE 9. RIGID PAVEMENT THICKNESS ADJUSTMENT FACTOR, h_t, FOR TRAFFIC INTENSITY

Traffic classification	A	B	C	D	E	F	G
h_t (cm)	−5	−5	−2	−2	+0	+0	+2

Note : See Table 1 for Traffic Classification.

An illustrative example of design of slab thickness is given in *Appendix 3*.

5. DESIGN OF JOINTS

5.1. Spacing and Layout

The recommendations of the IRC : 15-1981, para 8 and Supplementary Notes para N. 2 "Arrangement of Joints", may be followed with regard to joint layout and contraction joint spacings

As rrgards expansion joints. it is possible to adopt much greater spacings than recommended in the Code. Based on a recent study by the Central Road Research Institute which gives the maximum spacing of expansion joints that can be adopted for concrete pavements in India, from a consideration of daily and annual temperature variations in the pavements in different parts of the country, degree of foundation roughness as well as the season of construction, the maximum recommended spacings of expansion joints are given in Table 10.

TABLE 10. RECOMMENDED SPACING OF JOINTS IN RIGID PAVEMENTS FOR HIGHWAYS

(a) Expansion Joint Spacings (based on CRRI Study)
(for 25 mm wide expansion joints)

Period of construction	Degree of foundation roughnes	Maximum expansion joint spacing (m) Slab thickness (cm) 15	20	25
Winter (Oct-March)	Smooth	50	50	60
	Rough	140	140	140
Summer (April-Sept)	Smooth	90	90	120
	Rough	140	140	140

Notes : 1. See Table 4 for classification of different types of foundation layers according to degree of roughness.

(b) Contraction Joints Spacings (based on IRC : 15-1981)

Slab thickness (cm)	Maximum contraction joint spacing (m)	Weight of reinforcement in welded fabric (for reinforced pavements only) (kg m²)
Unreinforced Slabs		
10	4.5	—
15	4.5	—
20	4.5	—
Reinforced Slabs		
10	7.5	2.2
15	13.0	2.7
20	14.0	3.8

5.2. **Load Transfer at Transverse Joints**

5.2.1. Load transfer to relieve part of the load stresses in edge and corner regions of pavement slab at transverse joints is provided by means of mild steel dowel bars. For general provi-

sions in respect of dowel bars, stipulations laid down in IRC : 15-1981, Supplimentary Notes para N. 4.2 "Dowel Bars", may be followed. The method of design of dowel bars as per Bradbury's analysis is recommended.

5.2.2. **Design of dowel bars :** The dowel bar system may be designed on the basis of Bradbury's analysis which gives the following formulae for load transfer capacity of a single dowel bar in shear, in bending and in bearing on concrete :

$$\overline{P} = 0.785\, d^2 f_s' \qquad \text{(shear)} \qquad \ldots \; (10)$$

$$\overline{P} = \frac{2\, d^3 f_c}{r + 8.8z} \qquad \text{(bending in the bar)} \qquad \ldots \; (11)$$

$$\overline{P} = \frac{f_c r^2 d}{12.5\,(r + 1.5z)} \qquad \text{(bearing on the concrete)} \ldots \; (12)$$

with $\overline{P}$ = load transfer capacity of a single dowel bar,
d = diameter of dowel bar,
r = length of embedment of dowel bar,
z = joint width,
f_s' = permissible shear stress in dowel bar,
f_s = permissible flexural stress in dowel bar, and
f_c = permissible bearing stress in concrete.

For balanced design, for equal capacity in bending and bearing, the length of embedment of dowel is first obtained by equating $\overline{P}$ values from equations (11) and (12) as follows, for the assumed joint width z and dowel diameter d :

$$r = 5d\left[\frac{f_s}{f_c} \quad \frac{r + 1.5z}{r + 8.8z}\right]^{\frac{1}{2}} \qquad \ldots \; (13)$$

Knowing z, d and r, the load transfer capacity of a single dowel is determined from the equations (11) and (12) given above.

To calculate the spacing of dowel bars, the required capacity frctor, n, is first determined from :

$$n = \frac{\text{load transfer capacity required from the dowel system}}{\text{load transfer capacity of a single dowel bar}} \qquad \ldots \; (14)$$

The distance on either side of the load position upto which the dowel bars are effective in load transfer is taken as 1.8 l, where l is the radius of relative stiffness (Equation 6).

Assuming linear variation of the capacity factor for a single dowel bar from 1.0 under the load to 0 at a distance of 1.8 l therefrom, the capacity factors for the dowel system are calculated for different spacings. The spacing which conforms to the required capacity factor, n, is selected for adoption. An example of the design of dowel bars is given in *Appendix 4*.

5.2.3. Dowel bars are not satisfactory for slabs of small thickness, and shall not be provided for slabs less than 15 cm thick.

TABLE 11. DESIGN DETAILS OF DOWEL BARS FOR RIGID HIGHWAY PAVEMENTS

Design loading	Slab thickness (cm)	Dowel bar details		
		Diameter (mm)	Length (mm)	Spacing (mm)
5100 kg	15	25	500	200
	20	25	500	250
	25	25	500	300

Note : The recommended details are based on the following values of different design parameters; f_s = 1400 kg/cm²; f_f = 100 kg/cm²; E_c = 3.0×10⁵ kg/cm² μ = 0.15; K_s = 8.3 kg/cm³; max, joint width = 20 mm; design load transfer = 40 per cent.

5.2.4. Typical dowel bar designs for use in 20 mm wide expansion joints for highway pavements, for 40 per cent load transfer are given in Table 11. In case of dummy contraction joints, aggregate interlock is relied upon to provide load transfer to some extent, and dowel bars are not provided, ordinarily. Dowel bars shall, however, be provided in case of full depth construction joints.

5.3. Tie Bars for Longitudinal Joints

5.3.1. In case opening of longitudinal joints is anticipated in service, for example, in case of heavy traffic, sidelong ground, expansive subgrades, etc., tie bars may be designed in accordance with the recommendations of IRC: 15-1981, Supplimentary Note, para N. 5 Tie Bars For the sake of convenience of the designers the design procedure recommended in IRC: 15-1981 is given herein.

5.3.2. **Design of the bars :** The area of steel required per m length of joint may be computed by using the following formula :

$$A_s = \frac{bfW}{S} \quad \ldots (15)$$

in which

A_s = area of steel in cm² required per m length of joint,

b = distance between the joint in question and the nearest free joint or edge in m,

f = coefficient of friction between pavement and the subgrade (usually taken as 1.5),

W = weight of slab in kg/m², and

S = allowable working stress of steel in kg/cm².

The length of any tie bar should be at least twice that required to develop a bond strength equal to the working stress of the steel. Expressed as a formula, this becomes:

$$L = \frac{2SA}{B^* P} \quad \ldots (16)$$

in which

L = length of tie bar (cm)

S = allowable working stress in steel (kg/cm²)

A = cross-sectional area of one tie bar (cm²)

P = perimeter of tie bar (cm), and

B^* = permissible bond stress in (i) deformed tie bars—24.6 kg/cm², (ii) Plain tie bars—17.5 kg/cm²

5.3.3. To permit warping at the joint the maximum diameter in case of tie bars may be limited to 20 mm, and to avoid concentration of tensile forces they should not be spaced more than 75 cm apart. The calculated length, L, may be increased by 5-8 cm to account for any inaccuracy in placement during construction. An example of design of tie bar is given in *Appendix 4*.

5.3.4. Typical tie bar details for use at central longitudinal joint in double-lane rigid pavements with a lane width of 3.50 m are given in Table 12.

TABLE 12. DETAILS OF TIE BARS FOR CENTRAL LONGITUDINAL JOINT OF TWO-LANE RIGID HIGHWAY PAVEMENTS

Slab thickness (cm)	Tie bar details			
	Diameter (mm)	Maximum spacing (cm)	Minimum length (cm)	
			Plain bars	Deformed bars
15	8	38	40	30
	10	60	45	35
20	10	45	45	35
	12	64	55	40
25	10	30	45	35
	12	45	55	40
	14	62	65	46

Note : The recommended details are based on the following values of different design parameters :

f_s = 1400 kg/cm², B^* = 17.5 kg/cm² for plain bars and 24.6 kg/cm² for deformed bars, f = 1.5, W = 24 kg/m²/cm of slab thickness.

6. DESIGN OF REINFORCEMENT

6.1. Reinforcement, when provided in concrete pavements, is intended for holding the fractured faces at the cracks tightly closed together, so as to prevent deterioration of the cracks and to maintain aggregate interlock thereat for load transfer. It does not increase the flexural strength of unbroken slab when used in quantities which are considered economical. Where the slabs are provided adequately with joints to control cracking, such reinforcement has no function.

6.2. Reinforcement in concrete slabs is designed to counteract the tensile stresses caused by shrinkage and contraction due to temperature or moisture changes. The maximum tension in the steel across the crack equals the force required to overcome friction between the pavement and its foundation, from the crack to the nearest joint or free edge. This force is the greatest when the crack occurs at the middle of the slab. Reinforcement is designed for this critical location. However,

for practical reasons, reinforcement is kept uniform throughout the length, for short slabs.

The amount of longitudinal and transverse steel required per m width or length of slab is computed by the following formula:

$$A = \frac{LfW}{2S} \quad \ldots (17)$$

in which

A = area of steel in cm² required per m width or length of slab,

L = distance in m between free transverse joints (for longitudinal steel) or free longitudinal joints (for transverse steel).

f = coefficient of friction between pavement and subgrade (usually taken as 1.5),

W = weight of slab in kg/m², and

S = allowable working stress in steel in kg/cm² (usually taken as 50 to 60 per cent of the minimum yield stress of steel).

6.3. Since reinforcement in the concrete slabs is not intended to contribute towards its flexural strength, its position within the slab is not important except that it should be adequately protected from corrosion. Since cracks starting with higher tensile stress at the top surface are more critical when they tend to open, the general preference is for the placing of reinforcement about 50 mm below the surface. Reinforcement is often continued across dummy groove joints to serve the same purpose as tie bars, but at all full depth joints it is kept at least 50 mm away from the face of the joint or edge.

Appendix 1

EXTRACTS FROM IRC : 15-1981 "STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR CONSTRUCTION OF CONCRETE ROADS" (Second Revision)

6. PREPARATION OF SUBGRADE AND SUB-BASE

6.1. **General**

The subgrade or sub-base for laying of paving concrete slabs shall comply with the following requirements :

(1) that no soft sports are present in the subgrade or sub-base ;

(2) that the uniformly compacted subgrade or sub-base extends at least 300 mm on either side of the width to be concreted ;

(3) that the subgrade is properly drained ;

(4) that the minimum modulus of subgrade reaction obtained with a plate bearing test shall be 5.5 kg/cm^3

The manner of achieving these requirements shall be determined depending upon the type of subgrade or sub-base on which concrete is to be laid, and the following requirements in respect of the various types shall be satisfactorily met. The construction procedures for subgrade and sub-bases should follow relevant IRC specifications, and quality control should be exercised as laid down in IRC : SP-11.

6.2. **Subgrade**

6.2.1. Where the type of soil in the formation of the road is of a quality to ensure the requirements in the aforementioned para, no intermediate sub-base need be used. The top 150 mm layer of the formation shall be compacted at or slightly above the optimum moisture content to the exact profile shown in the drawing. It shall be checked for trueness by means of a scratch template (see IRC : 43-1972 for details) resting on the side forms and set to the exact profile of the base course. The template shall be drawn along the forms at right angles to the centre line of the road. Unevenness of the surface as indicated by the scratch points shall not exceed 12 mm in 3 m. The surface irregularities in excess of this shall be properly rectified and the surface rolled or tamped until it is smooth and firm. The subgrade shall be prepared and checked at least two days in advance of concreting.

6.2.2. Where no sub-base is considered necessary and concrete is laid directly on the prepared subgrade, the subgrade, shall be in moist condition at the time the concrete is placed. If necessary, it should be saturated with water not less than 6 hours nor more thnn 20 hours in advance of placing concrete. If it becomes dry prior to the actual placing of the concrete, it shall be sprinkled with water taking care to see that no pools of water or soft patches are formed on the surface. It is desirable to lay a layer of water-proof paper whenever concrete is laid directly over soil subgrade. Where such a layer of waterpoorf paper is proposed to be placed between concrete and the subgrade, the moistening of the subgrade prior to placing of the concrete shall be omitted.

6.3. **Sub-base**

6.3.1. Where the subgrade is of a type not satisfying the requirements of para 6.1., a Sub-base layer should be provided before laying the concrete. The sub-base may be of granular material, stabilised soil or semi-rigid material as listed below :

(a) *Granular material*

(i) one layer flat brick soling having joints filled with sand under one layer of water bound macadam conforming to IRC : 19-1977.

(ii) Two layers of water bound macadam.

(iii) Well-graded granular materials like natural gravel, crushed slag, crushed concrete, brick metal, laterite, kankar, etc. conforming to IRC : 63-1975.

(iv) Well-graded soil aggregate mixtures conforming to IRC : 63-1976.

(b) *Stabilised soil*

Local soil or moorum stabilised with lime or lime-fly ash or cement, as appropriate to give a minimum soaked CBR of 50 after 7 days curing. For guidance as regards design of mixes with lime or cement, reference may be made to IRC : 51 and 50 respectively

(c) *Semi-rigid material*

(i) Lime-burnt clay puzzolana concrete. The lime-pazzolana mixture should conform to L.P. 40 or L.P. 20 of IS : 4098-1967. The 28 day compressive strength of the concrete should be in the range of 40-60 km/cm².

(ii) Lime-fly ash concrete conforming to IRC : 60-1976.

(iii) Lean cement concrete or lean cement-fly ash concrete conforming to IRC : 74-1979.

6.3.2. Thickness of sub-base should be 15 cm when the material used is of any of the types listed in paras 6.3.1. (a) and (b). This may, however, be reduced to 10 cm for semi-rigid materials in para 6.3.1. (c). The sub-base should be constructed in accordance with the respective specification and the surface finished to the required lines, levels and cross-section.

6.3.3. Where the subgrade consists of heavy clay (L.L. >50) such as black cotton soil, the sub-base should be laid over a 15 cm thick blanket course consisting of non-plastic granular material like local sand, gravel, kankar, etc. or local soil stabilised with lime.

6.3.4. In water-logged areas and where the sub-grade soil is impregnated with deleterious salts such as sodium sulphate etc. in injurious amounts** a capillary cut-off should be provided before constructing the sub-base, vide details given in para 6.4.

6.3.5. The sub-base or blanket course, as the case may be, shall be laid over a properly compacted subgrade to give uniform support.

**Sulphate concentration (as sulphur trioxide) more than 0.2% in subgrade soil and more than 0.3% in ground water.

6.3.6. The sub-base shall be in moist condition at the time the concrete is placed. There shall, however, be no pools of water or soft patches formed on the sub-base surface. In case where a sand layer is placed between the sub-base and pavement concrete, a layer of water-proof paper shall be laid over the sand layer. No moistening of the sub-base shall be done in this case.

6.4. **Capillary Cut-off**

6.4.1. As a result of migration of water by capillarity from the high water table, the soil immediately below the pavement gets more and more wet and this leads to gradual loss in its bearing value besides unequal support. Several measures such as depressing the sub-soil water table by drainage measures, raising of the embankment and provision of a capillary cut-off are available for mitigating this deficiency and should be investigated for arriving at the optimum solution. However, where deleterious salts in excess of the safe limits are present in the subgrade soil, a capillary cut-off should be provided in addition to other measures.

6.4.2. The capillary cut-off may be a layer of coarse or fine sand, graded gravel, bituminised material, or an impermeable membrane. Layer thicknesses recommended for different situations are given in Table 4.

TABLE 4. RECOMMENDED THICKNESS OF SAND/GRADED GRAVEL LAYER FOR CAPILLARY CUT-OFF

Sl. No.	Situation	Thickness of layer cm		
		Coarse sand (mean dia 0.64 mm)	Fine sand (mean dia 0.18 mm)	Graded gravel (40 mm and down without fines)
1.	Water table at the same level as the subgrade surface	15	45	15
2.	Embankment about 0.6—1.0 m high	12	35	11
3.	Embankment about 0.6—1.0 m high but with the top 15 cm subgrade layer being of sandy soil having PI of 5 or less and sand content not less than 50 per cent	10	30	8

6.4.3. Cut-off with bituminised or other materials may be provided in any of the following ways :

(i) **Bituminous impregnation using primer treatment**
50 per cent straight run bitumen (80-100) with 50 per cent high speed diesel oil or its equivalent in two applications of 1 kg sq. m. each, allowing the first application to penetrate before applying the second one. These applications should be given under the roadbed as well as onto the sides.

(ii) **Heavy-duty tar felt**
Enveloping sides and bottom of the roadbed with heavy-duty tar felt.

(iii) **Polyethylene envelope**
Enveloping sides and bottom of the roadbed with polyethylene sheets of at least 400 gauge.

(iv) **Bituminouse stabilise soil**
Providing bituminous stabilised soil in a thickness of at least 4 cm.

Note : Experience on the successful use of the above capillary cut-offs is, however, limited.

6.4.4. For more details about mitigating the adverse effects of high water table, reference may be made to IRC : 34-1970 "Recommendations for Road Construction in Waterlogged Areas".

6.5. Frost Affected Areas

6.5.1. In frost affected areas, the sub-base may consist of any of the specifications given in 6. 3.1. (a), (b) or (c) excepting that in the case of the items 6.3 1. (b) and 6.3.1. (c), the compressive strength of the stabilised or semi-rigid material cured in wet condition shall be at least 35kg/cm^2 at 7 days. For moderate conditions, such as those prevailing in areas at an altitude of 3,000 m and below, the thickness of frost affected depth will be about 45 cm. For protection against frost, the balance between the frost depth (45) cm and total pavement thickness should be made up with non-frost susceptible material.

6.5.2. For extreme conditions, such as those prevailing in areas above an altitude of 3,000 m, the foundation may be designed individually for every location after determining the depth of frost.

6.5.3. The suggested criteria for the selection of non-frost susceptible materials are as follows :

(i) Graded gravel : Not more than 8 per cent passing 75 micron sieve Plasticity index not more than 6. Liquid limit not more than 25.

(ii) Poorly graded sands, generally 100 per cent passing 4.75 mm sieve :
Max. 10 per cent passing 75 micron sieve
Max. 5 per cent passing 50 micron sive

(iii) Fine uniform sand, generally 100 per cent passing 425 micron sieve :
Max. 18 per cent passing 75 micron sieve
Max. 8 per cent passing 50 micron sieve

Appendix 2

LOGARITHMS

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
10	0000	0043	0086	0128	0170	0212	0253	0294	0334	0374	4	8	12	17	21	25	29	33	37
11	0414	0453	0492	0531	0569	0607	0645	0682	0719	0755	4	8	11	15	19	23	26	30	34
12	0792	0828	0864	0899	0934	0969	1004	1038	1072	1106	3	7	10	14	17	21	24	28	31
13	1139	1173	1206	1239	1271	1303	1335	1367	1399	1430	3	6	10	13	16	19	23	26	29
14	1461	1492	1523	1553	1584	1614	1644	1673	1703	1732	3	6	9	12	15	18	21	24	27
15	1761	1790	1818	1847	1875	1903	1931	1959	1987	2014	3	6	8	11	14	17	20	22	25
16	2041	2068	2095	2122	2148	2175	2201	2227	2253	2279	3	5	8	11	13	16	18	21	24
17	2304	2330	2355	2380	2405	2430	2455	2480	2504	2529	2	5	7	10	12	15	17	20	22
18	2553	2577	2601	2625	2648	2672	2695	2718	2742	2765	2	5	7	9	12	14	16	19	21
19	2788	2810	2833	2856	2878	2900	2923	2945	2967	2989	2	4	7	9	11	13	16	18	20
20	3010	3032	3054	3075	3096	3118	3139	3160	3181	3201	2	4	6	8	11	13	15	17	19
21	3222	3243	3263	3284	3304	3324	3345	3365	3385	3404	2	4	6	8	10	12	14	16	18
22	3424	3444	3464	3483	3502	3522	3541	3560	3579	3598	2	4	6	8	10	12	14	15	17
23	3617	3636	3655	3674	3692	3711	3729	3747	3766	3784	2	4	6	7	9	11	13	15	17
24	3802	3820	3838	3856	3874	3892	3909	3927	3945	3962	2	4	5	7	9	11	12	14	16
25	3979	3997	4014	4031	4048	4065	4082	4099	4116	4133	2	3	5	7	9	10	12	14	15
26	4150	4166	4183	4200	4216	4232	4249	4265	4281	4298	2	3	5	7	8	10	11	13	15
27	4314	4330	4346	4362	4378	4393	4409	4425	4440	4456	2	3	5	6	8	9	11	13	14
28	4472	4487	4502	4518	4533	4548	4564	4579	4594	4609	2	3	5	6	8	9	11	12	14
29	4624	4639	4654	4669	4683	4698	4713	4728	4742	4757	1	3	4	6	7	9	10	12	13

(Contd).

Appendix 2 (Contd.)

LOGARITHMS

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
30	4771	4786	4800	4814	4829	4843	4857	4871	4886	4900	1	3	4	6	7	9	10	11	13
31	4914	4928	4942	4955	4969	4983	4997	5011	5024	5038	1	3	4	6	7	8	10	11	12
32	5051	5065	5079	5092	5105	5119	5132	5145	5159	5172	1	3	4	5	7	8	9	11	12
33	5185	5198	5211	5224	5237	5250	5263	5276	5289	5302	1	3	4	5	6	8	9	10	12
34	5315	5328	5340	5353	5366	5378	5391	5404	5416	5428	1	3	4	5	6	8	9	10	11
35	5441	5453	5465	5478	5490	5502	5514	5527	5539	5551	1	2	4	5	6	7	9	10	11
36	5563	5575	5587	5599	5611	5623	5635	5647	5658	5670	1	2	4	5	6	7	8	10	11
37	5682	5694	5705	5717	5729	5740	5752	5763	5775	5786	1	2	3	5	6	7	8	9	10
38	5798	5809	5821	5832	5843	5855	5866	5877	5888	5899	1	2	3	5	6	7	8	9	10
39	5911	5922	5933	5944	5955	5966	5977	5988	5999	6010	1	2	3	4	5	7	8	9	10
40	6021	6031	6042	6053	6064	6075	6085	6096	6107	6117	1	2	3	4	5	6	8	9	10
41	6128	6138	6149	6160	6170	6180	6191	6201	6212	6222	1	2	3	4	5	6	7	8	9
42	6232	6243	6253	6263	6274	6284	6294	6304	6314	6325	1	2	3	4	5	6	7	8	9
43	6335	6345	6355	6365	6375	6385	6395	6405	6415	6425	1	2	3	4	5	6	7	8	9
44	6435	6444	6454	6464	6474	6484	6493	6503	6513	6522	1	2	3	4	5	6	7	8	9
45	6532	6542	6551	6561	6571	6580	6590	6599	6609	6618	1	2	3	4	5	6	7	8	9
46	6628	6637	6646	6656	6665	6675	6684	6693	6702	6712	1	2	3	4	5	6	7	7	8
47	6721	6730	6739	6749	6758	6767	6776	6785	6794	6803	1	2	3	4	5	5	6	7	8
48	6812	6821	6830	6839	6848	6857	6866	6875	6884	6893	1	2	3	4	4	5	6	7	8
49	6902	6911	6920	6928	6937	6946	6955	6964	6972	6981	1	2	3	4	4	5	6	7	8
50	6990	6998	7007	7016	7024	7033	7042	7050	7059	7067	1	2	3	3	4	5	6	7	8
51	7076	7084	7093	7101	7110	7118	7126	7135	7143	7152	1	2	3	3	4	5	6	7	8

(Contd.)

Appendix 2 (Contd.)

LOGARITHMS

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
52	7160	7168	7177	7185	7193	7202	7210	7218	7226	7235	1	2	2	3	4	5	6	7	7
53	7243	7251	7259	7267	7275	7284	7292	7300	7308	7316	1	2	2	3	4	5	6	6	7
54	7324	7332	7340	7348	7356	7364	7372	7380	7388	7396	1	2	2	3	4	5	6	6	7
55	7404	7412	7419	7427	7435	7443	7451	7459	7466	7474	1	2	2	3	4	5	5	6	7
56	7482	7490	7497	7505	7513	7520	7528	7536	7543	7551	1	2	2	3	4	5	5	6	7
57	7559	7566	7574	7582	7589	7597	7604	7612	7619	7627	1	2	2	3	4	5	5	6	7
58	7634	7642	7649	7657	7664	7672	7679	7686	7694	7701	1	1	2	3	4	4	5	6	7
59	7709	7716	7723	7731	7738	7745	7752	7760	7767	7774	1	1	2	3	4	4	5	6	7
60	7782	7789	7796	7803	7810	7818	7825	7832	7839	7846	1	1	2	3	4	4	5	6	6
61	7853	7860	7868	7875	7882	7889	7896	7903	7910	7917	1	1	2	3	4	4	5	6	6
62	7924	7931	7938	7945	7952	7959	7966	7973	7980	7987	1	1	2	3	3	4	5	6	6
63	7993	8000	8007	8014	8021	8028	8035	8041	8048	8055	1	1	2	3	3	4	5	5	6
64	8062	8069	8075	8082	8084	8096	8102	8109	8116	8122	1	1	2	3	3	4	5	5	6
65	8129	8136	8142	8149	8156	8162	8169	8176	8182	8189	1	1	2	3	3	4	5	5	6
66	8195	8202	8209	8215	8222	8228	8235	8241	8248	8254	1	1	2	3	3	4	5	5	6
67	8261	8267	8274	8280	8287	8293	8299	8306	8312	8319	1	1	2	3	3	4	5	5	6
68	8325	8331	8338	8344	8351	8357	8363	8370	8376	8382	1	1	2	3	3	4	4	5	6
69	8388	8395	8401	8407	8414	8420	8426	8432	8439	8445	1	1	2	2	3	4	4	5	6
70	8451	8457	8463	8470	8476	8482	8488	8494	8500	8506	1	1	2	2	3	4	4	5	6
71	8513	8519	8525	8531	8537	8543	8549	8555	8561	8567	1	1	2	2	3	4	4	5	5
72	8573	8579	8585	8591	8597	8603	8609	8615	8621	8627	1	1	2	2	3	4	4	5	5
73	8633	8639	8645	8651	8657	8663	8669	8675	8681	8686	1	1	2	2	3	4	4	5	5
74	8692	8798	8704	8710	8716	8722	8727	8733	8739	8745	1	1	2	2	3	4	4	5	5
75	8751	8756	8762	8768	8774	8779	8785	8791	8797	8802	1	1	2	2	3	3	4	5	5
76	8808	8814	8820	8825	8831	8837	8842	8848	8854	8859	1	1	2	2	3	3	4	5	5

(Contd.)

Appendix 2 (Contd.)

LOGARITHMS

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
77	8865	8871	8876	8882	8887	8893	8899	8904	8910	8915	1	1	2	2	3	3	4	4	5
78	8921	8927	8932	8938	8943	8949	8954	8960	8965	8971	1	1	2	2	3	3	4	4	5
79	8976	8982	8987	8993	8998	9004	9009	9015	9020	9025	1	1	2	2	3	3	4	4	5
80	9031	9036	9042	9047	9053	9058	9063	9069	9074	9079	1	1	2	2	3	3	4	4	5
81	9085	9090	9096	9101	9106	9112	9117	9122	9128	9133	1	1	2	2	3	3	4	4	5
82	9138	9143	9149	9154	9159	9165	9170	9175	9180	9186	1	1	2	2	3	3	4	4	5
83	9191	9196	9201	9206	9212	9217	9222	9227	9232	9238	1	1	2	2	3	3	4	4	5
84	9243	9248	9253	9258	9263	9269	9274	9279	9284	9289	1	1	2	2	3	3	4	4	5
85	9294	9299	9304	9309	9315	9320	9325	9330	9335	9340	0	1	2	2	3	3	4	4	5
86	9345	9350	9355	9360	9365	9370	9375	9380	9385	9390	0	1	2	2	3	3	4	4	5
87	9395	9400	9405	9410	9415	9420	9425	9430	9435	9440	0	1	1	2	2	3	3	4	4
88	9445	9450	9455	9460	9465	9469	9474	9479	9484	9489	0	1	1	2	2	3	3	4	4
89	9494	9499	9504	9509	9513	9518	9523	9528	9533	9538	0	1	1	2	2	3	3	4	4
90	9542	9547	9552	9557	9562	9566	9571	9576	9581	9586	0	1	1	2	2	3	3	4	4
91	9590	9595	9600	9605	9609	9614	9619	9624	9628	9633	0	1	1	2	2	3	3	4	4
92	9638	9643	9647	9652	9657	9661	9666	9671	9675	9680	0	1	1	2	2	3	3	4	4
93	9685	9689	9694	9699	9703	9708	9713	9717	9722	9727	0	1	1	2	2	3	3	4	4
94	9731	9736	9741	9745	9750	9754	9759	9763	9768	9773	0	1	1	2	2	3	3	4	4
95	9777	9782	9786	9791	9795	9800	9805	9809	9814	9818	0	1	1	2	2	3	3	4	4
96	9823	9827	9832	9836	9841	9845	9850	9854	9859	9863	0	1	1	2	2	3	3	4	4
97	9868	9872	9877	9881	9886	9890	9894	9899	9903	9908	0	1	1	2	2	3	3	4	4
98	9912	9917	9921	9926	9930	9934	9939	9943	9948	9952	0	1	1	2	2	3	3	4	4
99	9956	9961	9965	9969	9974	9978	9983	9987	9991	9996	0	1	1	2	2	3	3	4	4

ANTILOGARITHMS

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
.00	1000	1002	1005	1007	1009	1012	1014	1016	1019	1021	0	0	1	1	1	1	2	2	2
.01	1023	1026	1028	1030	1033	1035	1038	1040	1042	1045	0	0	1	1	1	1	2	2	2
.02	1047	1050	1052	1054	1057	1059	1062	1064	1067	1069	0	0	1	1	1	1	2	2	2
.03	1072	1074	1076	1079	1081	1084	1086	1089	1091	1094	0	0	1	1	1	1	2	2	2
.04	1096	1099	1102	1104	1107	1109	1112	1114	1117	1119	0	1	1	1	1	2	2	2	2
.05	1122	1125	1127	1130	1132	1135	1138	1140	1143	1146	0	1	1	1	1	2	2	2	2
.06	1148	1151	1153	1156	1159	1161	1164	1167	1169	1172	0	1	1	1	1	2	2	2	2
.07	1175	1178	1180	1183	1186	1189	1191	1194	1197	1199	0	1	1	1	1	2	2	2	2
.08	1202	1205	1208	1211	1213	1216	1219	1222	1225	1227	0	1	1	1	1	2	2	2	3
.09	1230	1233	1236	1239	1242	1245	1247	1250	1253	1256	0	1	1	1	1	2	2	2	3
.10	1259	1262	1265	1268	1271	1274	1276	1279	1282	1285	0	1	1	1	1	2	2	2	3
.11	1288	1291	1294	1297	1300	1303	1306	1309	1312	1315	0	1	1	1	2	2	2	2	3
.12	1318	1321	1324	1327	1330	1334	1337	1340	1343	1346	0	1	1	1	2	2	2	2	3
.13	1349	1352	1355	1358	1361	1365	1368	1371	1374	1377	0	1	1	1	2	2	2	3	3
.14	1380	1384	1387	1390	1393	1396	1400	1403	1406	1409	0	1	1	1	2	2	2	3	3
.15	1413	1416	1419	1422	1426	1429	1432	1435	1439	1442	0	1	1	1	2	2	2	3	3
.16	1445	1449	1452	1455	1459	1462	1466	1469	1472	1476	0	1	1	1	2	2	2	3	3
.17	1479	1483	1486	1489	1493	1496	1500	1503	1507	1510	0	1	1	1	2	2	2	3	3
.18	1514	1517	1521	1524	1528	1531	1535	1538	1542	1545	0	1	1	1	2	2	2	3	3
.19	1549	1552	1556	1560	1563	1567	1570	1574	1578	1581	0	1	1	1	2	2	3	3	3

(Contd.)

Antilogarithms *(Contd.)*

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
.20	1585	1589	1592	1596	1600	1603	1607	1611	1614	1618	0	1	1	1	2	2	3	3	3
.21	1622	1626	1629	1633	1637	1641	1644	1648	1652	1656	0	1	1	2	2	2	3	3	3
.22	1660	1663	1667	1671	1675	1679	1683	1687	1690	1694	0	1	1	2	2	2	3	3	3
.23	1698	1702	1706	1710	1714	1718	1722	1726	1730	1734	0	1	1	2	2	2	3	3	4
.24	1738	1742	1746	1750	1754	1758	1762	1766	1770	1774	0	1	1	2	2	2	3	3	4
.25	1778	1782	1786	1791	1795	1799	1803	1807	1811	1816	0	1	1	2	2	2	3	3	4
.26	1820	1824	1828	1832	1837	1841	1845	1849	1854	1859	0	1	1	2	2	3	3	3	4
.27	1862	1866	1871	1875	1879	1884	1888	1892	1897	1901	0	1	1	2	2	3	3	3	4
.28	1905	1910	1914	1919	1923	1928	1932	1936	1941	1945	0	1	1	2	2	3	3	4	4
.29	1950	1954	1959	1963	1968	1972	1977	1982	1986	1991	0	1	1	2	2	3	3	4	4
.30	1995	2000	2004	2009	2014	2018	2023	2028	2032	2037	0	1	1	2	2	3	3	4	4
.31	2042	2046	2051	2056	2061	2065	2070	2075	2080	2084	0	1	1	2	2	3	3	4	4
.32	2089	2094	2099	2104	2109	2113	2118	2123	2128	2133	0	1	1	2	2	3	3	4	4
.33	2138	2143	2148	2153	2158	2163	2168	2173	2178	2183	0	1	1	2	2	3	3	4	4
.34	2188	2193	2198	2203	2208	2213	2218	2223	2228	2234	1	1	2	2	3	3	4	4	5
.35	2239	2244	2249	2254	2259	2265	2270	2275	2280	2286	1	1	2	2	3	3	4	4	5
.36	2291	2296	2301	2307	2312	2317	2323	2328	2333	2339	1	1	2	2	3	3	4	4	5
.37	2344	2350	2355	2360	2366	2371	2377	2382	2388	2393	1	1	2	2	3	3	4	4	5
.38	2399	2404	2410	2415	2421	2427	2432	2438	2443	2449	1	1	2	2	3	3	4	4	5
.39	2455	2460	2466	2472	2477	2483	2489	2495	2500	2506	1	1	2	2	3	3	4	5	5
.40	2512	2518	2523	2529	2535	2541	2547	2553	2559	2564	1	1	2	2	3	4	4	5	5
.41	2570	2576	2582	2588	2594	2600	2606	2612	2618	2624	1	1	2	2	3	4	4	5	5

(Contd.)

Antilogarithms *(Contd.)*

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
.42	2630	2636	2642	2649	2655	2661	2667	2673	2679	2685	1	1	2	2	3	4	4	5	6
.43	2692	2698	2704	2710	2716	2723	2729	2735	2742	2748	1	1	2	3	3	4	4	5	6
.44	2754	2761	2767	2773	2780	2786	2793	2799	2805	2812	1	1	2	3	3	4	4	5	6
.45	2818	2825	2831	2838	2844	2851	2858	2864	2871	2877	1	1	2	3	3	4	5	5	6
.46	2884	2891	2897	2904	2911	2917	2924	2931	2938	2944	1	1	2	3	3	4	5	5	6
.47	2951	2958	2965	2972	2979	2985	2992	2999	3006	3013	1	1	2	3	3	4	5	5	6
.48	3020	3027	3034	3041	3048	3055	3062	3069	3076	3083	1	1	2	3	4	4	5	6	6
.49	3090	3097	3105	3112	3119	3126	3133	3142	3148	3155	1	1	2	3	4	4	5	6	6
.50	3162	3170	3177	3184	3192	3199	3206	3214	3221	3228	1	1	2	3	4	4	5	6	7
.51	3236	3243	3251	3258	3266	3273	3281	3289	3296	3304	1	2	2	3	4	5	5	6	7
.52	3311	3319	3327	3334	3342	3350	3357	3365	3373	3381	1	2	2	3	4	5	5	6	7
.53	3388	3396	3404	3412	3420	3428	3436	3443	3451	3459	1	2	2	3	4	5	6	6	7
.54	3467	3475	3483	3491	3499	3508	3516	3524	3532	3540	1	2	2	3	4	5	6	6	7
.55	3548	3556	3565	3573	3581	3589	3597	3606	3614	3622	1	2	2	3	4	5	6	7	7
.56	3631	3639	3648	3656	3664	3673	3681	3690	3698	3707	1	2	3	3	4	5	6	7	8
.57	3715	3724	3733	3741	3750	3758	3767	3776	3784	3793	1	2	3	3	4	5	6	7	8
.58	3802	3811	3819	3828	3837	3846	3855	3864	3873	3882	1	2	3	4	4	5	6	7	8
.59	3890	3899	3908	3917	3926	3936	3945	3954	3963	3972	1	2	3	4	4	5	6	7	8
.60	3981	3990	3999	4009	4018	4027	4036	4046	4055	4064	1	2	3	4	5	6	6	7	8
.61	4074	4083	4093	4102	4111	4121	4130	4140	4150	4159	1	2	3	4	5	6	7	8	9

(Contd.)

Antilogarithms *(Contd.)*

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
62	4169	4178	4188	4198	4207	4217	4227	4236	4246	4256	1	2	3	4	5	6	7	8	9
63	4266	4276	4285	4295	4305	4315	4325	4335	4345	4355	1	2	3	4	5	6	7	8	9
64	4365	4375	4385	4395	4400	4416	4426	4436	4446	4457	1	2	3	4	5	6	7	8	9
65	4467	4477	4487	4498	4508	4519	4529	4539	4550	4560	1	2	3	4	5	6	7	8	9
66	4571	4581	4592	4603	4613	4624	4634	4645	4656	4667	1	2	3	4	5	6	7	9	10
67	4677	4688	4699	4710	4721	4732	4742	4753	4764	4775	1	2	3	4	5	7	8	9	10
68	4786	4797	4808	4819	4931	4842	4853	4864	4875	4887	1	2	3	4	6	7	8	9	10
69	4898	4909	4920	4932	4943	4955	4966	4977	4989	5000	1	2	3	5	6	7	8	9	10
70	5012	5023	5035	5047	5058	5070	5082	5093	5105	5117	1	2	4	5	6	7	8	9	11
71	5129	5140	5152	5164	5176	5188	5200	5212	5224	5236	1	2	4	5	6	7	8	10	11
72	5248	5260	5272	5284	5297	5309	5321	5333	5346	5358	1	2	4	5	6	7	9	10	11
73	5370	5383	5395	5408	5420	5433	5445	5458	5470	5483	1	3	4	5	6	8	9	10	11
74	5495	5508	5521	5534	5546	5559	5572	5585	5598	5610	1	3	4	5	6	8	9	10	12
75	5623	5636	5649	5662	5675	5689	5702	5715	5728	5741	1	3	4	5	7	8	9	10	12
76	5754	5768	5781	5794	5808	5821	5834	5848	5861	5875	1	3	4	5	7	8	9	11	12
77	5888	5902	5916	5929	5943	5957	5970	5984	5998	6012	1	3	4	5	7	8	10	11	12
78	6026	6039	6053	6067	6081	6095	6109	6124	6138	6152	1	3	4	6	7	8	10	11	13
79	6166	6180	6194	6209	6223	6237	6252	6266	6281	6295	1	3	4	6	7	9	10	11	13
80	6310	6324	6339	6350	6368	6383	6397	6412	6427	6442	1	3	4	6	7	9	10	12	13
81	6457	6471	6486	6501	6516	6531	6546	6561	6577	6592	2	3	5	6	8	9	10	12	14

(Contd.)

Antilogarithms *(Contd.)*

	0	1	2	3	4	5	6	7	8	9	Mean difference 1	2	3	4	5	6	7	8	9
.82	6607	6622	6637	6653	6668	6683	6699	6714	6730	6745	2	3	5	6	8	9	11	12	14
83	6761	6776	6792	6808	6823	6839	6855	6871	6887	6902	2	3	5	6	8	9	11	13	14
84	6918	6934	6950	6966	6982	6998	7015	7031	7047	7063	2	3	5	6	8	10	11	13	15
85	7079	7096	7112	7129	7145	7161	7178	7194	7211	7228	2	3	5	7	8	10	12	13	15
86	7244	7261	7278	7295	7311	7328	7345	7362	7379	7396	2	3	5	7	8	10	12	13	15
87	7413	7430	7447	7464	7482	7499	7516	7534	7551	7568	2	3	5	7	9	10	12	14	16
88	7586	7603	7621	7638	7656	7674	7691	7709	7727	7745	2	4	5	7	9	11	12	14	16
89	7762	7780	7798	7816	7834	7852	7870	7889	7907	7925	2	4	5	7	9	11	13	14	17
90	7943	7962	7980	7998	8017	8035	8054	8072	8091	8110	2	4	6	7	9	11	13	15	17
91	8128	8147	8166	8185	8204	8222	8241	8260	8279	8299	2	4	6	8	9	11	13	15	17
92	8318	8337	8356	8375	8395	8414	8433	8453	8472	8492	2	4	6	8	10	12	14	15	17
93	8511	8531	8551	8570	8590	8610	8630	8650	8670	8690	2	4	6	8	10	12	14	16	18
94	8710	8730	8750	8770	8790	8810	8831	8851	8872	8892	2	4	6	8	10	12	14	16	18
95	8913	8933	8954	8974	8995	9016	9036	9057	9078	9099	2	4	6	8	10	12	15	17	19
96	9120	9141	9162	9183	9204	9226	9247	9268	9290	9311	2	4	6	8	11	13	15	17	19
97	9333	9354	9376	9397	9419	9441	9462	9484	9506	9528	2	4	7	9	11	13	15	17	20
98	9550	9572	9594	9616	9638	9661	9683	9705	9727	9750	2	4	7	9	11	13	16	18	20
99	9772	9795	9817	9840	9863	9886	9908	9931	9954	9977	2	5	7	9	18	14	16	18	20

Appendix 3

AN ILLUSTRATIVE EXAMPLE OF DESIGN OF SLAB THICKNESS

1. Design Parameters

Location of pavement Delhi
Design wheel load $p=5100$ kg
Present traffic intensity $=300$ veh/day
Design tyre pressure $p=7.2$ kg/cm²
Foundation Strength $k=6$ kg/cm³
Concrete flexural strength $f_R=40$ kg/cm²
Other concrete parameters
$E=3.0\times10^5$ kg/cm²
$u=0.15$
$=10\times10^{-6}$ °C

2. Design Procedure

Step 1. As in para 1 above.

Step 2. Joint spacing and lane-widths

Contraction Joint spacing $L=4.5$ m
Lane width, $W=3.5$ m

Step 3. Tentative design value of slab thickness,

$h=22$ cm

Step 4. Temperature stress for edge region.

(i) From Table 2 for 22 cm thick pavement slabs in Delhi max. value of temperature differential.
$t=$ about 13.5 °C

(ii) for $h=22$ cm ; $E=3.0\times10^5$ kg/cm²
From Table 6, $l=81.89$ cm
$\therefore L/l=5.5$, $W/l=4.3$
From Table 8, $C_L=0.82$, $C_W=0.52$

(iii) From Fig. 3, for $C=0.82$ and $t=13.5$ °C
$\sigma t\ e=16.0$ kg/cm²

Step 5. Residual concrete strength for supporting loads

$f_L=f_R-\sigma t_e=40.0-16.0=24.0$ kg/cm²

Step 6. Load Stress for edge region

From Fig. 1, for $h=22$ cm, $k=6$ kg/cm³
$\sigma l_e=23.4$ kg/cm²

Step 7. Available factor of safety on load stress

$f_L/\sigma l_e = \frac{24.0}{23.4}=1.03>1 \therefore$ O.K.

Step 8. Corner load Stress

From Fig. 2, $\sigma l_C=26.8$ kg/cm² $<f_R \therefore$ O.K.

Step 9. Adjustment for traffic intensity

Design traffic intensity
$T=300\cdot(1+0.075)^{20}$
$=300\times4.25$
$=1275$, which falls under traffic classification E (Table 1)
From Table 9
Required thickness adjustmen=0
Design thickness of payment slab=22 cm

Appendix 4

AN ILLUSTRATIVE EXAMPLE OF DESIGN OF DOWEL BARS AND TIE BARS

1. DOWEL BARS

1.1. Design Parametres

Design wheel load = 5100 kg
Design load transfer = 40%
Slab thickness, $h = 22$ cm
Joint width, $z = 20$ cm
Permissible flexural stress in dowel bar = 1400 kg/cm²
Permissible shear stress in dowel bar = 1000 kg/cm²
Permissible bearing stress on concrete = 100 kg/cm²
K – value on sub-base = 8 kg/cm²/cm
Other concrete parameters :
$L = 3 \times 10^5$ kg/cm²
$u = 0.15$

1.2. Design Procedure

Steps 1 : Dowel length

Assume Dowel diameter, $d = 2.5$ cm
Then, for equal capacity in bending and bearing, from Eqn (13)

$$r = 5 \times 25 \times \left[\frac{(1400}{100} \times \frac{r+3)}{r+17.6}\right]^{\frac{1}{2}}$$

which gives on solution, $r = 40.5$ cm
So that dowel length, $L = r + z = 40.5 + 2 = 42.5$ cm
say 45 cm

Step 2 : Load transfer capacity of single dowel

From equations (10), (11) and (12), load transfer capactty of a single dowel is obtained

$$\bar{P} \text{ (in shear)} = 0.785\, d^2 f^1 S$$
$$= 0.785 \times 2.5 \times 2.5 \times 1000$$
$$= 4900 \text{ kg}$$

$$\bar{P} \text{ (in bending)} = \frac{2d^3 f_b}{r + 8.8Z}$$
$$= \frac{2 \times 2.5 \times 2.5 \times 2.5 \times 1400}{40.5 + 17.6} = 753 \text{ kg}$$

$$\bar{P} \text{ (in bearing)} = \frac{f_c r^2 d}{12.5(r + 1.5z)}$$
$$= \frac{100 \times 40.5 \times 40.5 \times 2.5}{12.5(40.5 + 3)}$$
$$= 754 \text{ kg}$$

Taking the least of these values for design purposes,
$\bar{P}$ 753 kg

Step 3 : Capacity factor required of dowel system Load transfer capacity of the dowel system
$= 5100 \times 40\% = 2040$ kg

$$\therefore \text{ required capacity factor} = \frac{2040}{753} = 2.70$$

Step 4 : Spacing of dowel bars

Fork = 8 kg/cm²/cm, l = 76.20 (from Table 6)
Considering the joint corner, the distance over which dowel bars are effective in load transfer = $1.8 \times l = 1.8 \times 76.2 = 137.0$ cm about.
Assuming a dowel spacing of 25 cm
Available capacity factor

$$= 1 + \frac{137-25}{137} + \frac{137-50}{137} + \frac{137-75}{137}$$
$$= 4 - \frac{150}{137} = 4 - 1.09$$
$$= 2.91$$

which is slightly greater than the required capacity factor of 2.70. Hence adopt 25 cm as dowel spacing.

2. TIE BARS

2.1. Design Parameters

Slab thickness, $h = 22$ cm
Slab width, $b = 3.35$ m
No. of lanes to be tied = 2
Coefficient of friction between payment and subgrade = f = 1.5
Weight per m² of concrete slab, $w = 528$ kg
Allowable working tensile stress in steel S = 1400 kg/cm²
Maximum Permissible bond stress, B in :
(i) Plain tie bars = 17.5 kg/cm²
(ii) Deformed bars = 24 kg/cm²

2.2. Design Procedure

Step 1 : Diameter and spacing of tie bars
Weight per m² of concrete slab, $W = 528$ kg

$\therefore$ Area of steel required per m width of joint

$A_s = \frac{bfW}{S} = \frac{3.35 \times 1.5 \times 528}{1400} = 1.89 \text{ cm}^2/\text{m}$

Assuming dia of tie bars; $d = 10$ mm

A = cross section of one tie bar = 78.54 mm²

P = Perimeter of tie bar = 31.42 mm

$\therefore$ No. of tie bars required per m, $N = \frac{A_s}{A} = \frac{1.89}{0.7854}$, and

Spacing of tie bars $= \frac{100}{N} = \frac{100 \times 0.7854}{1.89} = 41.5$ cm Say 42 cm

Step 2 : Length of the tie bars :

Length of tie bar $= \frac{2\, sA}{B\, P} = \frac{2 \times 1400 \times 78.54 \times 10}{B \times 100 \times 31.42}$

$L = 40$ cm for plain tie bars.

and 29.2 cm for deformed tie bars

Increasing about 5 cm for tolerance in placement

$L = 45$ cm for plain tie bars

and $L = 35$ cm for deformed tie bars

LIST OF OTHER CEMENT CONCRETE ROAD STANDARDS

			Rs. P.
1.	IRC: 15-1981	Standard Specifications & Code of Practice for Construction of Concrete Roads (Second Revision)	16-00
2.	IRC: 43-1972	Recommended Practice for Tools, Equipment and Appliances for Concrete Pavement Construction	12-00
3.	IRC: 44-1972	Tentative Guidelines for Cement Concrete Mix Design (for Road Pavements for non-air entrained and continuously graded concrete (First Revision)	8-00
4.	IRC: 57-1974	Recommended Practice for Sealing of Joints in Concrete Pavements	3-00
5.	IRC: 59-1976	Tentative Guidelines for the Design of Gap Graded Cement Concrete Mixes for Road Pavements	5-00
6.	IRC: 60-1976	Tentative Guidelines for the Use of Lime-flyash Concrete as Pavement Base or Sub-Base	5-00
7.	IRC: 61-1976	Tentative Guidelines for the Construction of Cement Concrete Pavements in Hot-Weather	5-00
8.	IRC: 68-1976	Tentative Guidelines on Cement-Flyash Concrete for Rigid Pavement Construction	6-00
9.	IRC: 74-1979	Tentative Guidelines for Lean-Cement Concrete and Lean Cement Flyash Concrete as a Pavement Base or Sub-Base	8-00
10.	IRC: 76-1979	Tentative Guidelines for Structural Strength Evaluation of Rigid Airfield Pavements	10-00
11.	IRC: 77-1979	Tentative Guidelines for Repair of Concrete Pavements using Synthetic Resin	15-00
12.	IRC: 84-1983	Code of Practice for Curing of Cement Concrete Pavements	8-00
13.	IRC: 95-1983	Recommended Practice for Accelerated Strength Testing and Evaluation of Concrete for Road and Airfield Constructions	8-00
14.	IRC: 91-1985	Tentative Guidelines for Construction of Cement Concrete Pavements in Cold Weather	8-00
15.	IRC: 101-1988	Guidelines for Design of Continuously Reinforced Concrete Pavement with Elastic Joints	12-00
16.	IRC: SP: 11-1977	Handbook of Quality Control for Construction of Roads and Runways (Second Revision)	32-00

17.	IRC: SP: 16-1977	Surface Evenness of Highway Pavement	7-00
18.	IRC: SP: 17-1977	Recommendations About Overlays on Cement Concrete Pavements	15-00
19.	MOST	Ministry of Shipping & Transport (Roads Wing) Handbook on Road Construction, Machinery (1985)	32-00
20.	MOST	Ministry of Surface Transport (Roads Wing), Specifications for Road and Bridge Works (Second Revision)	80-00
21.	MOST	Ministry of Shipping & Transport (Roads Wing) Manual for Maintenance of Roads	24-00

IRC : 63-19

TENTATIVE GUIDELINES FOR THE USE OF LOW GRADE AGGREGATES AND SOIL AGGREGATE MIXTURES IN ROAD PAVEMENT CONSTRUCTION

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TENTATIVE GUIDELINES FOR THE USE OF LOW GRADE AGGREGATES AND SOIL AGGREGATE MIXTURES IN ROAD PAVEMENT CONSTRUCTION

1. INTRODUCTION

1.1. Hard aggregates are not available in some parts of the country. In some areas, these are costly and not available within economical leads. Under these circumstances, use of locally available low-grade aggregates assumes great importance. At present, the engineer often looks for hard aggregates even if the cost is high. The main handicap in the use of low-grade aggregates was that there was no guidance available in this respect. Keeping this aspect in view, the Soil Engineering Committee (personnel given below) in their meeting held at Gandhinagar on the 29th November 1972 prepared these tentative guidelines for the use of low-grade aggregates.

J.S. Marya	...	*Convenor*
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The Director General (Road Development) & Addl. Secretary to the Govt. of India	*ex-officio*	"

The draft guidelines were processed by the Specifications and Standards Committee in their meeting held on 31st January and 1st February 1974 and later approved by the Executive Committee and the Council in their meetings held on the 22nd December 1975 and 3rd January 1976.

2. SCOPE

2.1. Low grade aggregates are those aggregates which lose strength generally by more than 15 per cent upon wetting, when measured in terms of their Aggregate Impact Value. Low-grade aggregates can be used as such if their Wet Aggregate Impact Value does not exceed 50 per cent. If the Wet Aggregate Impact Value exceeds 50 per cent, they would require to be suitably processed before being used. This could be achieved either through stabilisation, say in accordance with IRC: 28-1967 "Tentative Specification for the Construction of Stabilized Soil Roads with Soft Aggregate in Areas of Moderate and High Rainfall" or using the aggregates in a soil-aggregate mixture as discussed in para 5.

2.2. Since one of the significant characteristics of most types of low grade aggregates is loss of mechanical strength upon wetting, the testing of such aggregates should invariably be done in the soaked condition, as per IS: 5640-1970, "Method of Test for Determining Aggregate Impact Value of Soft Coarse Aggregates". In addition to this, it is advisable to run dry Aggregate Impact Value tests occasionally (vide IS: 2386 Part IV)-1963 to get an idea about the comparative performance of the concerned aggregates.

3. COMMON TYPES OF LOW GRADE AGGREGATES

3.1. Some of the common types of low grade aggregates that are normally encountered in India are mentioned in Table 1. It should be noted that the aggregates can sometimes be of a very variable quality, for instance Dhandla. As such mere nomenclature should not be the guide for selection of aggregates but their actual physical characteristics.

TABLE 1. LOW GRADE AGGREGATES

(i) Laterite
(ii) Kankar
(iii) Shale
(iv) Moorum
(v) Soft Gravel
(vi) Dhandla
(vii) Brick Aggregate
(viii) Soft Stone

4. TESTING OF LOW GRADE AGGREGATES AND SOIL-AGGREGATE MIXTURES

4.1. Some of the tests considered appropriate for evaluating the suitability of low grade aggregates and of soil-aggregate mixtures, for use in pavement construction, are indicated in Tables 2 and 3.

TABLE 2. TESTING OF LOW GRADE AGGREGATES

(i) Aggregate Impact Value Test (wet/dry as mentioned in Clause I)—IS:5640-1970/IS:2386 (Part IV)-1963.
(ii) Sodium sulphate soundness test (This test need be conducted only if the aggregates are to be used in a sulphate-infested area)—IS:2386 (Part V)-1963.
(iii) CBR on samples soaked for 4 days (only in the case of moorum or soil-aggregate admixtures)—IS:2720 (Part XVI)-1965.

TABLE 3. TESTING OF SOIL AGGREGATE MIXTURES

(i) Gradation test—IS:2720 (Part IV)-1965
(ii) Liquid limit and plasticity index of soil fraction—IS:2720 (Part V)-1970
(iii) CBR on samples soaked for 4 days (wherever applicable)—IS:2720 (Part XVI)-1965.

5. CRITERIA FOR USE OF LOW GRADE AGGREGATES

5.1. Physical Requirements

5.1.1. Low grade aggregates can be used for sub-base or base courses of road pavements, or even sometimes as surfacing. For application in individual cases, the suitability of aggregates, except for materials like moorum, should be based on the Wet Aggregate Impact Value. Recommended limits in this regard are set forth in Table 4.

TABLE 4. PHYSICAL REQUIREMENTS OF LOW GRADE AGGREGATES

Sl. No.	Type of Construction	Test*	Test Method	Requirement
1.	Sub-base	Wet Aggregate Impact Value	IS:5640-1970	Max. 50%
2.	Base course with bituminous surfacing	—do—	—do—	Max. 40%
3.	Surfacing course	—do—	—do—	Max. 30%

5.1.2. In the case of materials like moorum, suitability for use in pavement courses should be judged on the basis of soaked CBR values. Whether moorum is used as sub-base for high class roads, or as a surfacing for lightly trafficked roads, the soaked CBR value should not be less than 20. Where this requirement is not satisfied, or a still higher strength is desired, this could be achieved through cement or lime stabilisation in accordance with IRC:50-1973 and IRC : 51-1973 respectively.

5.2. Gradation

5.2.1. Low grade aggregates should be reasonably well graded so as to achieve a dense and well interlocked mass. Recommended gradings for aggregates to be used in Water Bound Macadam construction are given in IRC : 19-1972. For low grade aggregates, these gradings should be taken by way of guidance only since such aggregates are generally of a crushable nature.

6. CRITERIA FOR USE OF SOIL-AGGREGATE MIXTURES

6.1. Soil-aggregate mixtures may be in the form of naturally occurring materials like soil-gravel, or soil purposely blended with suitable aggregate fractions. The primary criteria for acceptability of such materials are plasticity index and gradation. Plasticity index of the material should be less than 6 when used as sub-base or base course with bituminous surfacing and between 6 and 9 when used as a surfacing for lightly trafficked roads. Criteria for gradation should be as set forth in para 6.2.

6.2. The material should be smoothly graded for achieving the maximum possible dry density. Fuller's grading rule** could be used as a guide to work out the optimum grading in different cases. A few typical gradings are given in Table 5 for general application. The first three gradings indicated in Table 5 are especially suited for base courses whereas the remaining two are suitable both for base course and for surfacing.

Note : *Samples for tests should be representative of the materials to be used and collected in accordance with the procedure set forth in IS:2430-1969.

**Fuller's grading rule is given by per cent passing

$$\text{sieve} = 100\left[\frac{\text{aperture size of sieve}}{\text{size of the largest particle}}\right]^{\frac{1}{2}}$$

TABLE 5. TYPICAL GRADING LIMITS FOR SOIL-AGGREGATES MIXTURES

Sieve designation (IS:460-1962)	Nominal Maximum size of material				
	80 mm	40 mm	20 mm	10 mm	5 mm
	Per cent by weight passing the sieve				
80 mm	100				
40 mm	80-100	100			
20 mm	60-80	80-100	100		
10 mm	45-65	55-80	80-100	100	
4.75 mm	30-50	40-60	50-75	80-100	100
2.36 mm	—	30-50	35-60	50-80	80-100
1.18 mm	—	—	—	40-65	50-80
600 micron	10-30	15-30	15-35	—	30-60
300 micron	—	—	—	20-40	20-45
75 micron	5-15	5-15	5-15	10-25	10-25

Note : Not less than 10 per cent should be retained between each pair of successive sieves specified for use except for the larger pair.

6.3. Apart from PI value and gradation, soil-aggregate mixtures may also be evaluated on the basis of soaked CBR value determined in accordance with IS: 2720 (Part XVI)-1965. Where this approach is followed, CBR should desirably be not less than 20 for use as a sub-base. In the case of base courses, the acceptable value of CBR for heavily trafficked routes is normally 80, but a somewhat lower value could be permitted for arid areas, or light volume roads depending on the discretion of the Engineer-in-charge.

7. PAVEMENT DESIGN

7.1. Thickness of flexible pavements using low grade aggregates or soil-aggregate mixtures should be designed in the normal way in accordance with IRC : 37-1970 "Guidelines for the Design of Flexible Pavements". Minimum thickness of any such courses should be 10-15 cm, except in the case of moorum, when it should be 15 cm.

7.2. For use as sub-base under rigid pavements, guidance can be had from IRC : 58-1974, "Guidelines for the Design of Rigid Pavements for Highways".

7.3. Whenever low-grade aggregates are used as a sub-base/base course, these should be laid preferably on a well compacted subgrade. Also the road should be kept well drained.

RECOMMENDED PRACTICE
FOR
SIGHT DISTANCE ON RURAL HIGHWAYS

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1. INTRODUCTION

1.1. Ability to see ahead is of profound importance for the safe and efficient operation of vehicles on a highway. If greater safety is to be built into highway alignments, the design must ensure sight distance of adequate length in different situations to permit drivers enough time and distance to control their vehicles so as to avoid unforeseen accidents.

1.2. In 1950, the Specifications and Standards Committee had published a Paper on sight distances (Paper No. 149, "Standards for Sight Distances for Highways", IRC Journal Vol. XV-1) which has remained the mainstay of highway practice in the country so far. There have been significant developments since then. Taking these into account, revised recommendations on this subject have been evolved for uniform adoption on all rural highways. The present Recommended Practice was approved by the Specifications and Standards Committee in their meeting held on the 12th and 13th December, 1975 subject to a few modifications and later it was approved by the Executive Committee in their meeting held on the 14th April 1976 and by the Council in their 87th meeting held on 27th August 1976.

1.3. In applying this standard, effort should not be to limit the design of any highway to the minimum values laid down. Where conditions are favourable, good engineering practice will lie in adopting more liberal values, particularly for stopping sight distance.

2. STOPPING SIGHT DISTANCE

2.1. General

2.1.1. Stopping sight distance is the minimum sight distance for which all roads must always be designed, regardless of any other consideration. It is the clear distance ahead needed by a driver to stop his vehicle before meeting a stationary object in his path on the road.

2.1.2. Stopping distance has two components:

(i) the distance travelled during perception and brake reaction time; and

(ii) the distance travelled during the time brakes are under application till the vehicle comes to a stop.

2.2. Perception and Brake Reaction Time

2.2.1. Perception and brake reaction time is the time interval between the instant the driver sights a dangerous object for which a stop is necessary and the instant the brakes are applied.

2.2.2. Perception and brake reaction time depends on a variety of factors, viz., age, sex, alertness and visual acuity of the driver, atmospheric visibility, vehicle design, the size and type of the object etc. For purposes of highway design, the total reaction time should be large enough to cover nearly all drivers and highway conditions. A value of 2.5 seconds is considered reasonable for most situations. The distance travelled during this time will be given by the expression:

$$d_1 = 0.278\ Vt$$

where d_1 = distance travelled during total reaction time in metres;

V = speed in kmph; and

t = the perception and reaction time in seconds.

2.3. Braking Distance

2.3.1. Braking distance is the distance required a vehicle to come to stop after the brakes are applied. On a level road, assuming friction remains constant during the period of deceleration, braking distance is given by:

$$d_2 = \frac{V^2}{254f}$$

where d_2 = braking distance in metres;

V = speed in kmph; and

f = coefficient of longitudinal friction between vehicle tyres and road pavement.

2.3.2. The value of the coefficient of friction varies with speed, tyre pressure, condition of tyre tread, type and condition of pavement, and whether the surface is wet or dry. For design purposes, the value should encompass nearly all significant pavement surface types and field conditions, and should be safe for tyres in reasonable condition. Based on these considerations, design values for coefficient of friction at different vehicle speeds are given in Table 1.

TABLE 1. STOPPING SIGHT DISTANCE ON RURAL HIGHWAYS

Speed kmph	Perception and brake reaction		Braking		Safe stopping sight distance (metres)	
	Time (seconds)	Distance (metres)	Coefficient of longitudinal friction	Distance (metres)	Calculated values	Rounded off values for design
20	2.5	14	0.40	4	18	20
25	2.5	18	0.40	6	24	25
30	2.5	21	0.40	9	30	30
40	2.5	28	0.38	17	45	45
50	2.5	35	0.37	27	62	60
60	2.5	42	0.36	39	81	80
65	2.5	45	0.36	46	91	90
80	2.5	56	0.35	72	118	120
100	2.5	70	0.35	112	182	180

2.4. Design Values

2.4.1. Minimum stopping sight distance is given by the sum of the components d_1 and d_2 discussed in preceding paragraphs. Calculated and rounded values of stopping distance for different vehicle speeds are given in Table 1. For application of values in this table, the speed chosen should be the same as the design speed of the road.

2.5. Effect of Grade

2.5.1. The braking distance of a vehicle is longer on downgrades and shorter on upgrades. The braking distance formula amended to take the effect of grades into account is :

$$d_2 = \frac{V^2}{254\ (f \pm 0.01G)}$$

in which G is the longitudinal grade in per cent (positive for upgrade and negative for downgrade) and other terms are the same as previously defined.

2.5.2. Correction for grade should not be applied on undivided roads with two way traffic, but must invariably be considered for divided highways which have independently designed profiles.

2.6. Criterion for Measurement

Safe stopping sight distance is measured between two points, one 1.2 m above the carriageway standing for in driver's eye and the other 0.15 m height representing the object.

3. OVERTAKING SIGHT DISTANCE

3.1. Design Criteria

3.1.1. For a higher level of service on undivided roads, it is necessary that vehicles moving at design speed should be frequently able to overtake vehicles slower than them. Since overtaking manoeuvre involves the occupation of road space normally used by opposing traffic, drivers must have sufficient sight distance available to them so that the whole operation can be accomplished with safety. Optimum condition is one in which the overtaking driver can follow the vehicle ahead for a short time while he assesses his chances of overtaking, pull out, overtake, and return to his own side of the road before meeting any oncoming vehicle.

3.1.2. In actual practice, there may be occasions to consider multiple overtakings where two or more vehicles overtake another vehicle, or are themselves overtaken in a single manoeuvre. It is, however, not practical to assume such conditions in developing minimum sight distance criteria. Sight distance values recommended here pertain basically to overtaking manoeuvres involving single vehicles. Longer sight distances are generally available along road alignments, e.g., in long relatively level sections, where an occasional multiple overtaking can take place without difficulty.

3.1.3. For computing minimum overtaking sight distance, certain assumptions for traffic behaviour are necessary. The assumptions made are :

(i) The vehicle being overtaking is travelling at a uniform speed which is 16 km per hour less than the design speed of the road ;

(ii) The overtaking vehicle follows the vehicle ahead for a short while to perceive the clear road ahead before beginning the overtaking movement;

(iii) Overtaking is done by accelerating rapidly to the design speed and is regarded as having been completed when the overtaking vehicle returns to its own side of the road ; and

(iv) Overtaking once begun is finished in the face of an oncoming vehicle travelling at design speed in such a way that the latter arrives alongside the former just at the completion of the manoeuvre.

3.1.4. Observations in the U.S.A. and elsewhere have shown that the overtaking manoeuvre takes roughly 8 to 14 seconds for a vehicle closing at the design speed. To this should be added the distance travelled by an opposing vehicle during the time of overtaking manoeuvre to minimise the chance of a collision while the overtaking vehicle is on right-hand side of the road. Conservatively, this distance should be the distance traversed by an opposing vehicle during the entire time of the overtaking manoeuvre. But this makes the overtaking distance too long and is seriously open to question. During the first phase of the overtaking manoeuvre when the overtaking vehicle has not yet pulled abreast of the vehicle being overtaken, the former can always return to its own side if an oncoming vehicle is sighted. The interval of the first phase manoeuvre is about one-third the total time required for overtaking. On this basis, the element in the overtaking sight distance for the opposing vehicle can be reasonably taken to be the distance it traverses during two-thirds of the actual time for overtaking. The opposing vehicle is assumed to travel at design speed during this period.

3.2. Design Values for Overtaking Sight Distance

3.2.1. Using the above assumptions, safe overtaking sight distances for different speeds have been calculated in Table 2 and rounded off values are given for design purposes.

TABLE 2. OVERTAKING SIGHT DISTANCE FOR TWO-LANE HIGHWAYS

Speed kmph	Time component, seconds			Safe overtaking* sight distance (metre)
	For overtaking manoeuvre	For opposing vehicle	Total	
40	9	6	15	165
50	10	7	17	235
60	10.8	7.2	18	300
65	11.5	7.5	19	340
80	12.5	8.5	21	470
100	14	9	23	640

* Rounded off values

3.2.2 The design values in Table 2 pertain to overtaking of a vehicle by a passenger car at level grade. On upgrades, the sight distance required would be more due to reduced acceleration of the overtaking vehicle and the likely speeding up of the vehicle from opposing direction. These factors are somewhat compensated by the loss in speed of the overtaken vehicle which may frequently be a heavy truck. No separate design values are, therefore, recommended for application on grades.

3.3. Application

3.3.1. On single carriageways with two-way traffic (i.e., undivided roads of single or two-lane width), normally the attempt should be to provide overtaking sight distance in as much length of the road as possible. Conditions ideal for this application will be—

(i) straight sections of road with isolated overbridges or summit vertical curves where the provision of overtaking sight distance would result in unobstructed sight distance over a long length of the road; and

(ii) relatively easy sections of terrain adjacent to long reaches with no opportunity for overtaking at all, e.g., at the ends of an excessively winding road in hilly/rolling country.

3.3.2. In sections where application of overtaking sight distance is considered impractical for reasons of economics or otherwise, as in an undulating terrain, as far as feasible the design should aim at providing the intermediate sight distance discussed in para 4. Where visibility corresponds to these conditions, drivers should be cautioned about the limited sight distance for overtaking through appropriate speed limit signs. The posted speed should be that at which the overtaking manoeuvre can be completed with full safety, vide Table 2.

3.3.3. At summit curves and horiozntal curves not satisfying requirements of even the intermediate sight distance, it will be necessary to provide restrictive pavement markings in accordance with IRC : 35-1970 "Code of Practice for Road Markings (with paints)". Where the road section involved is long, "No Overtaking" signs should be installed at the beginning and at intervals.

3.4. Criterion for Measurement

Overtaking sight distances is measured between two points both 1.2 metre above the carriageway, one standing for the driver eye height and the other for the height of object above the road surface.

4. INTERMEDIATE SIGHT DISTANCE

4.1. Design Values

4.1.1. Sections of roads where the customary overtaking sight distance cannot be provided should be designed, as far as possible, for "intermediate sight distance" which is defined as twice the normal safe stopping distance. It is the experience that intermediate sight distance improves visibility appreciably and affords a reasonable chance to drivers to overtake with caution.

4.1.2. Recommended values of intermediate sight distance for different speeds are given in Table 3.

TABLE 3. INTERMEDIATE SIGHT DISTANCE FOR TWO-LANE HIGHWAYS

Speed kmph	Intermediate sight distance (metre)
20	40
25	50
30	60
40	90
50	120
60	160
65	180
80	240
100	360

4.2. Application

As explained in paras 3.3.2. and 3.3.3.

4.3. Criterion for Measurement

Intermediate sight distance is measured between two points 1.2 metre above the carriageway.

5. HEADLIGHT SIGHT DISTANCE AT VALLEY CURVES

5.1. During day time, visibility is hardly a problem on valley curves. But for night travel the design must ensure that the roadway ahead is illuminated by vehicle headlights for a sufficient length which enables the vehicle to brake to a stop, if necessary. This is known as the headlight sight distance and is equal to the safe stop- ping sight distance given in Table 1. From safety considerations, it is essential that valley curves should be designed to provide for this visibility.

5.2. Criterion for Measurement

For designing valley curves, the following criteria should be followed to ensure the headlight sight distance :

(i) height of headlight above the road surface is 0.75 m;

(ii) the useful beam of headlight is one degree upwards from the grade of the road; and

(iii) the height of object is nil.

6. SIGHT DISTANCE FOR DIVIDED HIGHWAYS

6.1. On divided highways with 4 or more lanes, it is not necessary to provide overtaking sight distance as required for single carriageways with-two-way traffic. However, sight distance adequ- ate for safe stopping for the design speed vide Table 1 must be ensured at all points along the highway. In fact it will be a good practice to design for somewhat more liberal values to make allo- wance for the time a driver takes to recognise whether a vehicle ahead has stopped and, if it has, whether it is on the carriageway or the shoulder.

7. SIGHT DISTANCE ON HORIZONTAL CURVES

7.1. Sight distance across the inside of horizontal curves is an important element of design. Lack of visibility in the lateral direc- tion may arise due to obstructions like walls, cut slopes, buildings, wooded areas, high farm crops etc. The straightforward manner of achieving the necessary setback in these situations is to remove the obstruction. If somehow this is not feasible, alignment of the road may need adjustments. Preferably each such case should be studied separately to determine the best course to adopt.

7.2. The setback distance to give the desired sight distance on the inside of horizontal curves can be calculated from the

following equation (see Fig. 1 for definitions):

$$m = R - (R - n) \cos \theta$$

where $\theta = \dfrac{S}{2(R-n)}$ radians

m = the minimum setback distance to sight obstruction in metres at the middle of the curve (measured from the centre line of the road);

R = radius at the centre line of the road in metres

n = distance between the centre line of the road and the centre line of the inside lane in metres;

and S = sight distance in metres.

In the above equation, sight distance is measured along the middle of inner lane. On narrow, single-lane roads, this refinement is not necessary and the setback distance should be provided with respect to the centre-line of the road, i.e., assuming 'n' to be zero.

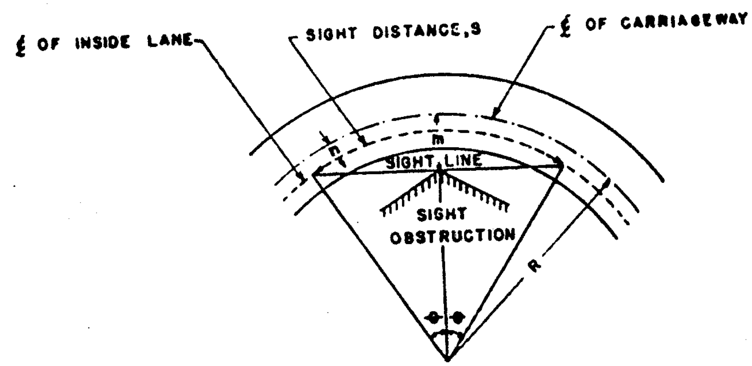

Fig. 1. Visibility at horizontal curves

7.3. Utilising the above equation, design charts for lateral clearance corresponding to the safe stopping distance are given in Fig. 2 for two-lane roads. The plotted values relate basically to circular curves longer than the design sight distance. For shorter curves, the values of setback distance found from Fig. 2 will be somewhat on the higher side, but these can any way be used as a guide.

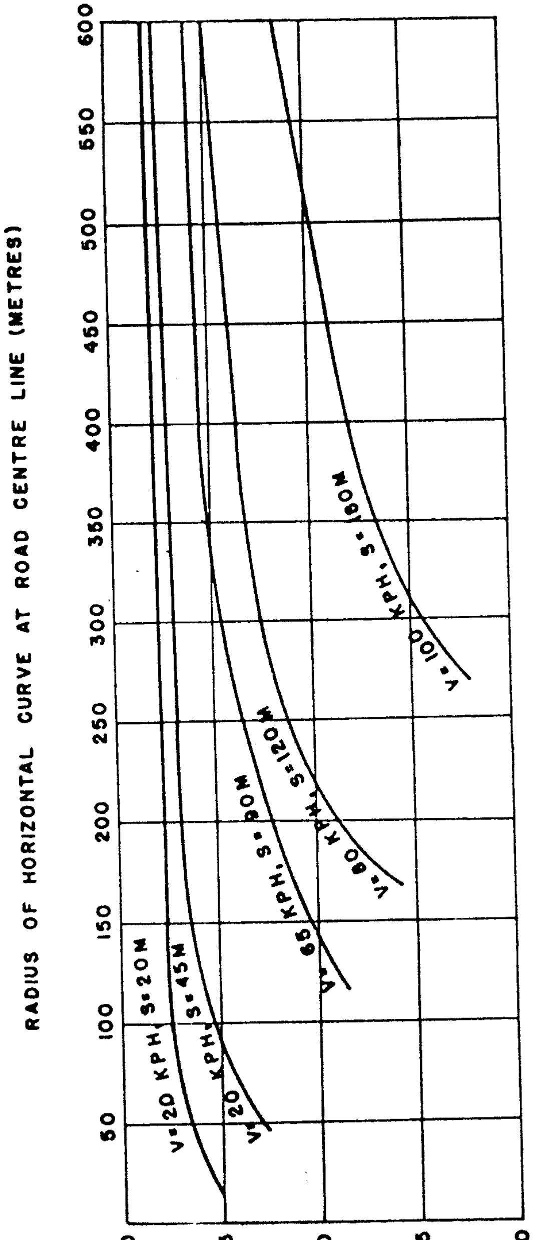

Fig. 2. Minimum setback distance required at horizontal curves on two-lane roads for safe stopping sight distance

7.4. Lateral clearance for overtaking or intermediate sight distance could be computed similarly. Calculations would, however, reveal that the setback distance required will usually be too large to be economically feasible except on very flat curves.

7.5. When there is a cut slope on the inside of the horizontal curve, a practical consideration in providing the setback distance is the average height of sight line above the ground level. For stopping sight distance, the average height can be assumed as 0.7 m since the height criteria are 1.2 m for the eye and 0.15 m for the object. Cut slopes should be kept clear above this height at the midpoint of the sight line by cutting back the slope or benching. In the case of meeting or overtaking sight distance, height of sight line above the ground would be 1.2 metre.

7.6. Where a horizontal and summit vertical curve overlap, the line of sight will not be over the top of the crest but to one side, and in part may be off the roadway. Design in such cases should provide for the required sight distance both in the vertical direction along the pavement and in the horizontal direction on the inside of the curve.

8. MEASURING AND RECORDING SIGHT DISTANCES GRAPHICALLY

8.1. Provision of the required sight distance should receive care right from early stages when the alignment of a highway is still flexible and subject to adjustments. Quick appraisals are best had by graphical means. By determining the available sight distance graphically from plans and profile drawings, and recording it at convenient intervals, deficiencies in visibility can be detected in time so that necessary modifications could be made before detailed design.

8.2. Horizontal sight distances can be directly scaled from plans on which obstructions to visibility such as buildings, plantation, hill slopes etc., have been marked. The measurement is done with the help of a straight edge.

8.3. Measurement of vertical sight may be done from plotted profiles of the highway. A transparent straight edge with parallel edges 1.2 m apart and a dotted line 0.15 m from the upper edge as per the vertical scale of the profile is the tool employed for these measurements. The transparent strip is placed on the profile with the lower edge at the station for which available sight distance is desired and the strip revolved about this point till the upper edge touches the profile. Stopping sight distance available is then the distance between the first station and the point of intersection of the dotted line with the profile. Overtaking and intermediate sight distances will be the distance between the initial station and the point where the lower edge of the strip meets the profile.

8.4. The horizontal and vertical sight distance whichever is smaller should be recorded on the plan—L section drawings. The available sight distance for stopping and overtaking should be shown in two separate columns below the profile drawing. Such records take up very little space on the drawings but are invaluable for highway design. These can also be used for fixing boundaries of the no-passing zones.

9. SIGHT DISTANCE AT INTERSECTIONS

9.1. General

9.1.1. Visibility is an important requirement at intersections. To avoid collisions, it is essential that sufficient sight distance is available along the intersecting roads and their included corners, to enable the operators of vehicles simultaneously approaching the intersection to see each other in time.

9.1.2. At-grade intersections can be broadly grouped under two headings:

(i) 'Uncontrolled intersections' where the intersecting roads are of more or less equal importance and there is no established priority;

(ii) 'Priority intersections', like minor-major road intersections, where one road takes virtual precedence over the other. Traffic on the minor road may be controlled by STOP on GIVE WAY signs/road markings, making it clear that the other road has the priority.

9.2. Uncontrolled Intersections

9.2.1. At these intersections, visibility should be provided on the principle that the drivers of vehicles on either highway are able to sight the intersection and the intersecting highway in good time to be able to halt their vehicles if that becomes essential. The area for clear visibility should be determined with respect to the stopping sight distance for each highway corresponding to the design speed, Table 1.

9.2.2. Minimum sight triangles in the included corners of uncontrolled intersections, which must be kept free of all obstructions to sight, could be demarcated as illustrated in Fig. 3. If visibility conditions upto this standard are ensured, drivers of vehicles will be able to either stop or adjust their speed in the event of a dangerous situation ahead.

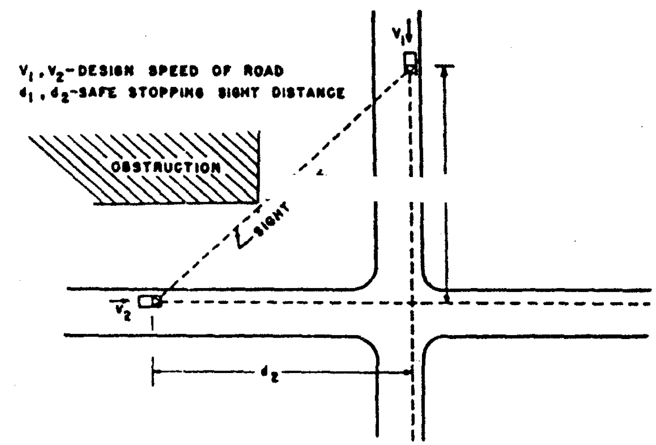

Fig. 3. Minimum sight triangle at uncontrolled intersections

9.2.3. Occasionally, the size of the sight triangle available may be less than the desirable minimum due to presence of an obstruction which cannot be removed except at prohibitive cost. In such circumstances, the vehicles must be appropriately warned to travel at speeds corresponding to the available sight distance and not at the design speed of the highway. One solution can be to permit vehicles on one of the roads to travel at the design speed and evaluate the corresponding critical speed for the other road which might be posted. Alternatively, the approach speed for both the roads could be restricted in accordance with the sight triangle available by installing suitable speed limit signs.

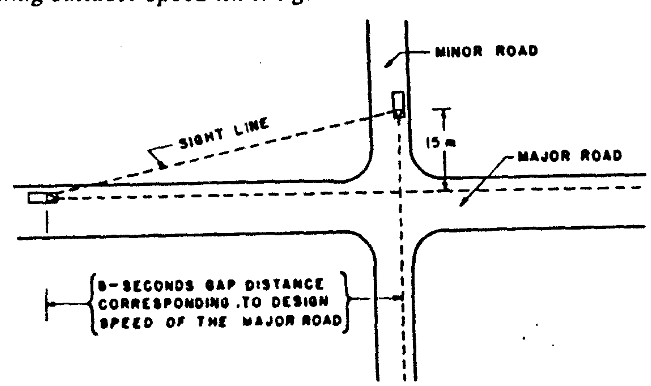

Fig. 4. Minimum sight triangle at priority intersections

9.3. Priority Intersections

9.3.1. On priority intersections, the visibility provided should be such that drivers approaching from the minor road are able to see vehicles on the major road in adequate time and to judge whether the required gap is available in the main road traffic stream for a safe crossing so that the vehicle could be brought to a halt, if necessary. For this purpose, a minimum visibility distance of 15 m along the minor road is recommended.

TABLE 4. MINIMUM VISIBILITY DISTANCES ALONG MAJOR ROADS AT PRIORITY INTERSECTIONS

Design speed of major road (kmph)	Minimum visibility distance along major roads (metres)
100	220
80	180
65	145
50	110

9.3.2. Visibility distance along the major road depends on the time required by the driver on the minor road to perceive the traffic conditions on the intersection, evaluate the gaps in the vehicle stream, take a decision about actual crossing, and finally to accelerate the vehicle to complete the manoeuvre. The total time required for these operations may be taken as 8 seconds. On this basis, the sight triangle at priority intersections should be formed by measuring 15 m along the minor road and a distance along the major road equal to 8 seconds travel at the design speed. This is illustrated in Fig. 4. Visibility distances (rounded values) corresponding to 8 seconds travel time are set out in Table 4.

IRC: 72-1978

RECOMMENDED PRACTICE FOR USE AND UPKEEP OF EQUIPMENT, TOOLS AND APPLIANCES FOR BITUMINOUS PAVEMENT CONSTRUCTION

INDIAN ROADS CONGRESS

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19.	H. C. Malhotra	Chief Engineer, Himachal Pradesh
20.	M. R. Malya	Development Manager, Gammon India Ltd. Bombay
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RECOMMENDED PRACTICE FOR USE AND UPKEEP OF EQUIPMENT, TOOLS AND APPLIANCES FOR BITUMINOUS PAVEMENT CONSTRUCTION

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RECOMMENDED PRACTICE FOR USE AND UPKEEP OF EQUIPMENT, TOOLS AND APPLIANCES FOR BITUMINOUS PAVEMENT CONSTRUCTION

1. INTRODUCTION

1.1. A large variety of equipment, tools and appliances is needed for bituminous pavement construction. Modern construction techniques need sophisticated equipment and even the specifications which permit manual mixing and laying involve the use of smaller tools and appliances. As such some guidance is necessary for field engineers regarding the use and proper upkeep of different items of equipment, tools and appliances. This will help in systematic procurement, planning and execution of works and to exercise requisite quality control in the field.

1.2. It is with the above objective that this Recommended Practice has been prepared by the Bituminous Pavements Committee (personnel given below). It was then processed and approved by the Specifications and Standards Committee in their meeting held on 21st December, 1977. Later it was finally approved by the Executive Committee in their meeting held on the 22nd April, 1978 and by the Council in their 93rd meeting held on the 3rd June 1978.

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The Director General (Road Development) and Additional Secretary to the Government of India) (*Ex-officio*)

2. SCOPE

2.1. The Recommended Practice lists out the equipment, tools and appliances required for different types of bituminous pavement construction and gives some details about these for helping the field engineer in the planning and execution of works. For commercially fabricated equipment, reference has been drawn to the relative standard of ISI. For tools and appliances not covered by any standard, details along with dimensioned sketches have been included to facilitate their fabrication.

2.2. For convenience of reference, the items of equipment, tools and appliances are given under the following two categories:

(i) Common tools and appliances which will generally be needed on all types of bituminous works; and
(ii) Special equipment needed for individual Specifications.

3. COMMON TOOLS AND APPLIANCES

A list of tools and appliances commonly required for all types of bituminous works is given below:

A. Tools and appliances for picking up or removing old pavement

(1) Pickaxes
(2) Crowbars
(3) Hammers
(4) Chisels

B. Tools and appliances for laying out

(1) Pegs
(2) Nails
(3) Rope
(4) Measuring tape
(5) Chalk
(6) Angle iron or wooden strips of required dimension for edge support.

C. Tools and appliances for cleaning the surface

(1) Wire brushes
(2) Coir brushes
(3) Brooms
(4) Old gunny bags

D. Tools and appliances for handling materials

(1) Baskets (lined with gunny cloth)

(2) Buckets (G.I.) 6-12 litres capacity
(3) Empty drums or G.I. Sheet tanks for storage of water (200 litres capacity)
(4) Wheel barrows
(5) Hammer and cutter for opening bitumen drums
(6) 30 and 15 litre capacity containers for measuring the aggregates
(7) Shovels
(8) Spades
(9) Rakes (small) with short handles
(10) Rakes (big) with long handles for levelling of mix
(11) Spring balances (10 kg and 25 kg)
(12) Bitumen boiler (preferably oil fired with pressure burner)
(13) Chain pulley arrangement for lifting of drums
(14) Tractor or other arrangement to pull the bitumen boiler.

E. Tools and appliances for checking the accuracy of the work

(1) Thermometres, dial type 0°—250°C, long lead mercury in steel
(2) Thermometres, mercury in glass type, 0°—250°C
(3) Straight edge (3 metre)
(4) Unevenness indicator (Optional)
(5) Camber board
(6) Depth gauge

F. Tools and appliances for safety during construction

(1) Road barriers
(2) Diversion boards
(3) Caution boards
(4) Red flags
(5) Red lamps
(6) Field tent and accessories
(7) Gumboots
(8) Gloves
(9) Goggles
(10) Firstaid box

4. SPECIAL EQUIPMENT FOR SURFACE DRESSING

4.1. Manual Methods

Where surface dressing is done by manual methods the following equipment are needed:

(1) Manually operated sprayers
(2) Three-wheel steel roller, 6-8 tonnes, or alternatively smooth pneumatic tyred roller.

4.2. **Mechanized methods**

Where surface dressing is done by purely mechanized methods, the following equipment are needed:

(1) Self-propelled bitumen pressure distributor
(2) Gritter
(3) Three-wheel steel roller, 6-8 tonnes, or alternatively, smooth pneumatic tyred roller.

5. SPECIAL EQUIPMENT FOR PREMIX-CARPET AND PREMIX SEAL COAT

For premix carpet seal coat works, the following special equipment are needed:

(1) Manually operated sprayers (manual method)

or

Self-propelled bitumen pressure distributor (mechanised method), (for tack coat application)

(2) Hand operated drum mixers (for small jobs)

or

Cold mixing plant (where cold mixing is permitted)

or

Mixing plant with arrangement for drying of aggregates

(3) Steel-tyred three-wheel/tandem roller, 8-10 tonne or smooth pneumatic tyred roller.

6. SPECIAL EQUIPMENT FOR HOT-MIX CONSTRUCTIONS LIKE BITUMINOUS MACADAM, SEMI-DENSE CARPET AND ASPHALTIC CONCRETE

The following special equipment is needed for hot-mix constructions work:

(1) Sprayer unit, such as a self-propelled bitumen pressure distributor or manually operated sprayer unit (for tack coat application)
(2) Hot mix plant of adequate capacity with arrangement for heating/batching/mixing and storage
(3) Tripper trucks for transport of mix
(4) Paver finisher
(5) Road rollers
    (a) Three-wheel Steel roller—8-10 tonne capacity or pneumatic, smooth wheel roller 15-20 tonne for break-down rolling
    (b) Tandem steel wheel roller, 8-10 tonne for final rolling.

7. SPECIAL EQUIPMENT FOR PENETRATION MACADAM AND BUILT-UP SPRAY GROUT

The following special equipment is needed for penetration

macadam and built up spray grout:

(1) Sprayer unit, such as a self-propelled bitumen pressure distributor or manually operated sprayer unit
(2) Three-wheel steel tyred roller, 8-10 tonne.

8. SPECIAL EQUIPMENT FOR MASTIC ASPHALT

The following special equipment is needed for mastic asphalt:

(1) Mastic cooker of adequate capacity with arrangement for heating and mixing of aggregates with bitumen
(2) Wooden floats of suitable design.

9. FIELD LABORATORY

Apart from the equipment, tools and appliances required for construction as mentioned above, it will be necessary to set up a well-equipped field laboratory for regularly carrying out quality control and acceptance tests. Equipment required for such a laboratory are listed in *Appendix*.

10. BRIEF DESCRIPTION OF TOOLS & EQUIPMENT AND THEIR SPECIFICATIONS

10.1. **Cleaning tools**

10.1.1. In all bituminous specifications, it will be necessary to clean the surface which is to receive the bituminous layers. The usual equipment needed for this operation are:

(i) Brooms
(ii) Wire brushes
(iii) Coir brushes
(iv) Gunny bags

The use of mechanical brooms is not yet widespread in this country and such brooms are not manufactured indigenously.

10.1.2. **Brooms:** Ordinary brooms made of coconut sticks/any other locally available material found suitable for sweeping layer of dust, leaves etc.

10.1.3. **Wire brushes:** Wire brushes as shown in Fig. 1 will be found useful for cleaning water bound macadam and bituminous surface which are badly rutted and worn out. These brushes will also be effective in removing caked mud, cow dung and similar extraneous matter.

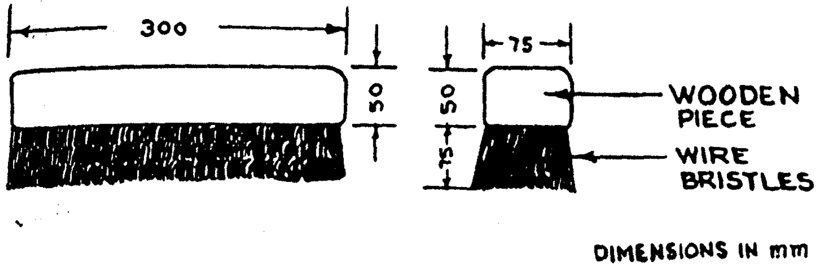


Fig. 1. Wire brush
Source—IS:898-1964

10.1.4. **Coir brushes:** Coir brushes as shown in Fig. 2 will be found useful for sweeping black-topped surfaces.

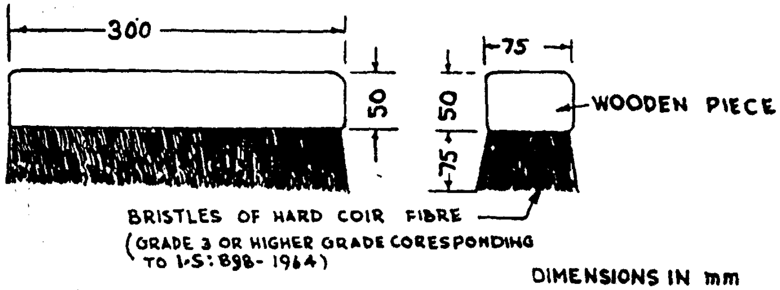


Fig. 2. Coir brush
Source-IS: 898—1964

The coir fibres should meet with the requirements of IS:898-1964 "Specifications for Coir Fibre" and be of Grade III or higher specification. The brushes should be replaced when the coir bristles wear down to a length of about 40 mm.

10.2. **Bitumen Heating and Handling Equipment**

10.2.1. **Equipment for bulk supply:** Bulk supply of bitumen has a number of advantages over supply in drums. These are:

(i) Saving in the cost of steel sheeting for manufacture of drums;
(ii) No wastage of bitumen on account of drums not being completely emptied at the time of filling the boiler; and
(iii) Savings in the cost of heating the bitumen and handling of drums.

In view of the above inherent advantages of bulk supply of bitumen, it is envisaged that this supply method will become increasingly popular in the country.

10.2.2. Where the demand for bitumen is fairly large, it would be desirable to establish bitumen storage depots which would receive and store bulk bitumen and distribute it to individual customers. Suggested layouts for storage depots are given in Figs. 3 and 4. These layouts should be considered as illustrative and may need alterations depending upon the actual site conditions.

10.2.3. **Bitumen tankers:** Bulk bitumen lorries are intended to transport bulk bitumen either directly from the refineries or from the bulk storage depots to the points of actual consumption. The lorries are fitted with tanks of 5,000 to 15,000 litre capacity and are used to operate within economic distance from the refineries and the storage depots. They are equipped with arrangements to maintain temperature of bitumen during transit. Oil fired heaters are used for this purpose. Unloading of the lorries is usually carried out by compressed air or gear pumps. A sketch of the bulk bitumen lorry is given in Fig. 5.

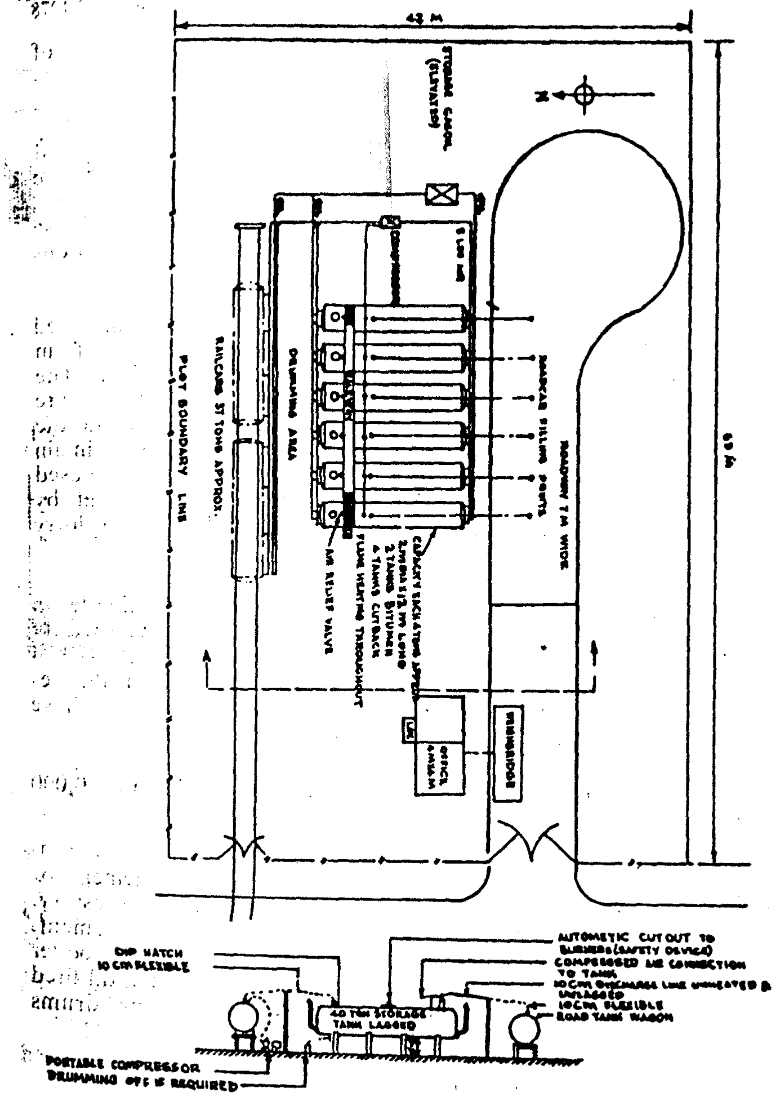
10.2.4. **Bitumen boilers:** Bitumen boilers are available in 100-10,000 litre capacity range. These are portable and are generally fired by oil, coal or fire-wood. The Indian Standard IS: 2094-1974 "Specification for Heaters for Tar and Bitumen" lays down the requirements of such boilers. The following nominal capacities have been prescribed in the above standard:

100, 300, 500, 1000, 1500, 2000, 3000, 5000, 7500 and 10,000 litres.

10.2.5. **Arrangement for lifting asphalt drums:** Where bitumen is supplied through packed steel-drums, an arrangement for lifting the drums to the inlet of bitumen boilers becomes necessary. This normally consists of a chain-pulley system. The arrangement, if it is an adjunct to the boiler, should be so designed that the boiler is stable under all normal working conditions. A small oil fired bitumen boiler with the chain pulley arrangement for lifting drums is depicted in Fig. 6.

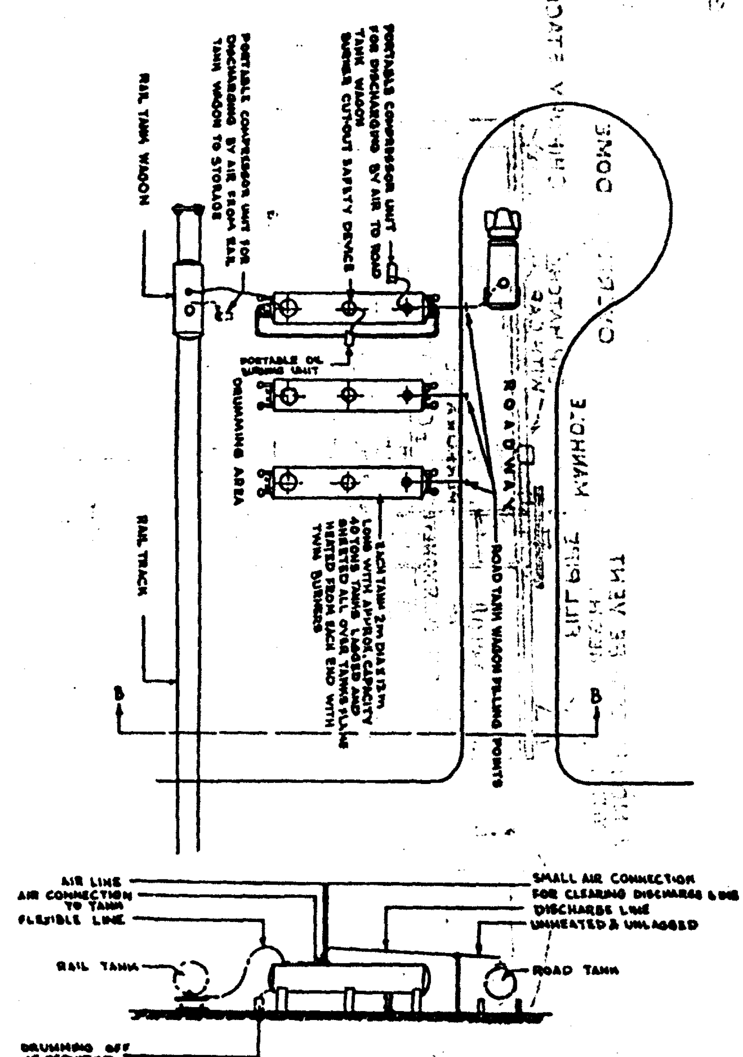
10.3. **Bitumen Spraying Equipment**

10.3.1. **Bitumen pressure distributors:** This is a mechanical equipment for spraying bitumen for surface dressing or grouting specifications, and is convenient to use in conjuntion with bulk bitumen



ELEVATION A-A

Fig. 3. Typical layout of bulk bitumen storing site (Example 1)
Source: M/s Bharat Petroleum Corporation Ltd.



ELEVATION B-B

Fig. 4. Typical layout of bulk bitumen storing site (Example 2)
Source: M/s Bharat Petroleum Corporation Ltd.

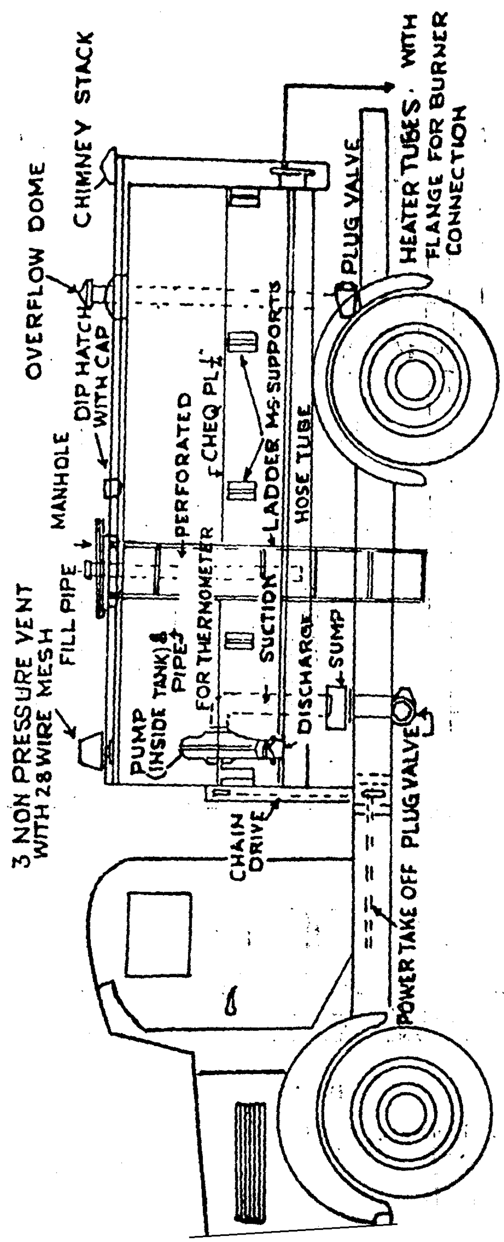


Fig. 5. Bulk bitumen Tanker
Source: M/s Hindustan Petroleum Corporation Ltd.

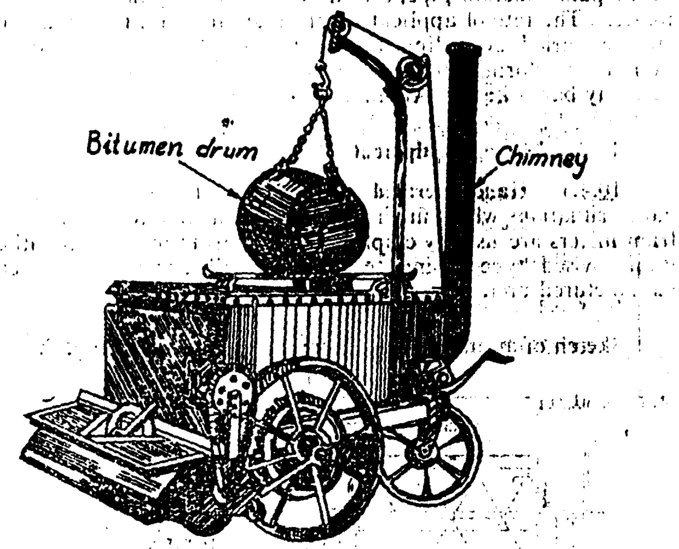


Fig. 6. An arrangement for lifting bitumen drums

supply. It consists of a pneumatic tyred lorry on which is mounted an insulated tank or a tank with a heating system, usually, oil fired burners, with direct heating from the flues passing through the tank. A power driven pump or a compressed air pump designed to handle bitumen is usually fitted to the vehicle. A spraybar unit with nozzles is fitted at the rear of the tank through which bitumen is applied under pressure on to the road surface. The quantity of bitumen flow is controlled by a metering device. The speed at which the vehicle is operated controls the rate of spread of the bitumen. Indian Standard, IS:2093-1974 "Specification for Distributors for Hot Tar and Bitumen" deals with the requirements of this equipment. The following sizes are listed in the I.S.:

1000, 1500, 2000, 3000, 5000, 7500, & 10,000 litres.

10.3.2. **Bitumen hand spraying equipment:** For small works not warranting the induction of a bitumen pressure distributor, the usual method of spraying bitumen on the road surface is by small pumps attached to the bitumen boiler itself. These pumps may be either mechanically operated on the pump suction pipe, a flexible pipe, a spray lance, and a spray nozzle. The rate of application of the bitumen on the road surface can be varied according to the height of the nozzle above the ground. Uniformity of spraying is controlled by the angle at which the spray bar is kept. An angle of 45° ensures uniform spraying.

10.4. Mixing Equipment

10.4.1. **Hand operated drum mixers:** For small premix jobs and in situations where mixing plant is not available, hand operated drum mixers are usually employed. The drum mixers can either be of improvised type (using second-hand oil drums) or a specially manufactured unit.

Sketch of an improvised drum mixer is given in Fig. 7.

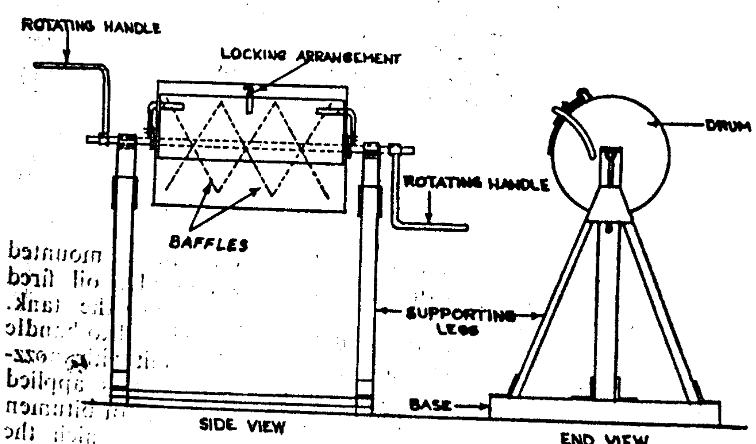


Fig. 7. Improvised drums mixer
Source : Bharat Petroleum Corporation Ltd.

The improvised drum mixer consists of a second-hand oil drum with baffles provided inside to facilitate the mixing of bitumen and aggregates. The drum is operated by rotating handles at either end and is supported on wodden or angle iron stands.

Specially manufactured hand-operated drum mixers are covered by IS:2434-1973, "Specification for Hand-operated Drum Asphalt Mixers". A sketch of a typical coal-fired hand-operated drum asphalt mixer recommended in the above standard is given in Fig. 8.

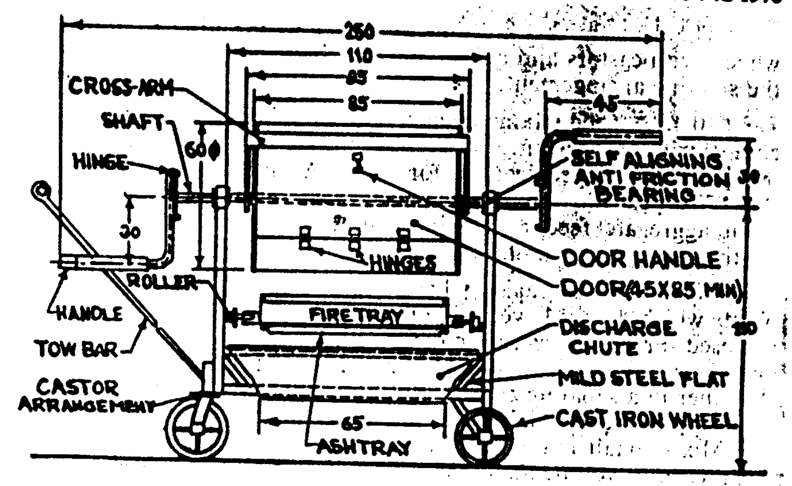


Fig. 8. Indian Standard hand drum mixer
Source : IS: 2434—1973

A sketch of a typical hand-operated combination type coal/oil fired, asphalt mixer also recommended in the Indian Standard is shown in Fig. 9.

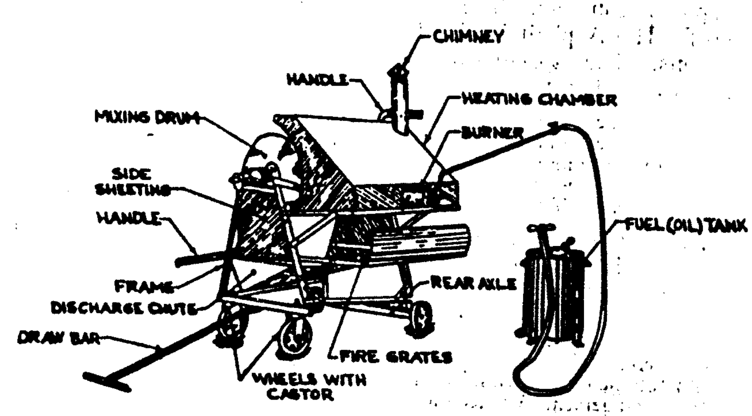


Fig. 9. Indian Standard hand operated combination type coal/oil fired asphalt mixer
Source : IS: 2434—1973

The drum mixers are sometimes made portable by providing wheels with castors and a draw bar for being pulled by a tractor or the suitable arrangement.

10.4.2. **Cold mixing plants**: Cold mixing plant is used where cold mixing is permitted by the specification. Indian Standard IS:5435-1969, "Specification for Cold Asphalt Macadam Mixing Plants" covers the requirements of such plants. The plant consists of an aggregate feeder, a mixing unit with elevator and bitumen heating and storage unit.

10.4.3. **Small capacity hot-mix plants**: Small capacity mixing plants which do not have elaborate system for heating the aggregates are used for premix work such as open graded carpets and bitumen macadam. Such plants have an arrangement for drying the aggregates either in a separate drier unit or in the mixing drum itself. The former type is covered by IS:5890-1970, "Specification for Mobile Hot Mix Asphalt Plant, Light Duty". This standard requires that a separate provision be made for a drier unit and in no case shall the mixing be carried out in the drier drum. The capacity of such plant as standardized by the IS is either 3-5 tonnes/hr. or 6-10 tonnes/hr.

Light duty plants where the mixing drum contains burners for drying the aggregates are also available in the country.

10.4.4. **Large capacity hot-mix plants**: Hot mix plants are required for major bituminous works such as bituminous macadam and asphaltic concrete. The Indian Standard IS:3066-1965 "Specifications for Hot Asphalt Mixing Plants" covers the requirements of such plants. The capacities of the plant as listed below are indicated in the above Indian Standard:

20-30 tonnes/hour
30-45 tonnes/hour
40-60 tonnes/hour
60-90 tonnes/hour
80-120 tonnes/hour

These plants can be either mobile or static. Further, these can be of the continuous type or batch type. The important components of the plant are :

(i) Arrangement for cold feed
(ii) Drier unit for heating the aggregates
(iii) Screens and bins for separating and storing
(iv) Bitumen heating and storage tank
(v) Arrangement for accurately proportioning each constituent such as the aggregate, filler and bitumen
(vi) A mechanical mixer

A typical arrangement of a batch-type hot mix plant is shown in Fig. 10.

Photo 1. Bitumen Boiler

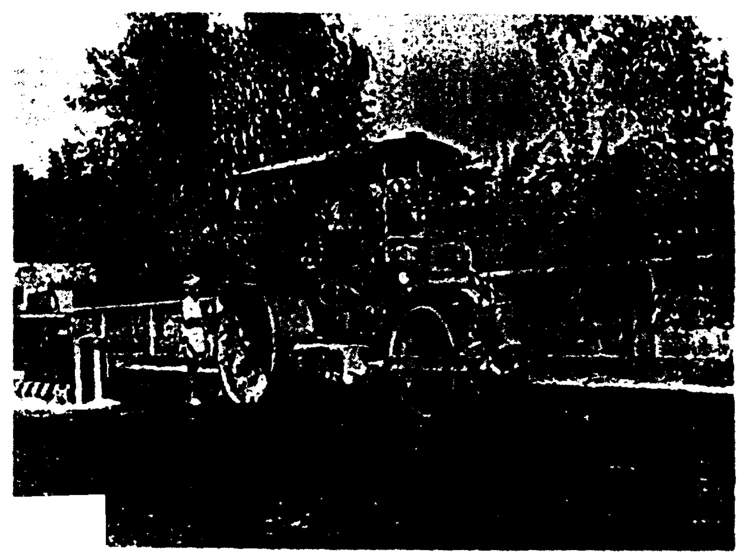


Photo 2. 3-Wheel Steel-tyred roller

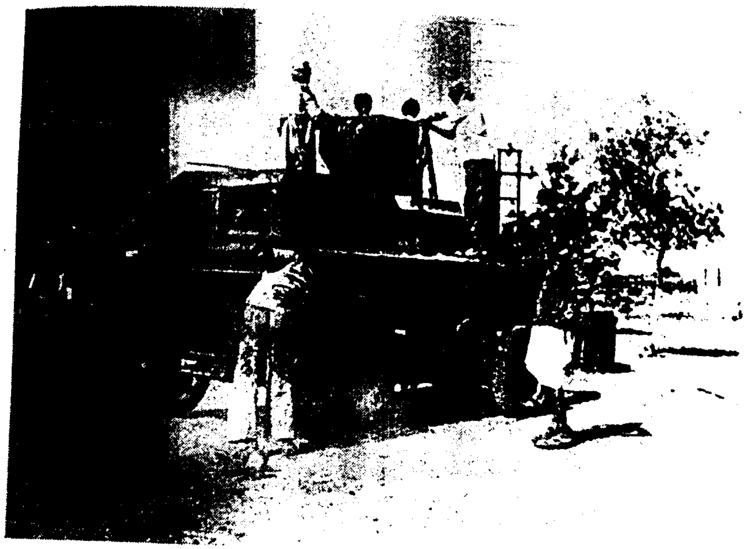


Photo 3. Cold mixing plant

Photo 4. Mini-hot-mix plant

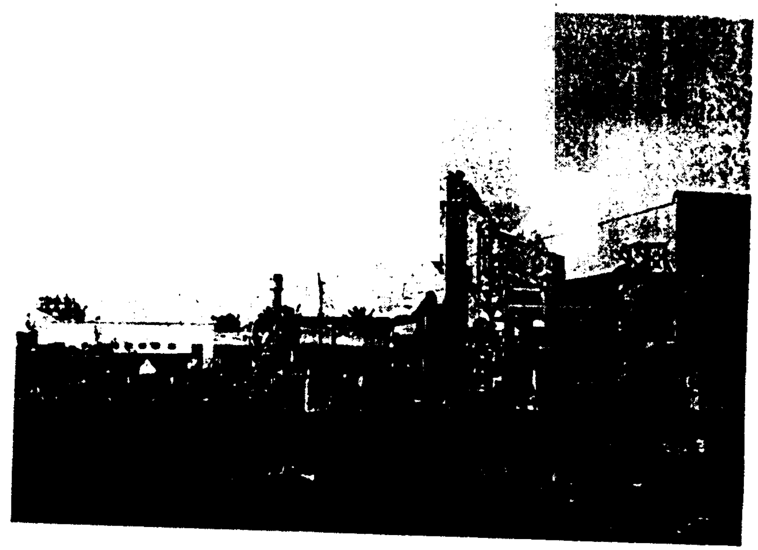


Photo 5. Hot-mix plant

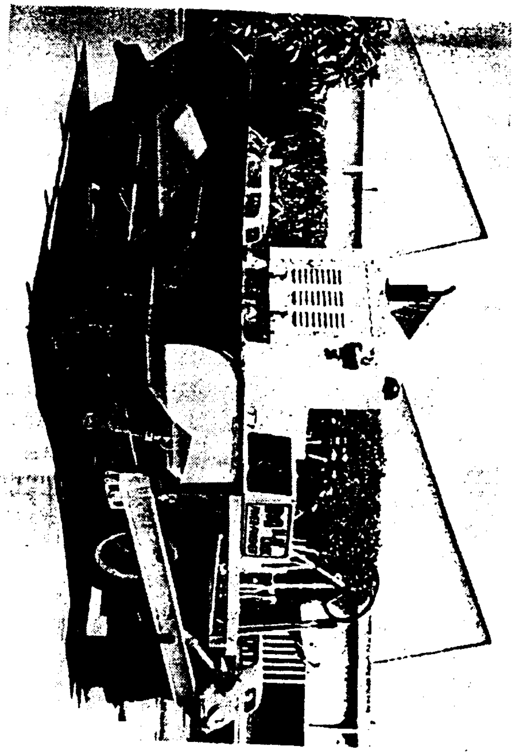


Photo 6. Paver Finisher

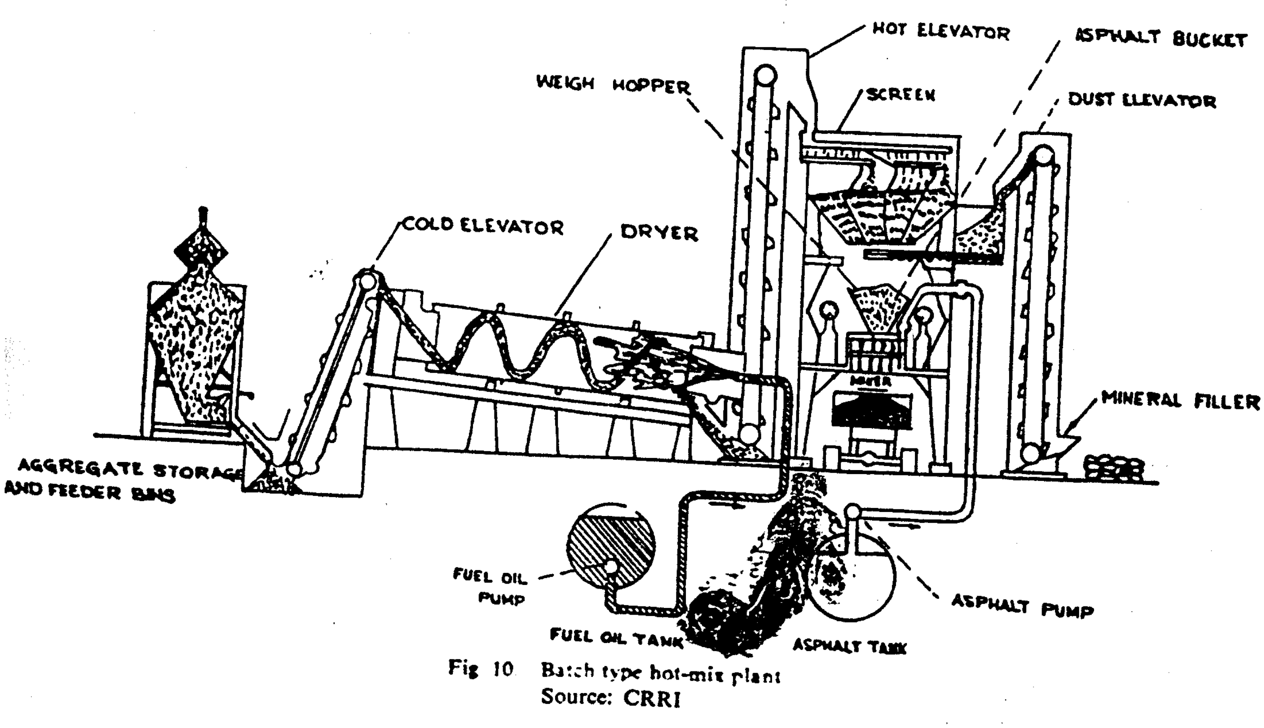


Fig 10 Batch type hot-mix plant
Source: CRRI

10.5. Equipment for Transport of Bituminous Mix

10.5.1. **Wheel barrows for small scale work**: The premixed material is conveyed from the mixer to the place where it is to be laid by means of wheel barrows. Wheel barrows can either be of single wheel type or two wheel type. The former is covered by IS: 2431-1963 "Specification for Steel Wheel Barrows (Single Wheel type)' and the latter by IS:4184-1967 "Specification for Steel Wheel Barrows (with two Wheels)". They are normally available in the following nominal capacities:

Single-wheel wheel barrows	:	60 and 85 litre
Two-wheel wheel barrows	:	75, 85, 110 and 140 litre

10.5.2. **Tipper trucks**: For large scale bituminous work, especially in conjunction with hot mix plants, it is necessary to use tipper trucks for conveying bituminous mixtures from the asphalt plants to paver finishers. These tipper trucks can conveniently empty the bituminous mixtures into the hoppers of paver finishers directly. Trucks of 5-7.5 tonne capacity are generally used. The parts of the truck which come in contact with the bituminous mixture should be clean, smooth and free from cracks and holes so as to prevent the liquid bitumen from flowing out. The trucks should be provided with tarpaulins to prevent loss of heat from bituminous mixture during transit in winter. Double-walled trucks are also used for this purpose.

10.6. Equipment for Spreading Stone Chips (Gritters)

For large scale bituminous work involving the spreading of aggregates, it is convenient to use some form of mechanical spreader also known as gritter. There are two types of aggregate spreaders available :

(i) Towed hopper type in which a large hopper with the spreading mechanism is towed by the bitumen distributor

(ii) Tail board type aggregate spreader which spreads the aggregates as they are discharged from a lorry travelling in reverse. This is illustrated in Fig. 11.

10.7. Equipment for Spreading and Laying Bituminous Courses

10.7.1. For efficient spreading and laying of bituminous courses modern specifications require the use of paver finishers. A schematic diagram of a paver finisher is given in Fig. 12.

IS:3251-1965, "Specification for Asphalt Paver Finisher" covers the requirements for this item of equipment. The paver finisher has a hopper into which the bituminous mix is dumped. The

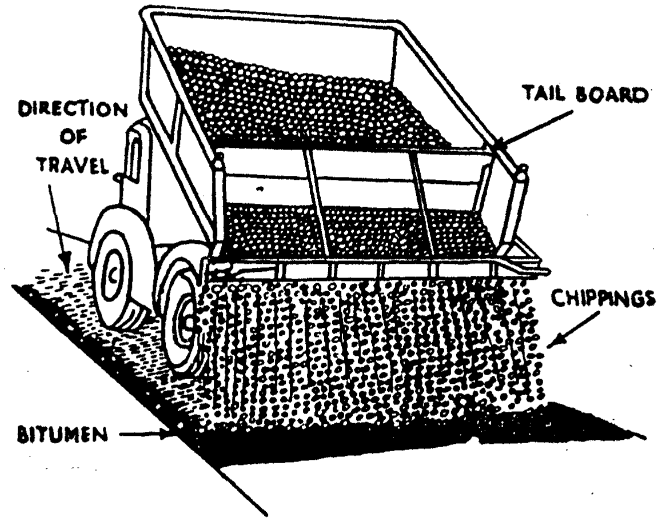


Fig. 11. Aggregate spreader

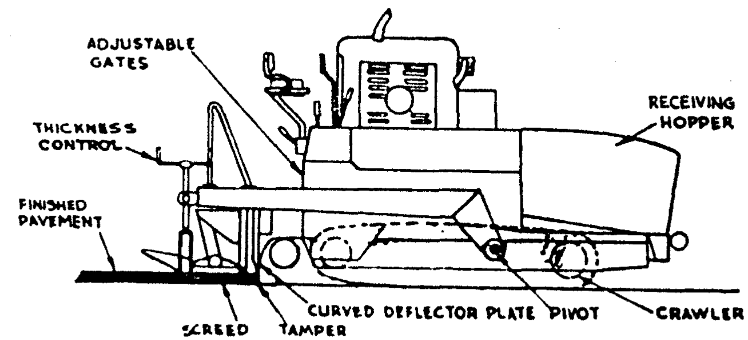


Fig. 12. Paver finisher

hopper has an adjustable opening at the bottom through which the mix can flow out on to the road surface as the paver moves forward. The equipment has a screed board attached to it which strikes off and imparts compaction to the mix. In order to prevent the mix from sticking to the screed plate, arrangement is also provided for heating the same at the start of the operations on a cold day.

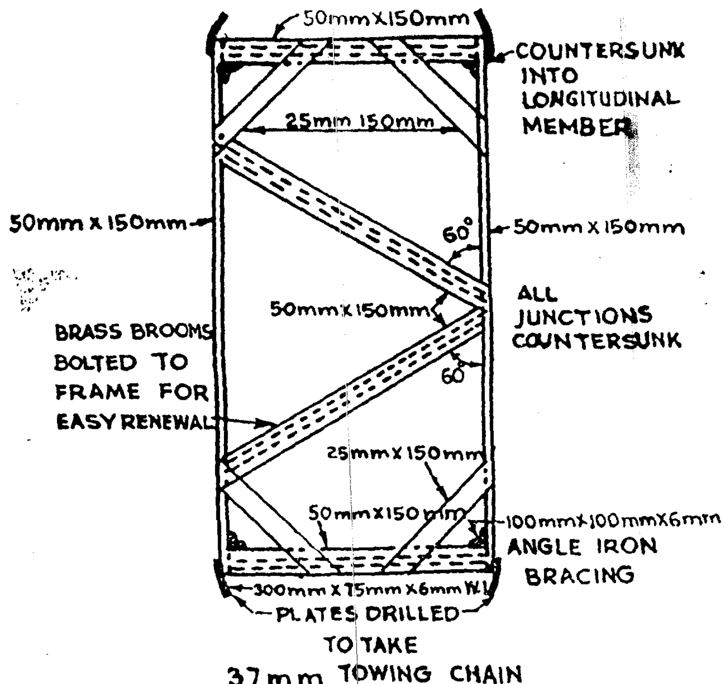


Fig. 13. Drag broom 3.75 m to 5 m long 1.8 m to 2.8 m wide frame of teak or pine position of brushes shown by = =

10.7.2. **Equipment for manual spreading**: When paver finishers are not available for laying the bituminous course, manual method of laying is resorted to. A drag spreader can be used with advantage for hand spreading. A sketch of suitable drag broom is given in Fig. 13.

Small tools such as camber board, templates, shovels, spades and rakes are also needed for hand spreading. IS:274-1966, "Specification for General purpose Shovels" and IS:1759-1961, "Specification for Powrahs" deal with shovels and powrahs may be referred to in this connection.

Fig. 14 gives a sketch for a rake with a long handle for spreading the bituminous mix manually.

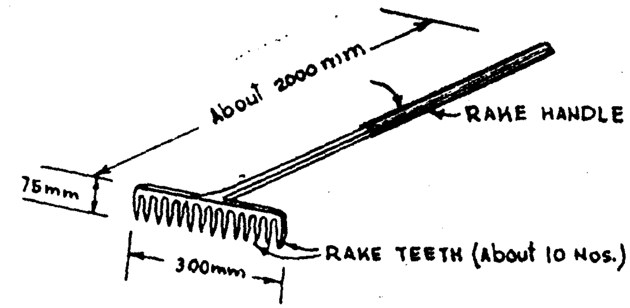


Fig. 14. Rake

In addition to the above, gumboots and gloves will be needed for the workmen handling bituminous mixtures.

10.8. **Equipment for Mastic Asphalt**

A mastic cooker is needed to prepare mastic asphalt. This cooks the aggregate and bitumen mixture to temperatures of 170 to 180 degrees centigrade. An arrangement for agitating the mix is provided inside the cooker. The period of cooking is generally about three hours for ensuring a thorough mix.

10.9. **Rolling Equipment**

10.9.1. **Three-wheel roller**: The standard equipment for break down rolling of asphaltic concrete, premix carpets and bituminous macadam is a three-wheeled steel roller 8/10 tonne and for rolling surface-dressing 6-8 tons. IS:5502-1969 "Standard Specifications for Smooth-Wheeled Diesel Road Rollers" covers the requirement of this item of equipment.

10.9.2. **Tandem roller**: Tandem rollers of 8/10 tonne capacity are needed for final finish rolling of asphaltic concrete and similar surfaces. IS:5502-1969 "Standard Specifications for Smooth-Wheeled Diesel Road Rollers" covers the requirement.

10.9.3. **Pneumatic tyred rollers**: These rollers consist of one or two axles on each of which is fitted a number of smooth pneumatic tyred wheels. Above the wheels is a flat bed or a hopper upon which weights are placed as ballast. The weight of the rollers can be varied by adjusting the ballast. The rollers may be either self propelled or towed by a tractor. They are used for intermediate rolling of asphaltic concrete courses. A capacity of 15-20 tonne, with tyre pressure of 5-2-7.0 kg/cm² will be suitable.

10.10. Tools for Checking Surface Evenness

10.10.1. **Camber board/template:** Two designs for camber board/templates for checking the cross-profile of road surface are given in Figs. 15 & 16.

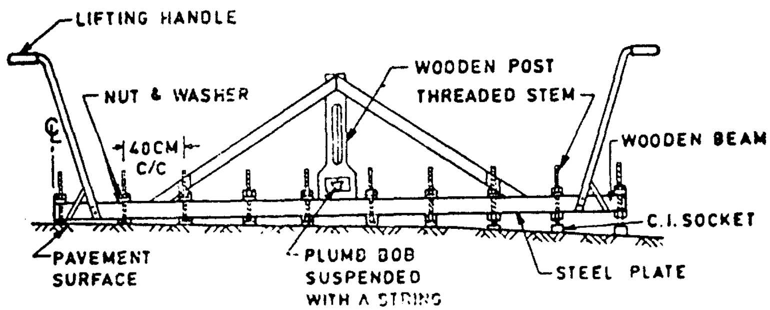


Fig. 15. One design of template with adjustable profile
Source: IRC : SP 11-1977

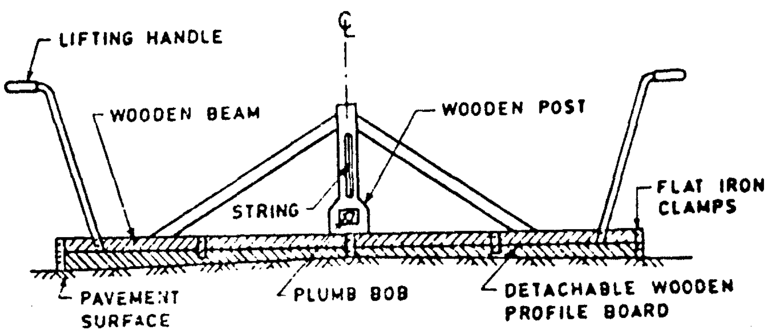


Fig. 16. Another design of template with adjustable profile
Source: IRC : SP 11-1977

10.10.2. **Straight edge:** For controlling the surface evenness of roads, a three-metre straight edge is needed. Fig. 17 gives suitable design for straight edge and graduated wedge.

10.10.3. For method of use of camber board and straight edge, reference may be made to Chapter 7 of IRC: SP 11-1977 "Handbook of Quality Control for Construction of Roads and Runways".

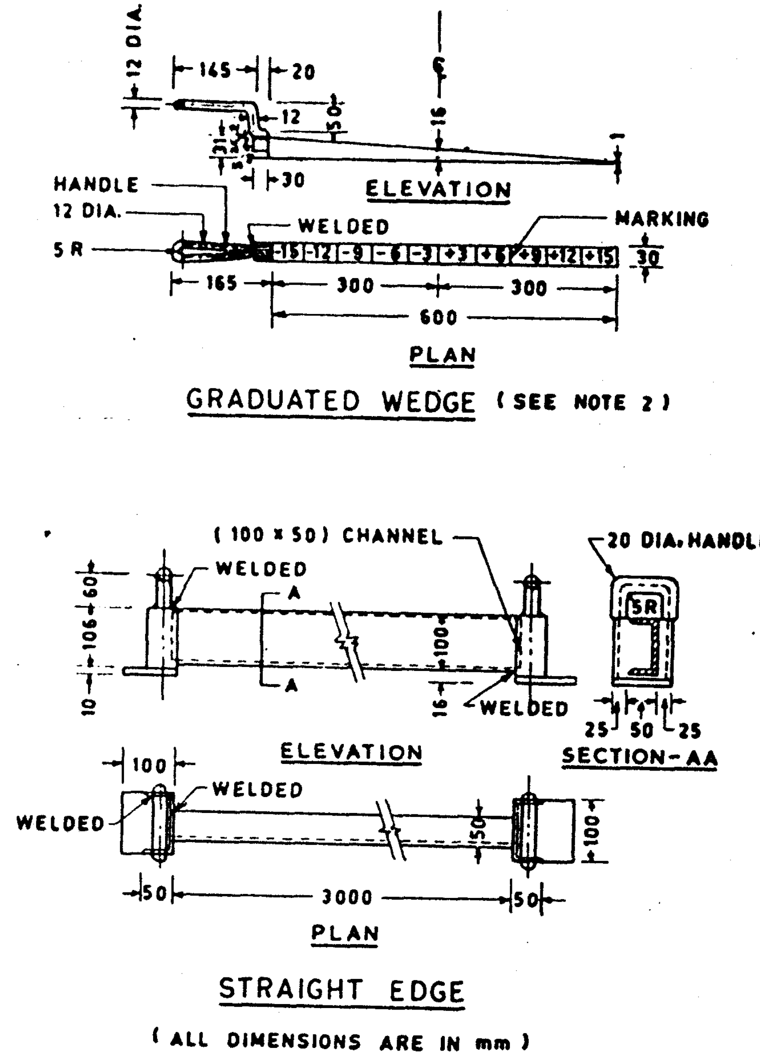


Fig. 17. Typical design plan straight edge of graduated wing and straight edge
Source: IRC : SP 11-1977

10.10.4. Unevenness Indicator: The unevenness Indicator, developed indigenously, is a travelling straight-edge type device, which, when pre-set according to the specification for the surface under check, performs the following functions on being run by two workmen along the lines of measurement at a walking speed of about 5 km per hour :

(i) Instantaneously indicates the size of irregularity through a pointer moving on a graduated dial, to an enlarged scale

(ii) Sounds a buzzer at locations where the irregularity is in excess of the maximum permissible (as pre-set)

(iii) Automatically marks, through colour spray, locations where the irregularity is in excess of the permissible maximum (as pre-set).

11. CARE IN OPERATION AND UPKEEP OF EQUIPMENT

11.1. The efficient operation of equipment and tools and their longevity depends to a large extent on observing set rules and procedures for handling and maintaining them. While suppliers of equipment normally indicate detailed procedures for the maintenance and operation of equipment, a few simple rules for the guidance of the field engineers are set forth below :

11.2. Bitumen Boilers

(i) When the boiler is empty, it should be thoroughly cleaned of all foreign matter before it cools down and the thin bitumen sets and becomes hard. This procedure for cleaning should be followed every day.

(ii) The portable boilers should be handled very carefully while being towed especially when loaded. They should not be towed by fast moving vehicles.

(iii) The wheels and the pivoting carriage should be oiled daily.

(iv) Before starting to heat the boiler in the morning, the inside should be inspected for the presence of any water which might have collected during the night. If water is present, the same should be dried before bitumen is poured in. The fire in the fire box should be lit only when there is at least some quantity of bitumen in the boiler.

(v) Boiler should never be completely filled with bitumen. The bitumen level should be always at least 15 cm lower.

(vi) The bitumen should be stirred from time to time with a long pole to ensure even distribution of heat.

11.3. Bitumen Hand Spraying Equipment

(i) The strainers inside the bitumen boilers should be inspected to see that they are not blocked. If so, they should be cleaned with a blow-lamp and kerosene.

(ii) The sprayer nozzle should be cleaned before and after each day's work.

(iii) Pumping should be started only after the bitumen has reached the appropriate application temperature.

(iv) The flexible hose connecting the heater with the nozzle should be inspected for any obstruction. The best way of clearing the obstruction is to blow hot air through the hose.

(v) The spray lance should be checked for cleanliness and if it is blocked with bitumen, it should be cleaned with kerosene.

(vi) As soon as pumping is stopped, the hose and the spraying parts should be hung up in a vertical position so that bitumen is drained out.

(vii) Under no circumstances should the hose pipe be disconnected if there is any pressure registered on the pressure gague as this may result in bitumen being splashed out and somebody being burnt.

(viii) As soon as spraying commences, the fire should be made low. Otherwise there is danger of overheating the bitumen and also burning the plates of the heater.

(ix) Once every month the whole pumping unit should be taken out and thoroughly cleaned with kerosene and the glands, gaskets and washers examined and replaced, if necessary.

11.4. Hand Operated Drum Mixers

(i) The drum mixers should not be towed by fast moving vehicles.

(ii) The wheels should be cleaned daily before and after use.

(iii) The shaft bearing on which the drum rotates should be lubricated regularly.

(iv) After each day's work, the mixer must be cleaned and the fire grate washed.

(v) The bolts and nuts of the vanes inside the mixer should be checked periodically for tightness. The hinges of the door of the mixer should be examined and lubricated, if necessary.

11.5. Mixing Plants of Small Capacity

(i) Adequate attention should be paid for lubricating the driving shafts, bearings, chains, hinged joints and all moving parts.

(ii) Lubricating oils and grease should be replaced periodically.

(iii) Linings inside the mixer should be examined and it should be ensured that they are not worn out.

(iv) The mixer plates should be examined and it should be ensured that the bolts are in tight position.

(v) The sliding door should be examined for ease of operation and any sticking bitumen. It should be cleared periodically.

(vi) The gear wheels provided for mobility of the equipment should be checked and lubricated periodically.

(vii) The mixer box must be cleaned thoroughly after each working day.

(viii) If a burner is provided inside the mixing drums, the same should be cleaned before and after each day's work. Nozzles should not be tampered with.

(ix) The burner should always be kept free of dust.

11.6. Hot Mix Plant

(i) The cold aggregate feeder controls must be set accurately and the setting should be checked before commencing each day's work.

(ii) The burners provided in the drier unit should be kept free of dust.

(iii) The sprayer nozzle holes of the burner unit should be cleaned before and after work. Nozzle should not be tampered with.

(iv) The temperature indicating devices inside the drier unit should be checked for accuracy periodically.

(v) The screens should be cleaned after each day's work and any worn out or broken screens replaced immediately.

(vi) The overflow vents from the hot-bins should be checked to ensure that they are clear of obstructions.

(vii) The pipelines connecting the hot bitumen heaters and the bitumen pumps should be cleaned of sticking bitumen periodically.

(viii) The asphalt heating units should be inspected and cleaned of any foreign matter.

(ix) Different parts of the mixer such as rotating blades and the lining should be periodically cleaned.

(x) The weighing mechanism should be checked for accuracy periodically.

(xi) The doors should be cleaned of all foreign matter and the hinges lubricated adequately.

(xii) All driving shafts, bearings, driven chains, hinged joints, moving parts should be lubricated periodically.

11.7. Paver Finishers

(i) The spreading screens in the hopper feeder should be checked for excessive wear and proper operation.

(ii) On pneumatic tyred machines, the air pressure in the tyres should be checked and it should be ensured that this pressure is maintained.

(iii) On crawler mounted machines, the crawler mechanism should be checked periodically.

(iv) The flow control gate at the bottom of the hopper should be checked regularly for adjustment.

(v) The heaters should not be used to heat the mix being delivered to the paver. These are intended essentially to heat the screed plate at the start of operations on a cool day.

(vi) The screed plate should be checked periodically for excessive wear.

(vii) At the end of the working day while the machine is still warm, hopper, feeders, spreading, screws, template boards and screed plate should all be cleaned with a petroleum distillate to ensure smooth start-up on next day.

(viii) All the parts of the engine should be kept in proper working order. Lubricating oils should be replaced periodically. The water tank should be checked for its level.

(ix) The oil filter should be cleaned periodically.

11.8. Rollers

(i) The wear of the wheel rims should be checked. The surface of the steel tyres should be checked periodically for any depressions or grooves which are likely to impair the smooth surface.

(ii) Bearings of the wheels should be checked for excessive wear and in case they are excessively worn out, they should be replaced before the rolling operations begin.

(iii) Every day the engine oil level should be made up. The tension in the fan belt should be checked and corrected. All grease points should be lubricated.

(iv) Once a week the fuel filter and the base of the air filter should be cleaned. Distilled water level in the battery

should be made up. All bolts, nuts and screws should be checked and the grease and oil points should be lubricated.

(v) Once a month, the engine pump should be drained and refilled. The lubricating oil filter should be cleaned periodically and new element fitted in.

(vi) The wheels of the roller should be free from caked mud and or other adhering matter.

(vii) The watering pipes over the wheels should be in working order and it should be ensured that there is enough water in the storage tank of the roller.

Appendix

RANGE OF EQUIPMENT REQUIRED FOR FIELD LABORATORY FOR BITUMINOUS WORKS

A. **General items**

1. **Balances**
   - (i) 7 kg to 10 kg capacity, self indicating type, accuracy 1 gm
   - (ii) Pan balance, 5 kg capacity
   - (iii) Chemical balance, 500 gm capacity, accuracy 0.0001 gm
2. Oven, Thermostatically controlled, upto 200°C
3. Riftle type sampler for coarse and fine aggregates
4. Sieves as per IS: 460-1962
   - (i) IS sieves—450 mm internal dia of sizes 100 mm, 80 mm, 63 mm, 50 mm, 40 mm, 25 mm, 20 mm, 12.5 mm, 10 mm, 6.3 mm, 4.75 mm, complete with lid and pan
   - (ii) IS sieves—200 mm internal dia (Brass frame) consisting of 2.36 mm, 1.18 mm, 600 microns, 300 microns, 212 microns, 150 microns and 75 microns with lid and pan.
5. Sieve shaker capable of taking 200 mm and 450 mm dia sieves.
6. Thermometers (glass).
   - (i) 0°—110°C range, six nos.
   - (ii) 0°—250°C range, six nos.
7. Thermometer, dial type
   - (i) 0°—250°C, two nos.
   - (ii) 0°—250°C, with sufficient lead for use in boiler, one no.
8. Kerosene/gas stove
9. Sand pouring cylinder with conical funnel and tap.

B. **Items for testing aggregates**

1. Aggregate impact testing machine
2. Flakiness index test apparatus
3. Specific gravity bottle (for fine aggregate)
4. Cylindrical wire cage, 10 cm dia and 15 cm high, 6 mm mesh for specific gravity test on coarse aggregate
5. Standard cylindrical measures of 30, 15 and 3 litres capacity with standard tampering rod 60 cm length, 16 mm dia and bullet nosed for bulk density determination.

C. **Items for testing bitumen and bitumen mixes**

1. Penetration test apparatus

**2. Centrifuge extractor for bitumen content test

***3. Constant temperature bath, thermostatically controlled

***4. Marshall test apparatus complete with moulds, compactor, loading/measuring units etc.

D. **Miscellaneous items**

1. Enamel trays of various sizes, caseroles (Porcelain) spatulas, tongs, tripods, hammer, clamps, rubber gloves, sample tins etc.
2. Kerosene oil, Commercial benyene, distilled water.
3. Grease, Cotton waste, paint, chalk etc.

** Required only for premix type constructions

*** These are for Marshall test, required only where the construction involves