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# STRUCTURAL DESIGN OF RAILWAY BRIDGES

Date: 06 February 2022

Session 44 : Bridge 6-Railway Bridge

SMEC
Member of the Surbana Jurong Group

ACE Consultants Ltd.

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AGM Design, ACE Consultants Ltd. Bangladesh

*S-8A SEng PRP Training Program*

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## Rail Lines Owner in Bangladesh


Logo of Dhaka Mass Transit Company Limited (DMTCL)

DHAKA MASS TRANSIT COMPANY LIMITED
Road Transport and Highways Division
Ministry of Road Transport and Bridges

- ❑ Bangladesh Railway (BR), Ministry of Railway
- ❑ Dhaka Mass Transit Company Limited (DMTCL) for MRT Line Ministry of Road Transport and Bridges

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## Course Contents on Structural Design of Railway Bridges

- Railway Bridge under Bangladesh Railway, Ministry of Railway, GoB.
- Design Criteria as per TOR / Clients Requirement
- Types of Railway Bridge used in Bangladesh
- Loads on bridge Superstructures (IRS Bridge Rules)
- Seismic Load (IRS Seismic Code)
- Loads of bridge Substructures (IRS Substructures Rules)
- IRS Steel Bridge Code
- Fatigue Requirement for Steel Bridges
- Analysis and Design for Steel Stringer of STPG

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## Standards Requirement for BR

- ❑ General Standards for Alignment, Embankment, Track, Bridge etc. (From TOR)
  - ❑ Bangladesh Railway (BR)
  - ❑ Indian Railway Standards (IRS)
  - ❑ Indian Standards (IS)
  - ❑ American Society for Testing Materials (ASTM)
  - ❑ International Union for Railway (UIC)
  - ❑ European (EN)
  - ❑ American Railway Engineering and Maintenance Association (AREMA)

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## Standards Requirement for BR

- General Standards for Alignment, Embankment, Track, Bridge etc. (From TOR)

Design parameters and criteria adopted for the design of rail alignment and track structure are based on:

- BRWWM (Bangladesh Railway Way and Works Manual, 2013);
- BR SOD BG (Bangladesh Railway Schedule of Dimensions for BG);
- BR SOD MG (Bangladesh Railway Schedule of Dimensions for MG);
- BRLWRM (Bangladesh Railway Manual of Instructions on Long Welded Rails, 2013);
- Instructions and policy circulars issued by Bangladesh Railway;
- Practices adopted by Indian and other Railways; and
- Modifications to suit DG track.



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## Standards Requirement for BR

- Standards for Bridges and Culverts Design (From TOR)

a) IRS Standard-Rules Specifying the Loads for Design of Super-structure and Sub-structure of Bridges (Bridge Rules);
b) IRS Standard-Code of Practice for the Design of Steel or Wrought Iron Bridges Carrying Rail, Road or Pedestrian Traffic (Steel Bridge Code);
c) IRS Standard-Code of Practice for Metal Arc Welding in Structural Steel Bridges Carrying Rail, Rail-cum-Road or Pedestrian Traffic (Welded Bridge Code);
d) IRS Standard-Code of Practice for Plain, Reinforced and Pre-stressed Concrete For General Bridge Construction (Concrete Bridge Code);
e) IRS Standard-Code of Practice for The Design of Sub-Structures and Foundations of Bridges (Bridge Sub-structures and Foundation Code);
f) Indian Railways Bridge Manual-1998; and
g) Indian Roads Congress (IRC) Standard: 89-1997 Guidelines for Design and Construction of River Training and Control Works for Road Bridges.



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# Standards Requirement for BR

## ❑ Standards for Bridges and Culverts

Railway Master Plan. All bridges, culverts, and substructures have to be designed in accordance with modified BGML loading (IRS 2008, 25-ton axle load) considering double stacking of containers, future electric traction lines, and oversized consignments

### 1.3.1 Design Life

In accordance with the design codes, all structures shall be designed for a design life of 100 years.

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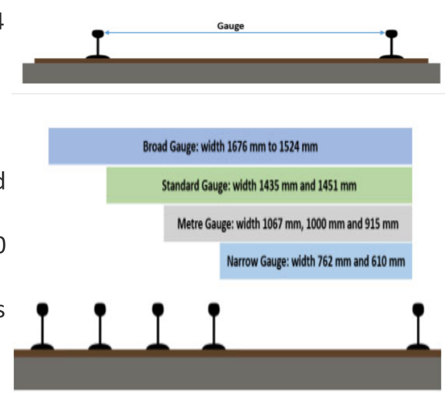
# Track Gauge for BR Different Types

- Broad Gauge : width 1676 mm to 1524 mm or 5'6" to 5'0" (BR)
- Standard Gauge: width 1435 mm and 1451 mm or 4'-8½" (DMRT)
- Metre Gauge: width 1067 mm, 1000 mm and 915 mm or 3'-6", 3'-33/8" and 3'-0" (BR)
- Narrow Gauge: width 762 mm and 610 mm or 2'-6" and 2'-0".
- Dual Gauge: BG+MG combined (3 Rails with 1 common rail) (BR)




Source Internet

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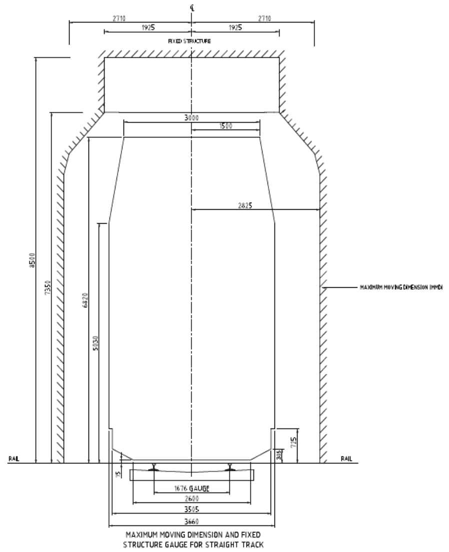
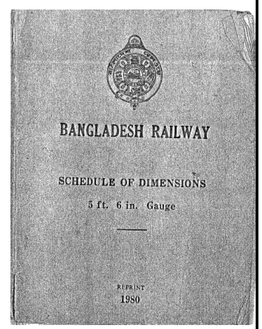
## Schedule of Dimension for BR BG Track Bridge

- Horizontal clearance 2825*2= 5650mm
- Vertical Clearance 7350mm
- Viaduct 8500mm

(BR Circular)



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## Types of Railway Bridge

- Types are:
  - Minor Bridge
  - Major Bridge
  - Important Bridge
  - Box Culvert
  - DTPG
  - STPG
  - TT (Open Web)

**2.12 MAJOR BRIDGES** are those which have either a total waterway of 18m or more or which have a clear opening of 12m or more in any one span.

**2.11 IMPORTANT BRIDGES** are those having a lineal waterway of 300m or a total waterway of 1000 Sq.m or more and those classified as important by the Chief Engineer/ Chief Bridge Engineer, depending on considerations such as depth of waterway, extent of river training works and maintenance problems.

Source: IRS Substructure Rules



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## Types of Railway Bridge
## Box Culverts

- Span Length 2m to 6m and Length up to 15m multi cell box culvert
- Used for Shallow depth, no navigation required
- Less cost
- Smooth riding as like
- Embankment

Source: Internet

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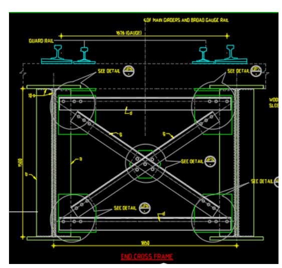
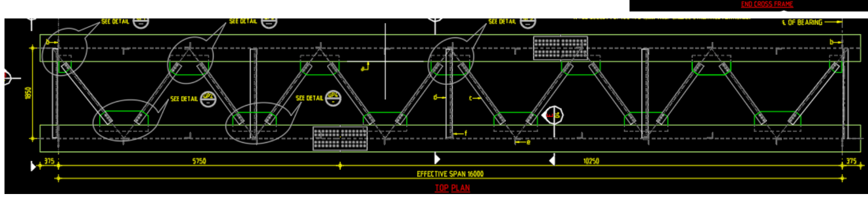
## Types of Railway Bridge
## Deck Type Plate Girder (DTPG)

- Rail line is above the steel Girder
- Span length from 10m to 20m
- Not suitable where navigation is required
- Less cost than that of STPG
- It can be open deck or Ballasted Deck





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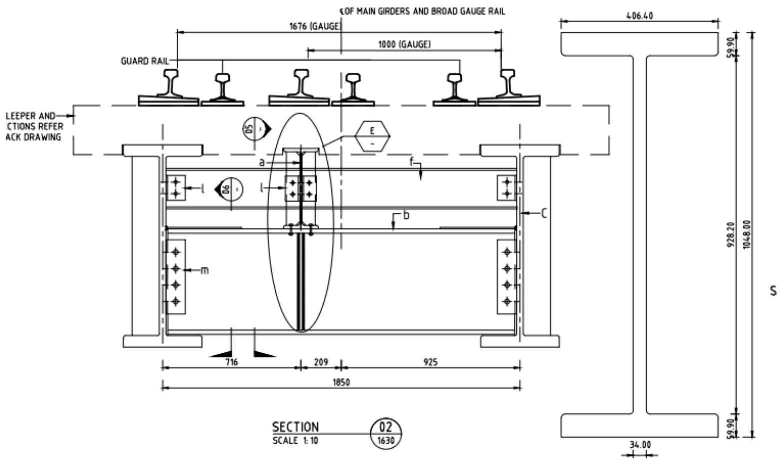
# Types of Railway Bridge
## Deck Type Plate Girder (DTPG)

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# Types of Railway Bridge
## Deck Type Plate Girder (DTPG)

Fabrication of DTPG in India for KGTP and MDP

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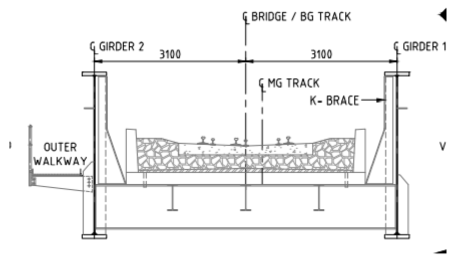
## Types of Railway Bridge
## Semi-Through Plate Girder (STPG)

- Rail Line passes inside the girder depth
- Span kept normally from 15m to 35m
- Suitable for navigable stream
- Need to follow the schedule of dimension
- Can be open deck or ballasted deck
- No Extra sound
and Riding smooth
for ballasted deck

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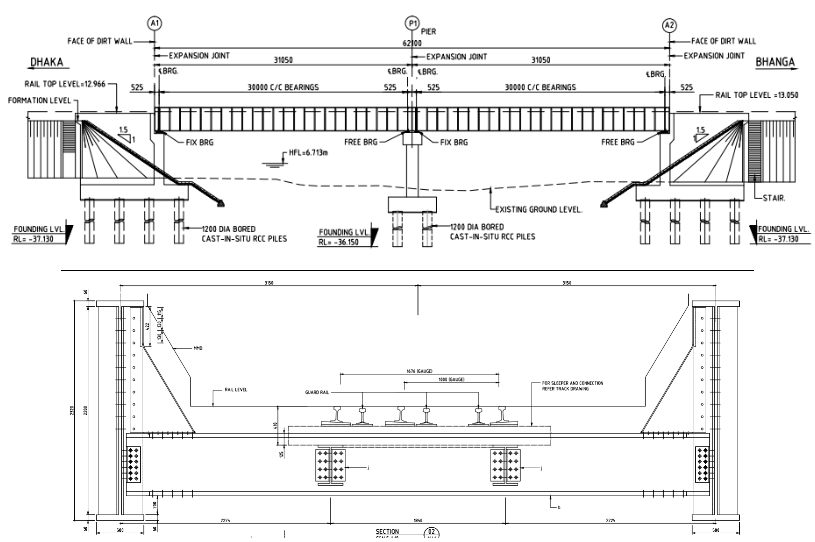
## Types of Railway Bridge
## Semi-Through Plate Girder (STPG)

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## Types of Railway Bridge
## Semi-Through Plate Girder (STPG)

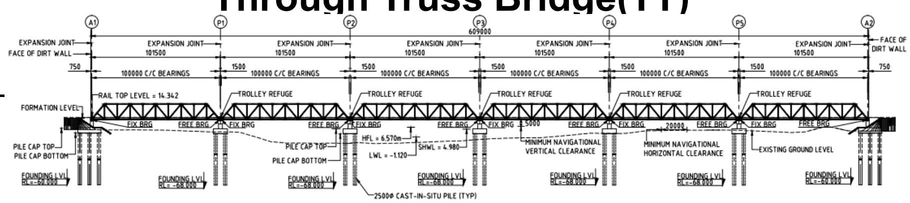
Semi Through Plate Girder
RBCP Parbatipur- Panchagarh
Under Construction Line (Opened now)

Fabricated at Macdonalds, Gazipur 35m span STPG Under Fabrication

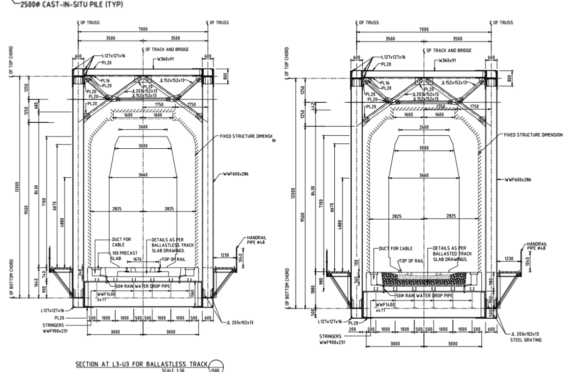




## Types of Railway Bridge
## Through Truss Bridge(TT)



- Span 35m to 100m or more,
- Long span needs for Navigation Clearance,
- It can be ballasted, Ballast less, Open deck.



## Types of Railway Bridge
## Through Truss Bridge(TT)

TT Bridge Fabricated in China
Weathered Steel on Feni River

TT Bridge Fabricated in India
For KGTP

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## Types of Railway Bridge
## Through Truss Bridge(TT)

TT Bridge Padma Rail Link Project

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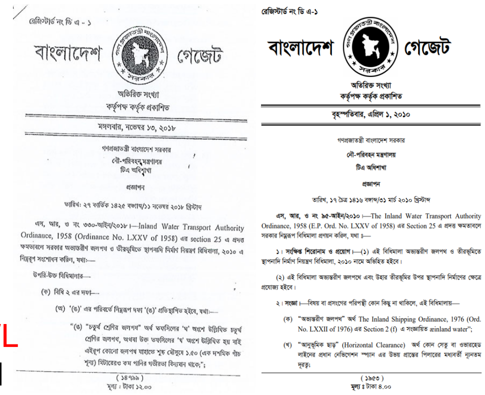
## Navigation Clearance
## BIWTA

(ক) প্রথম শ্রেণীর জলপথের ক্ষেত্রে উল্লম্ব ছাড়ের (Vertical Clearance) পরিমাপ হইবে ন্যূনতম ১৮.৩০ মিটার এবং আনুভূমিক ছাড়ের (Horizontal Clearance) পরিমাপ হইবে ন্যূনতম ৭৬.২২ মিটার;

(খ) দ্বিতীয় শ্রেণীর জলপথের ক্ষেত্রে উল্লম্ব ছাড়ের (Vertical Clearance) পরিমাপ হইবে ন্যূনতম ১২.২০ মিটার এবং আনুভূমিক ছাড়ের (Horizontal Clearance) পরিমাপ হইবে ন্যূনতম ৭৬.২২ মিটার;

(গ) তৃতীয় শ্রেণীর জলপথের ক্ষেত্রে উল্লম্ব ছাড়ের (Vertical Clearance) পরিমাপ হইবে ন্যূনতম ৭.৬২ মিটার এবং আনুভূমিক ছাড়ের (Horizontal Clearance) পরিমাপ হইবে ন্যূনতম ৩০.৪৮ মিটার;

(ঘ) চতুর্থ শ্রেণীর জলপথের ক্ষেত্রে উল্লম্ব ছাড়ের (Vertical Clearance) পরিমাপ হইবে ন্যূনতম ৫.০০ মিটার এবং আনুভূমিক ছাড়ের (Horizontal Clearance) পরিমাপ হইবে ন্যূনতম ২০.০০ মিটার;




- Vertical Clearance from SHWL (not HFL) to Girder soffit level
- NC Needs for span arrangement



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## IRS Bridge Rules

- Used for different types of Loads
- Corrected up to Slips 49 in 2017

सत्यमेव जयते

GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS
(Railway Board)

BRIDGE RULES
(IN SI UNITS)

RULES SPECIFYING THE LOADS FOR DESIGN OF SUPER-STRUCTURE AND SUB-STRUCTURE OF BRIDGES AND FOR ASSESSMENT OF THE STRENGTH OF EXISTING BRIDGES

ADOPTED –1941
FIRST REVISION - 1964

First Reprinting - 1989
Second Reprinting - 2008
Third Reprinting - 2014
(Incorporating Correction Slips Upto 49)

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

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LUCKNOW - 226011



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## IRS Bridge Rules
## Loads 2.1

- Dead load
- Live load
- Dynamic effects
- Forces due to curvature or eccentricity of track
- Temperature effect
- Frictional resistance of expansion bearings
- Longitudinal force
- Racking force
- Forces on parapets
- Wind pressure effect (BNBC)
- Forces and effects due to earthquake (BNBC)
- Erection forces and effects
- Derailment loads
- Load due to Plasser's Quick Relay System (PQRS)
- Forces due to accidental impact from any vehicles such as road vehicles, ships or derailed train vehicles using the bridge (ACS 48 dtd. 22.06.17)

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## IRS Bridge Rules
## Dead Load 2.2

- 2.2.1 Dead load is the weight of the structure itself together with the permanent loads carried thereon.
- Super Imposed Dead Load: Dead Load over the structure to be designed

- 2.2.2 For design of ballasted deck bridges, a ballast cushion of 400mm for BG and 300mm for MG shall be considered. However, ballasted deck bridges shall also be checked for a ballast cushion of 300mm on BG and 250mm on MG.

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## IRS Bridge Rules
## 2.3 Live Load

- (a) For Broad Gauge - 1676mm – "25t Loading-2008" with a maximum axle load of 245.2 kN (25.0t) for the locomotives and a train load of 91.53 kN/m (9.33t/m) on both sides of the locomotives (Appendix-XXII)
- (2) Diagrams of Standard loading and Equivalent Uniformly Distributed Loads (EUDL) on each track for calculating Bending Moment and Shear Force are shown in the accompanying Appendices XXII, XXIII & XXIII (a) respectively
- (4) EUDLs shall be used for simply supported spans. In case of continuous super-structures over supports, the Bending Moments and Shear Forces for design purposes at various sections shall be computed for loadings shown in Appendix-XXII.

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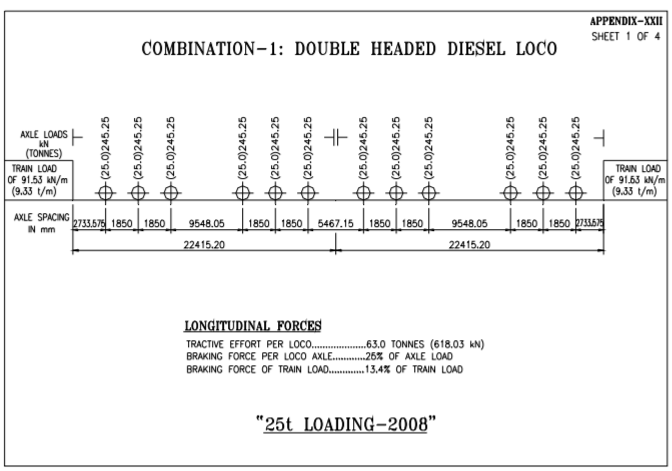
## IRS Bridge Rules
## Live Load 2.3

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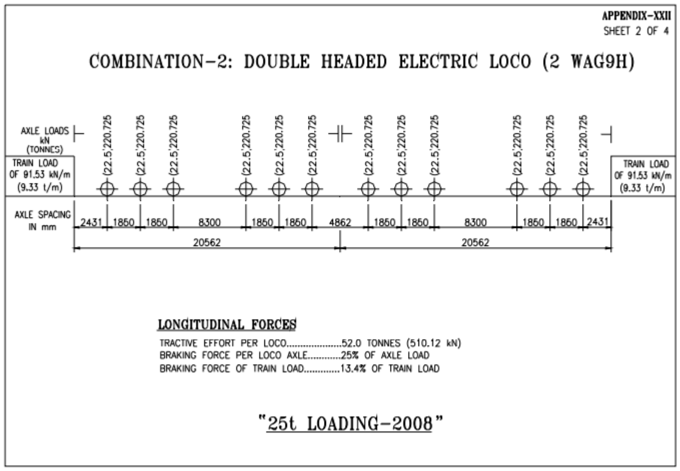
# IRS Bridge Rules
# Live Load 2.3

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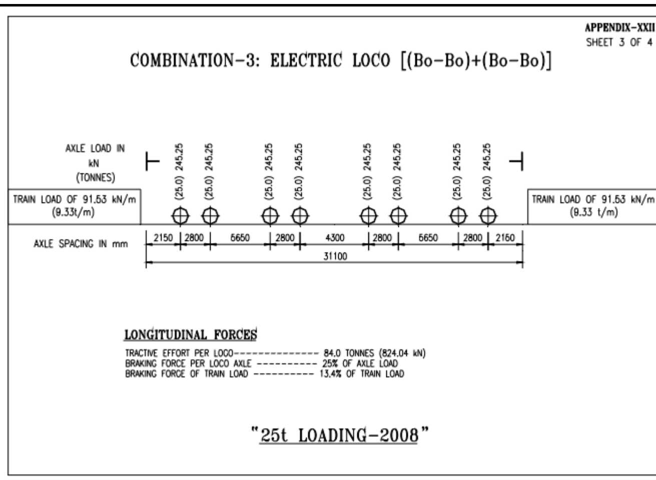
# IRS Bridge Rules
# Live Load 2.3

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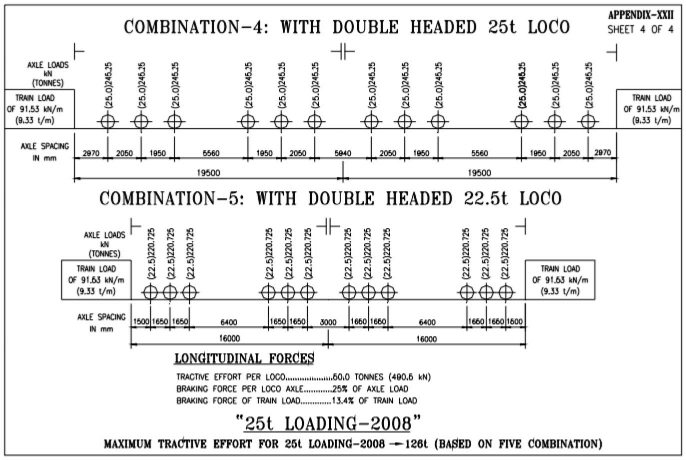
# IRS Bridge Rules
# Live Load 2.3






# IRS Bridge Rules, Live Load 2.3

**APPENDIX-XXIII**

**"25t Loading-2008"**

**BROAD GAUGE-1676 mm**

**Equivalent Uniformly Distributed Loads (EUDL) in kilo Newtons (tonnes) on each track, and Coefficient of Dynamic Augment (CDA).**

For Bending Moment, L is equal to the effective span in metres. For Shear Force, L is the loaded length in metres to give the maximum Shear Force in the member under consideration.

The Equivalent Uniformly Distributed Load (EUDL) for Bending Moment (BM), for spans upto 10m, is that uniformly distributed load which produces the BM at the centre of the span equal to the absolute maximum BM developed under the standard loads. For spans above 10m, the EUDL for BM is that uniformly distributed load which produces the BM at one-sixth of the span equal to the BM developed at that section under the standard loads.

EUDL for Shear Force (SF) is that uniformly distributed load which produces SF at the end of the span equal to the maximum SF developed under the standard loads at that section.

***NOTE:***

**(1)** *Cross girders – The live load on a cross girder will be equal to half the total load for bending in a length L, equal to twice the distance over centres of cross girders.*

**(2)** *L for Coefficient of Dynamic Augment (CDA) shall be as laid down in clause 2.4.1.*

(3) *When loaded length lies between the values given in the table, the EUDL for Bending Moment and Shear Force can be interpolated.*

| L (m) | Total load for Bending Moment | | Total load for Shear Force | | Impact Factor CDA= 0.15+8/(6+L) |
|---|---|---|---|---|---|
| | kN | t | kN | t | |
| 1 | 2 | 3 | 4 | 5 | 6 |
| 1.0 | 490.50 | 50.00 | 490.50 | 50.00 | 1.000 |
| 1.5 | 490.50 | 50.00 | 490.50 | 50.00 | 1.000 |
| 2.0 | 490.50 | 50.00 | 527.29 | 53.75 | 1.000 |
| 2.5 | 490.50 | 50.00 | 618.03 | 63.00 | 1.000 |
| 3.0 | 490.50 | 50.00 | 678.56 | 69.17 | 1.000 |
| 3.5 | 531.02 | 54.13 | 721.72 | 73.57 | 0.992 |
| 4.0 | 595.96 | 60.75 | 790.98 | 80.63 | 0.950 |
| 4.5 | 676.89 | 69.00 | 866.52 | 88.33 | 0.912 |
| 5.0 | 745.56 | 76.00 | 927.05 | 94.50 | 0.877 |
| 5.5 | 811.58 | 82.73 | 976.59 | 99.55 | 0.846 |
| 6.0 | 866.52 | 88.33 | 1017.79 | 103.75 | 0.817 |
| 6.5 | 913.11 | 93.08 | 1052.81 | 107.32 | 0.790 |
| 7.0 | 952.94 | 97.14 | 1086.75 | 110.78 | 0.765 |

| L (m) | Total load for Bending Moment | | Total load for Shear Force | | Impact Factor CDA= 0.15+8/(6+L) |
|---|---|---|---|---|---|
| | kN | t | kN | t | |
| 7.5 | 987.57 | 100.67 | 1119.42 | 114.11 | 0.743 |
| 8.0 | 1017.79 | 103.75 | 1168.37 | 119.10 | 0.721 |
| 8.5 | 1044.47 | 106.47 | 1214.67 | 123.82 | 0.702 |
| 9.0 | 1068.21 | 108.89 | 1265.49 | 129.00 | 0.683 |
| 9.5 | 1089.40 | 111.05 | 1315.03 | 134.05 | 0.666 |
| 10.0 | 1108.53 | 113.00 | 1377.32 | 140.40 | 0.650 |
| 11.0 | 1282.66 | 130.75 | 1492.89 | 152.18 | 0.621 |
| 12.0 | 1377.32 | 140.40 | 1589.22 | 162.00 | 0.594 |
| 13.0 | 1475.13 | 150.37 | 1670.74 | 170.31 | 0.571 |
| 14.0 | 1558.91 | 158.91 | 1740.59 | 177.43 | 0.550 |
| 15.0 | 1631.60 | 166.32 | 1813.48 | 184.86 | 0.531 |
| 16.0 | 1708.41 | 174.15 | 1905.40 | 194.23 | 0.514 |
| 17.0 | 1819.56 | 185.48 | 1997.12 | 203.58 | 0.498 |
| 18.0 | 1889.80 | 192.64 | 2088.94 | 212.94 | 0.483 |
| 19.0 | 1978.28 | 201.66 | 2180.66 | 222.29 | 0.470 |
| 20.0 | 2065.50 | 210.55 | 2272.42 | 231.64 | 0.458 |
| 21.0 | 2151.63 | 219.33 | 2364.21 | 241.00 | 0.446 |
| 22.0 | 2236.88 | 228.02 | 2455.84 | 250.34 | 0.436 |
| 23.0 | 2321.34 | 236.63 | 2547.46 | 259.68 | 0.426 |
| 24.0 | 2405.22 | 245.18 | 2639.18 | 269.03 | 0.417 |
| 25.0 | 2488.40 | 253.66 | 2730.81 | 278.37 | 0.408 |
| 26.0 | 2571.00 | 262.08 | 2822.44 | 287.71 | 0.400 |
| 27.0 | 2653.21 | 270.46 | 2914.06 | 297.05 | 0.392 |
| 28.0 | 2735.03 | 278.80 | 3005.69 | 306.39 | 0.385 |
| 29.0 | 2816.35 | 287.09 | 3097.31 | 315.73 | 0.379 |
| 30.0 | 2897.38 | 295.35 | 3188.94 | 325.07 | 0.372 |
| 32.0 | 3058.66 | 311.79 | 3372.19 | 343.75 | 0.361 |
| 34.0 | 3058.66 | 328.12 | 3555.34 | 362.42 | 0.350 |
| 36.0 | 3378.17 | 344.36 | 3738.59 | 381.10 | 0.340 |
| 38.0 | 3218.86 | 360.53 | 3921.74 | 399.77 | 0.332 |
| 40.0 | 3694.74 | 376.63 | 4104.90 | 418.44 | 0.324 |
| 42.0 | 3852.29 | 392.69 | 4288.05 | 437.11 | 0.317 |
| 44.0 | 4027.20 | 410.52 | 4471.20 | 455.78 | 0.310 |
| 46.0 | 4210.26 | 429.18 | 4654.26 | 474.44 | 0.304 |
| 48.0 | 4393.31 | 447.84 | 4837.41 | 493.11 | 0.298 |
| 50.0 | 4576.37 | 466.50 | 5020.56 | 511.78 | 0.293 |
| 55.0 | 5034.00 | 513.15 | 5478.30 | 558.44 | 0.281 |
| 60.0 | 5491.64 | 559.80 | 5936.03 | 605.10 | 0.271 |
| 65.0 | 5949.27 | 606.45 | 6393.77 | 651.76 | 0.263 |
| 70.0 | 6406.91 | 653.10 | 6851.50 | 698.42 | 0.255 |
| 75.0 | 6864.55 | 699.75 | 7309.23 | 745.08 | 0.249 |
| 80.0 | 7322.18 | 746.40 | 7766.97 | 791.74 | 0.243 |
| 85.0 | 7779.82 | 793.05 | 8224.61 | 838.39 | 0.238 |
| 90.0 | 8237.46 | 839.70 | 8682.34 | 885.05 | 0.233 |
| 95.0 | 8695.09 | 886.35 | 9139.98 | 931.70 | 0.229 |

# IRS Bridge Rules
# Live Load 2.3

| L (m) | Total load for Bending Moment | | Total load for Shear Force | | Impact Factor CDA= |
|---|---|---|---|---|---|
| | kN | t | kN | t | 0.15+8/(6+L) |
| 100.0 | 9152.73 | 933.00 | 9597.61 | 978.35 | 0.225 |
| 105.0 | 9610.37 | 979.65 | 10055.35 | 1025.01 | 0.222 |
| 110.0 | 10068.00 | 1026.30 | 10512.98 | 1071.66 | 0.219 |
| 115.0 | 10525.64 | 1072.95 | 10970.62 | 1118.31 | 0.216 |
| 120.0 | 10983.28 | 1119.60 | 11428.36 | 1164.97 | 0.213 |
| 125.0 | 11440.91 | 1166.25 | 11885.99 | 1211.62 | 0.211 |
| 130.0 | 11898.55 | 1212.90 | 12343.63 | 1258.27 | 0.209 |

EUDL for BM and Shear Force given in this Appendix are not applicable for ballasted deck for spans upto and including 8.0m for which Appendix XXIII(a) is to be referred.





# IRS Bridge Rules, Live Load 2.3

APPENDIX XXIII (a)

"25t LOADING-2008"

BROAD GAUGE 1676mm

1. **Equivalent Uniformly Distributed Load (EUDL) for Bending Moment in Kilo-Newton (tonnes) for cushions of various depths and spans upto and including 8m.**

For Bending Moment L is equal to the effective span in metres.

***NOTE:***

(1) *For intermediate values of L and cushions, the EUDL shall be arrived at by linear interpolation.*

(2) *The figures given below do not include dynamic effects.*

| L | EUDL for Bending Moment Cushion (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 200 | | 300 | | 400 | | 600 | |
| Metres | KN | t | kN | t | KN | t | KN | t |
| 0.5 | 267.5 | 27.3 | 221.1 | 22.6 | 188.2 | 19.2 | 143.5 | 14.6 |
| 1.0 | 378.8 | 38.7 | 354.3 | 36.2 | 329.8 | 33.7 | 280.8 | 28.7 |
| 1.5 | 415.8 | 42.4 | 399.5 | 40.8 | 383.2 | 39.1 | 350.5 | 35.8 |
| 2.0 | 434.4 | 44.3 | 422.1 | 43.1 | 409.9 | 41.8 | 385.4 | 39.3 |
| 2.5 | 445.5 | 45.5 | 435.7 | 44.5 | 425.9 | 43.5 | 406.3 | 41.5 |
| 3.0 | 452.9 | 46.2 | 444.8 | 45.4 | 436.6 | 44.6 | 420.2 | 42.9 |
| 3.5 | 498.6 | 50.9 | 491.7 | 50.2 | 484.6 | 49.5 | 471.1 | 48.1 |
| 4.0 | 570.6 | 58.2 | 564.8 | 57.6 | 559.3 | 57.1 | 549.0 | 56.0 |
| 4.5 | 654.0 | 66.7 | 649.1 | 66.2 | 644.2 | 65.7 | 634.4 | 64.7 |
| 5.0 | 722.6 | 73.7 | 717.7 | 73.2 | 712.8 | 72.7 | 703.0 | 71.7 |
| 5.5 | 790.5 | 80.7 | 786.1 | 80.2 | 781.6 | 79.8 | 772.7 | 78.9 |
| 6.0 | 847.1 | 86.4 | 843.0 | 86.0 | 839.0 | 85.6 | 830.8 | 84.8 |
| 7.0 | 936.1 | 95.5 | 932.7 | 95.2 | 929.1 | 94.8 | 922.1 | 94.1 |
| 8.0 | 1002.8 | 102.3 | 999.7 | 102.0 | 996.8 | 101.7 | 990.6 | 101.1 |

APPENDIX XXIII (a) ( Contd....)

2. **Equivalent Uniformly Distributed Load (EUDL) for Shear Force in Kilo-Newton (tones) for cushions of various depths and spans upto and including 8m.**

For Shear Force, **L** is the loaded length in metres to give the maximum Shear Force in the member.

***NOTE:***

(1) *For intermediate values of L and cushions, the EUDL shall be arrived at by linear interpolation.*

(2) *The figures given below do not include dynamic effects.*

| L | EUDL for Shear Force Cushion (mm) | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 200 | | 300 | | 400 | | 600 | |
| Metres | KN | t | KN | t | KN | t | KN | t |
| 0.5 | 270.1 | 27.5 | 221.3 | 22.6 | 187.5 | 19.1 | 143.6 | 14.6 |
| 1 | 379.3 | 38.7 | 354.6 | 36.2 | 330.1 | 33.7 | 281.1 | 28.7 |
| 1.5 | 416.3 | 42.4 | 400.0 | 40.8 | 383.6 | 39.1 | 350.9 | 35.8 |
| 2 | 434.9 | 44.3 | 422.6 | 43.1 | 410.4 | 41.8 | 385.7 | 39.3 |
| 2.5 | 529.0 | 53.9 | 509.3 | 51.9 | 489.7 | 49.9 | 455.3 | 46.4 |
| 3 | 604.3 | 61.6 | 588.0 | 59.9 | 571.5 | 58.3 | 539.0 | 54.9 |
| 3.5 | 658.1 | 67.1 | 644.1 | 65.7 | 630.0 | 64.2 | 601.9 | 61.4 |
| 4 | 710.6 | 72.4 | 696.1 | 71.0 | 682.4 | 69.6 | 655.9 | 66.9 |
| 4.5 | 792.3 | 80.8 | 776.0 | 79.1 | 759.7 | 77.4 | 727.1 | 74.1 |
| 5 | 860.2 | 87.7 | 845.5 | 86.2 | 831.2 | 84.7 | 801.4 | 81.7 |
| 5.5 | 915.8 | 93.4 | 902.4 | 92.0 | 889.0 | 90.6 | 862.3 | 87.9 |
| 6 | 962.2 | 98.1 | 949.8 | 96.8 | 937.6 | 95.6 | 912.9 | 93.1 |
| 7 | 1034.8 | 105.5 | 1024.4 | 104.4 | 1013.9 | 103.4 | 992.9 | 101.2 |
| 8 | 1089.5 | 111.1 | 1080.3 | 110.1 | 1071.1 | 109.2 | 1052.6 | 107.3 |

# IRS Bridge Rules
# Live Load 2.3

APPENDIX-XXV

DERAILMENT LOADS FOR BALLASTED DECK BRIDGES
(25t Loading-2008)

| S.N. | Condition and approach | Bridges with guard rails | Bridges without guard rails |
|---|---|---|---|
| 1. | **Ultimate –** The load at which a derailed vehicle shall not cause collapse of any major element. | a) Two vertical line loads of 75 kN/m each 1.6m* apart parallel to the track in the most unfavorable position inside an area of 1.3m on either side of track centre line.<br>b) A single load of 200 kN acting on an area of 1.3m on either side of track centre line in the most unfavorable position. | a) Two vertical line loads of 75 kN/m each 1.6m* apart parallel to the track in the most unfavorable position inside an area of 2.25m on either side of track centre line.<br>b) A single line load of 200 kN acting on an area of 2.25m on either side of track centre line in the most unfavorable position. |
| 2. | **Stability –** The structure shall not overturn. | A vertical line load of 94 kN/m with a total length of 20m acting on the edge of the structure under consideration. | A vertical line load of 94 kN/m with a total length of 20m acting on the edge of the structure under consideration |

** The distance 1.6m is based on Broad Gauge distance 1.676m as adopted for derailment loads for MBG-1987 loading and HM loading.*



# IRS Bridge Rules
# 2.3.4.2 Dispersion of railway live loads

(a) Distribution through sleepers and ballast:
The sleeper may be assumed to distribute the live load uniformly on top of the ballast over the area of contact given below:

| Type I | Type II |
|---|---|
| | Under each rail seat |
| BG 2745mm x 254mm | 760mm x 330mm |
| MG 1830mm x 203mm | 610mm x 270mm |

- The load under the sleeper shall be assumed to be dispersed by the fill including ballast at a slope not greater than half horizontal to one vertical and all deck slabs shall be designed for both types of sleepers.

## IRS Bridge Rules
## 2.3.4.2 Dispersion of railway live loads

(b) Distribution through R.C. Slab: When there is effective lateral transmission of Shear Force, the load may be further distributed in a direction at right angles to the span of the slab equal to the following:
(i) ¼ span on each side of the loaded area in the case of simply supported, fixed and continuous spans.
(ii) ¼ of loaded length on each side of the loaded area in the case of cantilever slabs.

- No distribution through the slab may be assumed in the direction of the span of the slab.



## IRS Bridge Rules
## 2.4 DYNAMIC EFFECT,
## 2.4.1 Railway Bridges (Steel)

- The CDA should be obtained as follows and shall be applicable up to 160 km/h on BG and 100 km/h on MG Line

(a) For single track spans: $\text{CDA}=0.15+\dfrac{8}{(6+L)}$

- CDA<=1.0, Where L is:
- (1) the loaded length of span in metres for the position of the train giving the maximum stress in the member under consideration.
- (2) 1.5 times the cross-girder spacing in the case of stringers (rail bearers) and
- (3) 2.5 times the cross girder spacing in the case of cross girders.

# IRS Bridge Rules
## 2.4 DYNAMIC EFFECT,
## 2.4.1 Railway Bridges (Steel)

CDA for Stringer, L=2.5m c/c of Cross Girder x1.5

Coefficient of Dynamic Augment, CDA $= 0.15 + \frac{8}{6 + L} \leq 1.0$

$= 0.15 + \frac{8}{6 + 2.5 \times 1.5} \leq 1.0$

$= 0.9705$

CDA for Cross Girder, L=2.5m c/c of Cross Girder x2.5

Coefficient of Dynamic Augment, CDA $= 0.15 + \frac{8}{6 + L} \leq 1.0$

$= 0.15 + \frac{8}{6 + 2.5 \times 2.5} \leq 1.0$

$= 0.8031$



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# IRS Bridge Rules
## 2.4 DYNAMIC EFFECT,
## 2.4.1 Railway Bridges

❑ Wheel Load Dispersion on Box Culvert

**Case - 1 : Live load and Impact**

"25 T Loading - 2008" as per RDSO

Effective L= 4.35 m

where, I = 0.15+8/(L+6) ≤ 1.0 Impact factor

= 0.923 ≯ 1.00

= 0.923

When the depth of fill < 0.9 m

Depth of Fill= 0.30 m

The co-efficient of Dynamic Augment shall be

[2-(d/0.9)]*1/2*CDA = 0.77

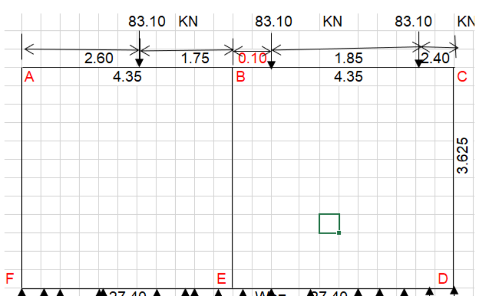
Effect of Dispersion

| | | | |
|---|---|---|---|
| Sleeper width | = | 2.745m | m |
| Depth of Fiil | = | 0.30 | m |
| Span | = | 4.35 | m |

Effect of Dispersion= Sleeper width+(Depth of fill/2+span/4)x2

E = 5.22 m

| | | | |
|---|---|---|---|
| For the axel load | = | 245.2 | Kn/m$^2$ |
| Live Load including Dynamic Augment, P | = | 433.8 | Kn/m$^2$ |
| Load on top slab= P/E | = | 83.1 | KN/m |
| Uplift Pressure on bottom slab | = | 27.396 | Kn/m$^2$ |
| Assume 1.0 m width | | | |



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## IRS Bridge Rules
## 2.4 DYNAMIC EFFECT,
## 2.4.1 Railway Bridges (Steel)

(b) For main girders of double track spans with 2 girders, CDA as calculated above may be multiplied by a factor of 0.72 and shall be subject to a maximum of 0.72. (c) For intermediate main girders of multiple track spans, the CDA as calculated in Clause 2.4.1.1(a) may be multiplied by a factor of 0.6 and shall be subject to a maximum of 0.6. (d) For the outside main girders of multiple track spans with intermediate girders, CDA shall be that specified in Clause 2.4.1.1(a) or (b) whichever applies. (e) For cross girders carrying two or more tracks, CDA as calculated in Clause 2.4.1.1(a) may be multiplied by a factor of 0.72 and shall be subject to a maximum of 0.72

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## IRS Bridge Rules
## 2.4 DYNAMIC EFFECT,
## 2.4.1 Railway Bridges (Steel)

(f) Where rails, with ordinary fish-plated joints, are supported directly on transverse steel troughing or steel sleepers (H Beam) the dynamic augment for calculating stresses in such troughing or sleepers shall be taken as:

$\dfrac{7.32}{B+5.49}$ for BG $\qquad \dfrac{5.49}{B+4.27}$ for MG

Where B = the spacing of main girders in metres.

The same Coefficient of dynamic augment (CDA) may be used for calculating the stresses in main girders up to 7.5m effective span, stringers with spans up to 7.5m and also chords of triangulated girders supporting the steel troughing or steel sleepers.

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## IRS Bridge Rules
## 2.4 DYNAMIC EFFECT,
## 2.4.2 Railway pipe culverts, arch bridges, concrete slabs and concrete girders

2.4.2.1 For all gauges

(a) If the depth of fill is less than 900mm, the Coefficient of Dynamic Augment shall be equal to- $[2-(d/0.9)] \times \frac{1}{2} \times CDA$

as obtained from Clause 2.4.1.1(a)

Where, **d** = depth of fill in 'm'.

(b) If the depth of fill is 900mm, the Coefficient of Dynamic Augment shall be half of that specified in clause 2.4.1.1(a) subject to a maximum of 0.5. Where depth of fill exceed 900mm, the Coefficient of Dynamic Augment shall be uniformly decreased to zero within the next 3 metres.

(c) In case of concrete girders of span of 25m and larger, the CDA shall be as specified in Clause 2.4.1.1. (a)



## IRS Bridge Rules
## 2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK

- 2.5.1 For ballasted deck bridges, even on straight alignment, an eccentricity of centre line of track from design alignment up to 100mm shall be considered for the purpose of designs.

Refer appendix XXIII of IRS bridge rules

For 35.00 m span,

EUDL = $\frac{3298.51 \text{ kN}}{35.00 \text{ m} \times 2 \text{ m}}$ = 47.12 kN/m

CDA = 0.350

Total EUDL = ( 1 + CDA) × Total EUDL

= ( 1 + 0.350 )× 47.1216 = 63.61 kN/m

On one Main Girder, EUDL = 1 × Total EUDL

= 1 × 63.6141 = 63.614 kN/m

onsidering effect of eccentricity of track as per Clause 2.5.1 of IRS Bridge Rules:

ddditional EUDL on main girder = $63.6141 \times \frac{0.1}{6.2}$ (dist between centre lines of main girder)

= 1.026 kN/m

So, Total EUDL on main girder = 63.6141 + 1.02603 = 64.640 kN/m

C/C of Girder 6.2m

## IRS Bridge Rules
## 2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK

- 2.5.3 For railway bridges the following loads must be considered: (a) The extra loads on one girder due to the additional reaction on one rail and to the lateral displacement of the track calculated under the following two conditions: (i) Live load running at the maximum speed. (ii) Live load standing with half normal dynamic augment
- (b) The horizontal load due to centrifugal force which may be assumed to act at a height of 1830mm for "25t Loading2008" for BG, 3000mm for "DFC loading (32.5t axle load)" for BG and 1450mm for MG above rail level is:

$$C = \frac{WV^2}{12.95R} \text{ OR } \left( \frac{WV^2}{127R} \text{ in MKS Units} \right)$$

Where, C= Horizontal effect in kN/m run (t/m run) of span.
W= Equivalent Distributed live load in t/m run.
V= Maximum speed in km per hour, and R= Radius of the curve in m.

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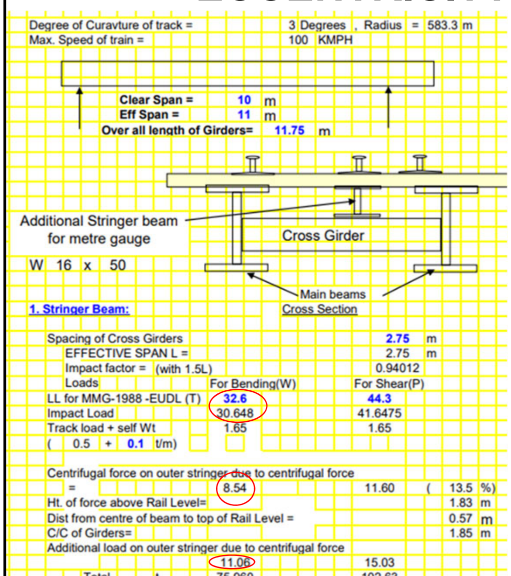
## 2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK




Degree of Curvature $3^0$
R=1750/D=1750/3=583.3m
**Let Speed, V=100 km/hr**
**W=LL+I=32.6+30.648t**
**=63.248t**
**C=WV^2/127R**
**= 63.248x100^2/(127x583.3)**
**=8.54t/m**
**Distance from center of beam to C force=**
**(1.83+0.57)m=2.4m**
**C/C of Girder= 1.85m**
**Add. Vertical Load**
**=8.54x2.4/1.85= 11.06t/m**

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# IRS Bridge Rules

## *2.6 TEMPERATURE EFFECT*

- 2.6.1 Where any portion of the structure is not free to expand or contract under variation of temperature, allowance shall be made for the stresses resulting from this condition.
- 2.6.2 The coefficient of expansion shall be taken as below: for steel and reinforced concrete 11.7 x $10^{-6}$ per $1^0$ C for plain concrete 10.8 x $10^{-6}$ per $1^0$ C
- Let Length of Girder L=35m

Average Temperature variation t=$30^0$ C
Coefficient of Expansion of steel =11.7x$10^{-6}$ per $1^0$ C
Expansion Length= 35x1000x11.7x$10^{-6}$x30=12.285mm



# IRS Bridge Rules

2.7 FRICTIONAL RESISTANCE OF EXPANSION BEARINGS

| | |
|---|---|
| • Roller bearing | 0.03 |
| • Sliding bearings of steel on cast iron or steel bearing | 0.25 |
| • Sliding bearing of steel on ferro bestos | 0.20 |
| • Sliding bearings of steel on hard copper alloy bearings | 0.15 |
| • For sliding bearings of PTFE/Elastomeric type | 0.10 |
| • Concrete over concrete with bitumen layer in between | 0.50 |
| • Concrete over concrete not intentionally roughened | 0.60 |

# IRS Bridge Rules

## *2.8 LONGITUDINAL FORCES*

- Tractive effort of the driving wheels of locomotives;
- Braking force resulting from the application of the brakes to all braked wheels;
- Resistance to the movement of the bearings due to change of temperature and deformation of the bridge girder. Roller, PTFE or elastomeric bearings may preferably be provided to minimize the longitudinal force arising on this account.
- Forces due to continuation of LWR/CWR over the bridges.



# IRS Bridge Rules, Live Load 2.8 Tractive/ Braking

APPENDIX-XXIV

**"25t Loading-2008"**
**BROAD GAUGE-1676 mm**
**Longitudinal Loads (Without Deduction for Dispersion)**

**NOTE:** Where loaded length lies between the values given in the Table, the tractive effort or braking force can, with safety, be assumed as that for the longer loaded length.

| L (Loaded length in metres) | Tractive effort | | Braking force | | Maximum LF | |
|---|---|---|---|---|---|---|
| | kN | t | kN | t | kN | t |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 |
| 1.0 | 103.01 | 10.50 | 61.80 | 6.30 | 103.01 | 10.50 |
| 1.5 | 103.01 | 10.50 | 61.80 | 6.30 | 103.01 | 10.50 |
| 2.0 | 206.01 | 21.00 | 122.63 | 12.50 | 206.01 | 21.00 |
| 2.5 | 206.01 | 21.00 | 122.63 | 12.50 | 206.01 | 21.00 |
| 3.0 | 206.01 | 21.00 | 122.63 | 12.50 | 206.01 | 21.00 |
| 3.5 | 245.25 | 25.00 | 165.79 | 16.90 | 245.25 | 25.00 |
| 4.0 | 309.02 | 31.50 | 184.43 | 18.80 | 309.02 | 31.50 |
| 4.5 | 309.02 | 31.50 | 184.43 | 18.80 | 309.02 | 31.50 |
| 5.0 | 309.02 | 31.50 | 184.43 | 18.80 | 309.02 | 31.50 |
| 5.5 | 309.02 | 31.50 | 184.43 | 18.80 | 309.02 | 31.50 |
| 6.0 | 309.02 | 31.50 | 184.43 | 18.80 | 309.02 | 31.50 |
| 6.5 | 326.87 | 33.32 | 220.73 | 22.50 | 326.87 | 33.32 |
| 7.0 | 326.87 | 33.32 | 220.73 | 22.50 | 326.87 | 33.32 |
| 7.5 | 326.87 | 33.32 | 220.73 | 22.50 | 326.87 | 33.32 |
| 8.0 | 408.59 | 41.65 | 275.66 | 28.10 | 408.59 | 41.65 |
| 8.5 | 408.59 | 41.65 | 275.66 | 28.10 | 408.59 | 41.65 |
| 9.0 | 408.59 | 41.65 | 275.66 | 28.10 | 408.59 | 41.65 |
| 9.5 | 412.02 | 42.00 | 275.66 | 28.10 | 412.02 | 42.00 |
| 10.0 | 490.30 | 49.98 | 331.58 | 33.80 | 490.30 | 49.98 |
| 11.0 | 490.30 | 49.98 | 331.58 | 33.80 | 490.30 | 49.98 |
| 12.0 | 515.03 | 52.50 | 331.58 | 33.80 | 515.03 | 52.50 |
| 13.0 | 618.03 | 63.00 | 331.58 | 33.80 | 618.03 | 63.00 |
| 14.0 | 618.03 | 63.00 | 367.88 | 37.50 | 618.03 | 63.00 |
| 15.0 | 618.03 | 63.00 | 367.88 | 37.50 | 618.03 | 63.00 |
| 16.0 | 618.03 | 63.00 | 386.51 | 39.40 | 618.03 | 63.00 |
| 17.0 | 618.03 | 63.00 | 386.51 | 39.40 | 618.03 | 63.00 |
| 18.0 | 653.74 | 66.64 | 441.45 | 45.00 | 653.74 | 66.64 |
| 19.0 | 653.74 | 66.64 | 441.45 | 45.00 | 653.74 | 66.64 |
| 20.0 | 735.46 | 74.97 | 496.39 | 50.60 | 735.46 | 74.97 |
| 21.0 | 735.46 | 74.97 | 499.33 | 50.90 | 735.46 | 74.97 |
| 22.0 | 735.46 | 74.97 | 511.10 | 52.10 | 735.46 | 74.97 |
| 23.0 | 735.46 | 74.97 | 523.12 | 53.38 | 735.46 | 74.97 |
| 24.0 | 735.46 | 74.97 | 551.32 | 56.20 | 735.46 | 74.97 |
| 25.0 | 824.04 | 84.00 | 551.32 | 56.20 | 824.04 | 84.00 |
| 26.0 | 824.04 | 84.0 | 560.15 | 57.10 | 824.04 | 84.0 |

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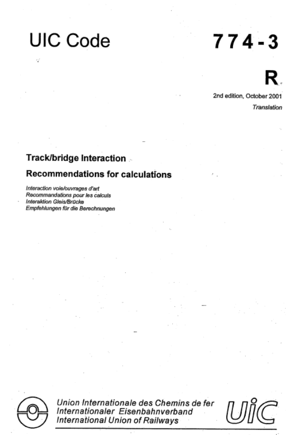
| L (Loaded length in metres) | Tractive effort | | Braking force | | Maximum LF | |
|---|---|---|---|---|---|---|
| | kN | t | kN | t | kN | t |
| 27.0 | 927.05 | 94.50 | 572.90 | 58.40 | 927.05 | 94.50 |
| 28.0 | 927.05 | 94.50 | 607.05 | 61.88 | 927.05 | 94.50 |
| 29.0 | 980.61 | 99.96 | 662.18 | 67.50 | 980.61 | 99.96 |
| 30.0 | 980.61 | 99.96 | 662.18 | 67.50 | 980.61 | 99.96 |
| 32.0 | 980.61 | 99.96 | 680.81 | 69.40 | 980.61 | 99.96 |
| 34.0 | 980.61 | 99.96 | 735.75 | 75.00 | 980.61 | 99.96 |
| 36.0 | 1030.05 | 105.00 | 735.75 | 75.00 | 1030.05 | 105.00 |
| 38.0 | 1133.06 | 115.50 | 760.28 | 77.50 | 1133.06 | 115.50 |
| 40.0 | 1236.06 | 126.00 | 784.80 | 80.00 | 1236.06 | 126.00 |
| 42.0 | 1236.06 | 126.00 | 809.33 | 82.50 | 1236.06 | 126.00 |
| 44.0 | 1236.06 | 126.00 | 833.85 | 85.00 | 1236.06 | 126.00 |
| 46.0 | 1236.06 | 126.00 | 858.38 | 87.50 | 1236.06 | 126.00 |
| 48.0 | 1236.06 | 126.00 | 882.90 | 90.00 | 1236.06 | 126.00 |
| 50.0 | 1236.06 | 126.00 | 907.43 | 92.50 | 1236.06 | 126.00 |
| 55.0 | 1236.06 | 126.00 | 968.25 | 98.70 | 1236.06 | 126.00 |
| 60.0 | 1236.06 | 126.00 | 1030.05 | 105.00 | 1236.06 | 126.00 |
| 65.0 | 1236.06 | 126.00 | 1090.87 | 111.20 | 1236.06 | 126.00 |
| 70.0 | 1236.06 | 126.00 | 1152.68 | 117.50 | 1236.06 | 126.00 |
| 75.0 | 1236.06 | 126.00 | 1213.50 | 123.70 | 1236.06 | 126.00 |
| 80.0 | 1236.06 | 126.00 | 1275.30 | 130.00 | 1275.30 | 130.00 |
| 85.0 | 1236.06 | 126.00 | 1336.12 | 136.20 | 1336.12 | 136.20 |
| 90.0 | 1236.06 | 126.00 | 1397.93 | 142.50 | 1397.93 | 142.50 |
| 95.0 | 1236.06 | 126.00 | 1458.75 | 148.70 | 1458.75 | 148.70 |
| 100.0 | 1236.06 | 126.00 | 1520.55 | 155.00 | 1520.55 | 155.00 |
| 105.0 | 1236.06 | 126.00 | 1581.37 | 161.20 | 1581.37 | 161.20 |
| 110.0 | 1236.06 | 126.00 | 1643.18 | 167.50 | 1643.18 | 167.50 |
| 115.0 | 1236.06 | 126.00 | 1704.00 | 173.70 | 1704.00 | 173.70 |
| 120.0 | 1236.06 | 126.00 | 1765.80 | 180.00 | 1765.80 | 180.00 |
| 125.0 | 1236.06 | 126.00 | 1826.62 | 186.20 | 1826.62 | 186.20 |
| 130.0 | 1236.06 | 126.00 | 1888.43 | 192.50 | 1888.43 | 192.50 |

- ***In between given Length Longer loaded length***

# IRS Bridge Rules

## 2.8 LONGITUDINAL FORCES LWR/CWR

- A length of greater than 250 meter on Broad Gauge and 500 m on Meter Gauge will normally 4 function as LWR (Fig. 1.1). The maximum length of LWR under Indian conditions shall normally be restricted to one block section.
- 2) Continuous Welded Rail (CWR) is a LWR which would continue through station yards including points and crossings. (internet Source)
- IRS Refers to UIC 774-3 R-2001


UIC Code
774-3
R
2nd edition, October 2001
Translation
Track/bridge Interaction
Recommendations for calculations
Union Internationale des Chemins de fer
Internationaler Eisenbahnverband
International Union of Railways
UIC

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# IRS Bridge Rules

## 2.8 LONGITUDINAL FORCES LWR/CWR

- Ballasted deck bridges without bearings (slabs, box culverts and arches) need not be checked for forces/effects due to continuation of LWR/CWR.
- (g) If rail-free fastenings, SEJ (Switch Expansion Joint)are provided as per provisions of Manual Of Instructions On Long Welded Rails, such that there is no interaction between the rail and the bridge, then there is no need for checking for forces/effects due to continuation of LWR/CWR.
- It shall be ensured that the additional stresses in rail as per computations done using provisions of UIC 774-3R do not exceed the values given in table below:

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## IRS Bridge Rules

2.8 LONGITUDINAL FORCES LWR/CWR

- Span and sub structure arrangement shall be such that the various checks on rotation/ deflection specified in UIC 774-3R are satisfied.

| Rail Section | Maximum additional Stresses in Compression | Maximum additional Stressess in Tension |
|---|---|---|
| 60 Kg 90 UTS Rail | 60 $N/mm^2$ | 75 $N/mm^2$ |
| 52 Kg 90 UTS Rail | 50 $N/mm^2$ | 60 $N/mm^2$ |

- Track Resistance in Ballasted Deck Bridges : For track structure minimum 52 kg 90 UTS rails and PRC sleepers at sleeper density 1540 nos/KM with elastic fastenings, the value of track resistance for computations as per UIC 774-3R shall be taken as 25kN per meter of track in unloaded condition and 50 kN per meter of track in loaded condition.



## IRS Bridge Rules

2.8 LONGITUDINAL FORCES LWR/CWR

- 2.8.3.4 For the design of new bridges and in case of rebuilding of existing bridges, dispersion of longitudinal force shall not be allowed.
- Multiplication Factor of LF for More than 1 track

| | |
|---|---|
| 1 | 1.00 |
| 2 | 1.00 |
| 3 | 0.90* |
| 4 or more | 0.75* |

# IRS Bridge Rules

## 2.8 LONGITUDINAL FORCES

2.8.5 When considering seismic forces, only 50% of gross tractive effort/braking force, to be reduced by taking dispersion and distribution of longitudinal forces, shall be considered along with horizontal seismic forces along/across the direction of the traffic.



# IRS Bridge Rules

## 2.9 RACKING FORCES

2.9.1 Lateral bracings of the loaded deck of railway spans shall be designed to resist, in addition to the wind and centrifugal loads specified above, a lateral load due to racking forces of 5.88 kN/m (600 kg/m) treated as moving load.
This lateral load need not be taken into account when calculating stresses in chords or flanges of main girders.

## IRS Bridge Rules

**_2.10 FORCES ON PARAPETS_**

- shall be designed to resist a lateral horizontal force and a vertical force of 1.47 kN/m(150 kg/m) applied simultaneously at the top of the railing or parapet.



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## IRS Bridge Rules

**_2.11 WIND PRESSURE EFFECT_**

- 2.11.1.1 Wind pressures are expressed in terms of a basic wind pressure ‘P’ which is an equivalent static pressure in the windward direction. (From BNBC wind speed)
- 2.11.2 The wind pressure specified above shall apply to all loaded or unloaded bridges provided that a bridge shall not be considered to be carrying any live load when the wind pressure at deck level exceeds the following limits:

| | |
|---|---|
| Broad Gauge bridges | 1.47 kN/m$^2$ (150 kg/m$^2$) |
| Metre and Narrow Gauge Bridges | 0.98 kN/m$^2$ (100kg/m$^2$) |
| Foot-bridges | 0.74 kN/m$^2$ (75 kg/m$^2$) |



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# IRS Bridge Rules

## *2.11 WIND PRESSURE EFFECT*

- (a) For unloaded spans and trestles net exposed area shall be considered as one and half times the horizontal projected area of the span or the trestle, except for plate girders for which the area of the leeward girder shall be multiplied by the factors shown below and added to the area of the windward girder: -

| | |
|---|---|
| When the spacing of the leeward girder does not exceed half its depth | 0.00 |
| For spacing exceeding half depth and upto full depth | 0.25 |
| For spacing exceeding full depth and upto one and half times depth | 0.50 |
| For spacing exceeding one and a half times depth and upto twice its depth or more | 1.00 |

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# IRS SEISMIC CODE

## *2.12 FORCES AND EFFECTS DUE TO EARTHQUAKE*

***Refers to "Seismic Code for Earthquake resistant design of Railway Bridges"***

3.7 Effects of Earthquake - The effects of earthquake motion that shall be considered in seismic design of bridge include inertial force, displacements, earth pressure, hydrodynamic pressure and liquefaction of soil.


Government of India
Ministry of Railways
SEISMIC CODE
FOR
EARTHQUAKE RESISTANT DESIGN
OF RAILWAY BRIDGES
First Revision - 2020
(ADOPTED - 2017)
RDSO
ISSUED BY
RESEARCH DESIGNS AND STANDARDS ORGANISATION
LUCKNOW – 226011

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# IRS Seismic Code
# 5 DESIGN PHILOSOPHY

5.1 Serviceability Limit State The design of bridge should meet the serviceability limit state under design basis earthquake (DBE). The parts of the bridge intended to contribute to energy dissipation shall undergo minor damage without giving rise to need for reduction of traffic or immediate repair.

5.2. Ultimate Limit State The design of bridge should meet non-collapse requirement that is, ultimate limit state under maximum considered earthquake (MCE). While designing as per IRS concrete bridge code, DBE may be considered. The bridge shall be retain its structural integrity and adequate residual resistance, although considerable damage may occur in some portions of the bridge. The structure should be able to sustain emergency traffic, inspections and repair could be performed easily after the earthquake. The bridge superstructure however shall in general be protected from the formation of plastic hinges and from unseating due to extreme seismic displacements under MCE.



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# IRS Seismic Code
# 5 DESIGN PHILOSOPHY

5.3 Ductile Behavior The reinforced and pre-stressed concrete components shall be designed as under-reinforced so as to cause a tensile failure. Further, they should be suitably designed to ensure that premature failure due to shear or bond does not occur. Stresses induced in the superstructure due to earthquake ground motion are usually quite nominal. Therefore, ductility demand under seismic shaking has not been a major concern in the bridge superstructures during past earthquakes. However, the seismic response of bridges is critically dependent on the ductile characteristics of the sub-structures. Provisions for appropriate ductile detailing of reinforced concrete members given in Annex B shall be applicable to sub-structures. Bridges shall be designed such that under severe seismic shaking plastic hinges form in the sub-structure, rather than in the deck or foundation. Ductile detailing is mandatory for piers/portals of bridges located in seismic III, IV & V.



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# IRS Seismic Code
# 2.12 Seismic Zoning Map India

***9.4.6 Seismic Zone Map***

For the purpose of determining design seismic forces, the country is classified into four seismic zones. A seismic zone map of India is shown in Fig. 1A. The peak ground acceleration (PGA) (or zero period acceleration, ZPA), associated with each zone, is called zone factor, Z. The zone factor is given in Table 1A. Zone factors for some important towns are given in Appendix E.

Table 1A - Zone Factor Z for Horizontal Motion

| Seismic Zone | II | III | IV | V |
|---|---|---|---|---|
| Z | 0.10 | 0.16 | 0.24 | 0.36 |

Note: Near Source Effect

*For bridges which are within a distance of 10 km from a known active fault, seismic hazard shall be specified after detailed geological study of the fault and the site condition. In absence of such detailed investigation, the near-source modification in the form of 20% increase in zone factor may be used.*

BNBC-2020

| I | II | III | IV |
|---|---|---|---|
| 0.12 | 0.20 | 0.28, | 0.36 |

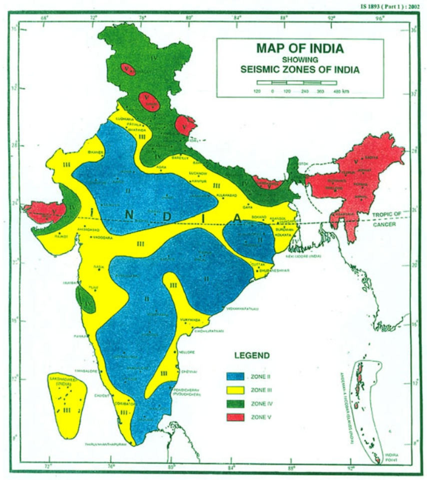


**Fig. 1A Seismic zone map of India**



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# 2.12 Seismic Zoning Map BNBC 2020

বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১১, ২০২১ ৩১৯৫

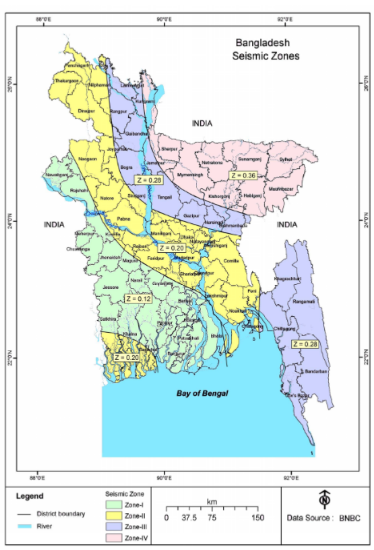


**Figure 6.2.24 Seismic zoning map of Bangladesh**

৩১৯৬ বাংলাদেশ গেজেট, অতিরিক্ত, ফেব্রুয়ারি ১১, ২০২১

**Table 6.2.14: Description of Seismic Zones**

| Seismic Zone | Location | Seismic Intensity | Seismic Zone Coefficient, $Z$ |
|---|---|---|---|
| 1 | Southwestern part including Barisal, Khulna, Jessore, Rajshahi | Low | 0.12 |
| 2 | Lower Central and Northwestern part including Noakhali, Dhaka, Pabna, Dinajpur, as well as Southwestern corner including Sundarbans | Moderate | 0.20 |
| 3 | Upper Central and Northwestern part including Brahmanbaria, Sirajganj, Rangpur | Severe | 0.28 |
| 4 | Northeastern part including Sylhet, Mymensingh, Kurigram | Very Severe | 0.36 |

Both Coefficient nearly same



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## IRS Seismic Code
## 7.4.2 CALCULATION OF NATURAL PERIOD OF BRIDGE

7.4.2 The seismic zone factor for vertical ground motions, when required may be taken as two-thirds of that for horizontal motions given in Table 2 of IS 1893 (Part 1).

However, the time period for the superstructure has to be worked out separately using the characteristic of the superstructure for vertical motion, in order to estimate S /g for vertical acceleration. The natural time period of superstructure can be a estimated using appropriate modelling and free vibration analysis using computer.

However, for simply supported superstructure with uniform flexural rigidity, the fundamental time period $T_v$ , for vertical motion $T_v$ can be estimated using the expression:

$$T_v = \frac{2}{\pi} l^2 \sqrt{\frac{m}{EI}}$$

where l = span; m = mass per unit length; and EI= flexural rigidity of the superstructure. For Elastic Analysis R=1.0

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## IRS Seismic Code

- 8. CALCULATION OF NATURAL PERIOD OF BRIDGE
- 8.1 Simply Supported Bridges
- Where the vibration unit of sub-structure can be idealized as a single cantilever pier carrying the superstructure mass, resting on well, pile or open foundation, the fundamental period shall be calculated from the following equation:

$$T = 2\pi \sqrt{\frac{\delta}{g}}$$

| | | | |
|---|---|---|---|
| Where | $\delta$ | = | horizontal displacement at the top of pier due to horizontal force (= $mg$) |
| where | $m$ | = | lumped mass at the top of pier. |

δ=Fh^2/(3EI) for single pier, F is the total Lumped Weight on pier

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# IRS Seismic Code

- 8. CALCULATION OF NATURAL PERIOD OF BRIDGE

IRC:6-2014

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

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

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

where,

D = Appropriate dead load of the superstructure and live load in kN

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





# IRS Seismic Code

## *2.12 LONGITUDINAL FORCES*

- In general pier shall be considered fixed at the foundation level. However, in case of soft soil or deep foundations, soil flexibility may be considered in the calculation of natural period as per 6.4 of IS 1893 (Part 1)
- Moment of Inertia may be considered for calculation of time period. In case of RCC bridge piers,75% of gross moment of Inertia may be considered as cracked moment of inertia, in absence of detailed calculation.

# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

- 9 METHOD OF CALCULATING SEISMIC FORCES
- 9.1 The following methods of seismic analysis may be employed for calculation of seismic forces in bridges:
- a) Seismic coefficient method (SCM);
- b) Response spectrum method (RSM);
- c) Time history method (THM); and
- d) Nonlinear pushover analysis (NPA).
- The recommended method of analysis for different category of bridges and earthquake level is given in Table 1.
- The linear analysis considering elastic behaviour is required for DBE (Design Basis Earthquake).



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# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**Table 1 Method of Seismic Analysis of Bridges**
*(Clause 9.1)*

| Earthquake level | Category of Bridge Type | | |
|---|---|---|---|
| | Regular | Special Regular | Special Irregular |
| (1) | (2) | (3) | (4) |
| DBE | SCM | RSM | RSM<br>THM<br>NPA |

*NOTE : In case of MCE, non-linear analysis and Tie History Method shall be adopted for regular, special regular and special irregular bridges.*



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# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**3.12 Regular Bridge** - A regular bridge has no abrupt or unusual changes in mass, stiffness or geometry along its span and has no large differences in these parameters between adjacent supports (abutments excluded). A bridge shall be considered regular for the purpose of this standard, if

(a) it is straight or it describes a sector of an arch which subtends an angle less than 90° at the center of the arch;

(b) the adjacent piers do not differ in stiffness by more than 25 percent (Percentage difference shall be calculated based on the lesser of the two stiffness); and

(c) girder bridges, T-beam bridges, truss bridges, hammer head bridges, bridges having single or multiple simply supported spans with each span less than 120 m and pier height above foundation level less than 30 m.

**3.9 Special Regular Bridge** - The bridges specified under regular bridges but single span more than 120 m or pier height measured from founding level to the top of pier cap to be more than 30 m. In case of pile foundation pier height shall be considered from the point of fixity.

**3.16 Special and Irregular Types of Bridges** - The bridges with innovative designs and bridges such as suspension bridge, cable stayed bridge, arch bridge, bascule bridge and irregular bridges such as skew bridge of angle 30° and above with span more than 60 m shall be categorized under these types.



# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**9.2 Seismic Co-efficient Method**

The seismic force to be resisted by bridge components shall be computed as follows:

$$F = A_h W$$

where

$F$ = horizontal seismic force to be resisted;

$W$ = weight of mass under consideration ignoring reduction due to buoyancy or uplift; and

$A_h$ = design horizontal seismic coefficient as determined from 9.4.1.

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

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

# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**9.4.1 Horizontal Seismic Coefficient, $A_h$**

The design horizontal seismic coefficient, $A_h$ shall be determined from following expression of 6.4.2 of IS1893 (Part 1)

$$A_h = \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g}$$

Provided that for any structure with $T < 0.1$ s, the value of $A_h$ shall not be taken less than $Z/2$, whatever be the value of $I/R$.



# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

where

$Z$ = *zone factor;*

$I$ = importance factor *(see* Table 2);

$R$ = response reduction factor *(see* Table 3); and

$S_a/g$ = average acceleration coefficient for rock or soil sites as given in Fig. 1.

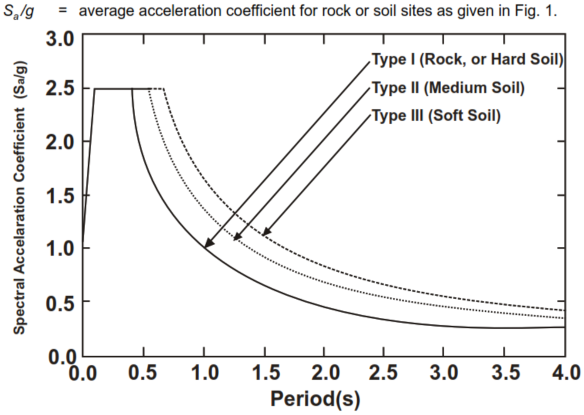


Fig. 1 : RESPONSE SPECTRA FOR ROCK AND SOIL SITES FOR 5 PERCENT DAMPING

# IRS Seismic Code

## *2.12 LONGITUDINAL FORCES*

**Table 2 Importance Factor**
**(Clause 9.4.4)**

| S.No. (1) | Seismic Class (2) | Illustrative Examples of Bridges (3) | Importance Factor 'I' (4) |
|---|---|---|---|
| i) | Normal bridges | All bridges except those mentioned in other class | 1.0 |
| ii) | Important bridges | a) River bridges and flyovers inside cities | 1.2 |
| | | b) Bridges on national and state highways | 1.2 |
| | | c) Bridges serving traffic near ports and other centres of economic activities. | 1.2 |
| | | d) Bridges crossing railway lines. | 1.2 |
| iii) | Large critical bridges in all Seismic Zones | a) Long bridges more than 1 m length across perennial rivers and creeks | 1.5 |
| | | b) Bridges for which alternative routes are not available | 1.5 |
| iv) | Railway bridges | a) All important bridges irrespective of route. | 1.5 |
| | | b) Major bridges on group A, B and C routes (Route classification as per IRP way manual) | 1.5 |
| | | c) Major bridges on all other routes. | 1.25 |
| | | d) All other bridges on group A,B, and C routes | 1.25 |
| | | e) All other bridges | 1.0 |

*NOTE : While checking for seismic effect during construction, the importance factor of I shall be considered for all bridges in all zones.*



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# IRS Seismic Code

## *2.12 LONGITUDINAL FORCES*

**Table 3 Response Reduction Factor R for Bridge Components**
**(Clause 9.4.5)**

| Sl. No. (1) | Structure, Component or Connection (2) | R (3) |
|---|---|---|
| i) | Superstructure | 2 |
| ii) | Substructure: | |
| | a) Reinforced concrete piers with ductile detailing cantilever type, wall type | 3.0 |
| | b) Reinforced concrete piers without ductile detailing*, cantilever type, wall type | 2.5 |
| | c) Masonry piers (un reinforced) cantilever type, wall type | 1.5 |
| | d) Reinforced concrete, framed construction in piers, with ductile detailing, columns of RCC bents, RCC single column piers | 4.0 |
| | e) Steel framed construction | 2.5 |
| | f) Steel cantilever piers | 1.0 |
| | g) Steel trussed arch | 1.5 |
| | h) Reinforced concrete arch | 3.5 |
| | i) Abutments of mass concrete and masonry | 1.0 |
| | j) R.C.C. abutment | 2.5 |
| | k) Integral frame with ductile detailing, and | 4.0 |
| | l) Integral frame without ductile detailing | 3.3 |
| iii) | Bearings (Elastomeric, pot, knuckle, roller-rocker) | 2.0 |
| iv) | Expansion joints and connections within a span of structure, hinge | 1.0 |
| v) | Stoppers in bearings | 1.0 |
| vi) | Foundations (well, piles or open). | 2.0 |

*NOTE - Response reduction factor, R should be taken as 1.0 for calculating displacements.*



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# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**7.2 Combination of Seismic Design Forces with Other Forces**

The design seismic force resultant at a cross-section of a bridge component shall be appropriately combined with those due to other forces as per Table 12 of IRS Concrete Bridge Code (reprint 2014). However, in lieu of combination 2 of Clause 11.0 of IRS Concrete Bridge Code, following load combinations shall be used:

**(A) Ultimate limit state design**

1) $1.25DL + 1.5\,DL(S) + 1.5EQ + 1.4\,PS + 1.7\,EP$
2) $1.25DL + 1.5DL(S) + 0.5(LL + LL\,(F)) + 1.2EQ + 1.7\,EP + 1.4PS + 1.\,4HY + 1.4BO$
3) $0.9DL + 0.8DL(S) + 1.5EQ + 1.4\,PS + 1.7\,EP$

**(B) Serviceability Limit State**

1) $1.0\,DL + 1.2\,DL(S) + 1.0\,EQ + 1.0\,EP + 1.0PS + 1.0HY + 1.0BO$
2) $1.0\,DL + 1.2\,DL(S) + 0.5(LL + LL(F)) + 1.0EQ + 1.0\,EP + 1.0PS + 1.0HY + 1.0\,BY$

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# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**C) During the construction stage, following load combination shall be used:**

$$1.0\,DL + 1.2DL(S) + 0.8EQ + 1.0ER + 1.3EP + 1.0PS + 1.0HY + 1.0BO$$

The symbols used in above equation are explained as below

*DL* = dead load,

*DL(S)* = superimposed dead load,

*LL* = Full live load (without any reduction)

*The live load (LL) includes impact effect, longitudinal forces (tractive and braking), and centrifugal force.*

*LL (F)* = live load on footpath,
*EQ* = earthquake load,
*EP* = earth pressure,
*ER* = erection load such as cranes, machines etc
*PS* = prestressing load,
*HY* = hydrodynamic load,
BO = buoyancy load,
*SH* = shrinkage load,
*CR* = creep load,
*TE* = temperature load.

a) $\pm EL_x \pm 0.3EL_y$

b) $\pm 0.3EL_x \pm EL_x$

1) $\pm EL_x \pm 0.3EL_y \pm 0.3EL_z$

2) $\pm 0.3EL_x \pm EL_y \pm 0.3EL_z$

3) $\pm 0.3EL_x \pm 0.3EL_y \pm EL_z$

$\sqrt{EL_x^2 + EL_y^2}$ or $\sqrt{EL_x^2 + EL_y^2 + EL_z^2}$

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# IRS Seismic Code

**10 HYDRODYNAMIC FORCE ON SUB-STRUCTURE**

**10.1** The hydrodynamic force on submerged portion of pier and foundation up to mean scour level shall be assumed to act in a horizontal direction corresponding to that of earthquake motion. The total horizontal force is given by the following formula:

$$F = C_e A_h W_e$$

where

$C_e$ = Coefficient(see table 4);

$A_h$ = design horizontal seismic coefficient;

$W_e$ = weight of the water in the enveloping cylinder,

= $\rho_w \pi a^2 H$, See **10.3**

$\rho_w$ = unit weight of water;

$H$ = height of submerged portion of pier; and

$a$ = radius of enveloping cylinder.

**Table 4 Values of $C_e$**
**(Clause 10.1)**

| S.No. (1) | H/a (2) | Ce (3) |
|---|---|---|
| i) | 1.0 | 0.390 |
| ii) | 2.0 | 0.575 |
| iii) | 3.0 | 0.675 |
| iv) | 4.0 | 0.730 |





# IRS Seismic Code

## *Hydrodynamic Forces*

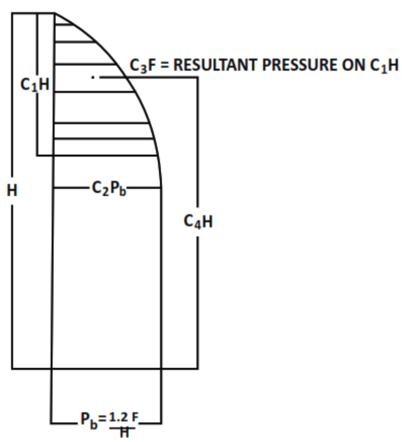


FIG. 2 DIAGRAM SHOWING HYDRODYNAMIC PRESSURE

**Table 5 Coefficients $C_1$, $C_2$, $C_3$, and $C_4$**
**(Clause 10.1)**

| Sl.No. (1) | $C_1$ (2) | $C_2$ (3) | $C_3$ (4) | $C_4$ (5) |
|---|---|---|---|---|
| i) | 0.1 | 0.410 | 0.026 | 0.934 |
| ii) | 0.2 | 0.673 | 0.093 | 0.871 |
| iii) | 0.3 | 0.832 | 0.184 | 0.810 |
| iv) | 0.4 | 0.922 | 0.289 | 0.751 |
| v) | 0.5 | 0.970 | 0.403 | 0.694 |
| vi) | 0.6 | 0.990 | 0.521 | 0.639 |
| vii) | 0.8 | 0.999 | 0.760 | 0.532 |
| viii) | 1.0 | 1.000 | 1.000 | 0.428 |

# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

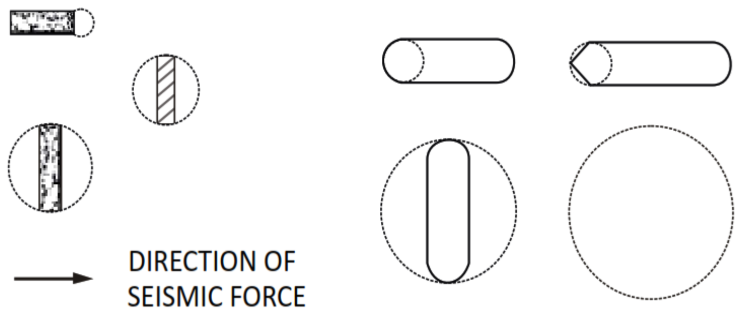


FIG. 3 CASES OF ENVELOPING CYLINDER





# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**11 SUPERSTRUCTURE**

**11.1** The superstructure shall be designed for the design seismic forces as specified in para 9 plus other loads required in design load combinations.

**11.2** Under simultaneous action of horizontal and vertical accelerations, the superstructure shall have a factor of safety of at least 1.5 against overturning under DBE condition.

**11.3** The superstructure shall be secured, when necessary to the sub-structure in all zones through bearings possessing adequate vertical holding down devices and/or unseating prevention system for superstructure. These devices should be used for suspended spans also with the restrained portion of the superstructure. However, frictional forces in the devices should not be relied upon for preventing dislodging and jumping of superstructure.

# IRS Seismic Code

## 2.12 LONGITUDINAL FORCES

**14 SEATING WIDTH**

The bearing seat width $S_E$, in mm, between the end of girder and edge of sub-structure, Fig. 4 and minimum $S_E$ between the ends of girder at suspended joint should be not less that the following values:

$S_E$ = 203 + 1.67 *L* + 6.66 *H* for seismic Zones II and III

$S_E$ = 305+2.50 *L* + 10.0 *H* for seismic Zones IV and V

where

*L* = length of the superstructure to the adjacent expansion joints or to the end of superstructure. In case of bearings under suspended spans, it is the sum of the lengths of two adjacent portions of the superstructure. In case of single span bridges, it is equal to the length of the superstructure, in m.

*H* = average height of all columns or piers supporting the superstructure to the next expansion joint, for bearings at abutments, in m. It is equal to zero for single span bridges. For bearings at column or piers, it is the height of column or pier. For bearings under suspended spans, it is the average height of two adjacent columns or piers.

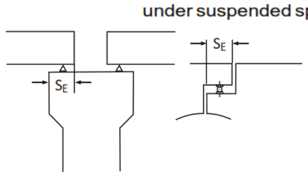


FIG. 4 BRIDGE SEATS ON PIER TOP OR AT SUSPENDED JOINT

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# IRS Seismic Code

## 22.1.1 Dynamic Active Earth Pressure

$$C_a = \frac{(1 \pm A_v)\cos^2(\phi - \lambda - \alpha)}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} \times \left[ \frac{1}{1 + \left\{ \frac{\sin(\phi + \delta)\sin(\phi - i - \lambda)}{\cos(\alpha - i)\cos(\delta + \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2$$

***22.1.1 Dynamic Active Earth Pressure Due to Backfill***

Figure 5 shows a wall of height *h*, inclined with an angle $\alpha$ with vertical, retaining dry/moist cohesionless earth fill. The dynamic active earth pressure exerted against the wall shall be:

$$(P_{A\gamma})_{dyn} = \frac{1}{2}\gamma h^2 C_a$$

where

$(P_{A\gamma})_{dyn}$ = dynamic total active earth pressure, in kN/m length of wall;

$\gamma$ = unit weight of soil, in kN/m$^3$; and

$h$ = height of wall, in m.

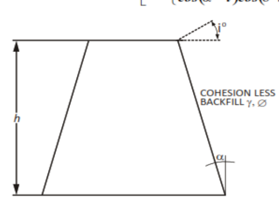


FIG. 5 CROSS-SECTION OF RETAINING WALL

rtical seismic coefficient - its direction being taken consistently throughout the stability analysis of wall and equal to $\frac{2}{3}A_h$;

$\phi$ = angle of internal friction of soil;

$\lambda$ = $\tan^{-1}\left[\frac{A_h}{1 \pm A_v}\right]$

$\alpha$ = angle with earth face of the wall makes with the vertical;

$i$ = slope of earth fill;

$\delta$ = angle of friction between the wall and earth fill; and

$Ah$ = horizontal seismic coefficient.

The expression of $(C_a)_{dyn}$ gives two values depending on the sign of *Av*. For design purpose higher of the two values shall be taken.

***Like Mononobe Okabe Equitation***

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# IRS Seismic Code

## 2.12 Dynamic Active Earth Pressure

| | | |
|---|---|---|
| Height of Soil Face, h | = | 8.00m |
| Unit weight of Soil, $\gamma$ | = | 17.80Kn/m³ |
| Angle of Internal Friction of Soil | = | 30.00deg. |
| $\delta$ = Angle of Friction between Soil and Abutment (backfill subjected to vibration) | = | 10.00deg. |
| i = Backfill Slope Angle | = | 0.00deg. |
| $\alpha$ = Wall Sloping Angle | = | 0.00deg. |
| Seismic Zone Co-efficient, Z, $\alpha_h$ | = | 0.120 |
| Vertical Acceleration Coefficient, $\alpha_v = \alpha_h / 2$ | = | 0.060 |
| $\phi$ = Angle of Friction of Soil | = | 30.00deg. |
| Horizontal Ground Acceleration, $A_{max} = Z * g$ | = | 1.18 |
| $\lambda$ = Arc Tan $(\alpha_h / (1 - \alpha_v))$ * 57.30 | = | 7.28deg. |
| $\sin(\phi + \delta) * \sin(\phi - \lambda - i)$ | = | 0.2483 |
| $\cos(\delta + \alpha + \lambda)\cos(i - \alpha)$ | = | 0.9549 |
| $\cos^2(\phi - \lambda - \alpha)$ | = | 0.8508 |
| $\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)$ | = | 0.9472 |
| $\psi$ = (1 + ( $\sin(\phi + \delta) * \sin(\phi - \lambda - i)$ / $\cos(\delta + \alpha + \lambda)\cos(i - \alpha)$)^05)^2 | = | 2.2799 |
| $C_{AD} = \dfrac{(1 \pm \alpha_v)(\cos^2(\phi - \lambda - \alpha))}{\cos\lambda \cos^2\alpha \cos(\delta + \alpha + \lambda)} * \psi$ | = | 0.418 |

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# IRS Seismic Code

## DUCTILE DETAILING

- B-1.1 Minimum grade of concrete should be M25 ($f_{ck}$ = 25 MPa).
- Longitudinal Bar 1% to 8% but not more than 6%
- Lap is not allowed in 2xD from the plastic hinge region

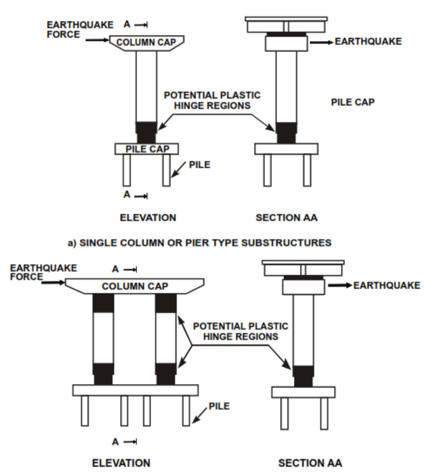


Fig. 13 POTENTIAL LOCATION OF PLASTIC HINGES IN SUBSTRUCTURES

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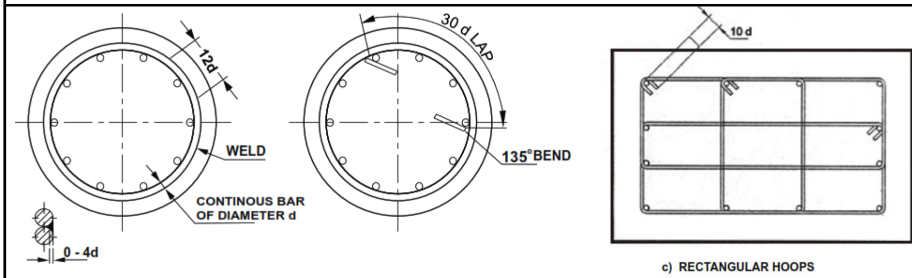
# IRS Seismic Code
## DUCTILE DETAILING




a) WELDING IN CIRCULAR HOOPS

b) LAPPING IN CIRCULAR HOOPS

c) RECTANGULAR HOOPS

FIG. 14 TRANSVERSE REINFORCEMENT IN COLUMN



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# IRS Seismic Code
## DUCTILE DETAILING

**B-5.5 Amount of Transverse Steel to be Provided**

**B-5.5.1** The area of cross-section, $A_{sh}$, of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sw} = 0.09SD_k\left[\frac{A_g}{A_c} - 1.0\right]\frac{f_{ck}}{f_y}$$

OR,

$$A_{sw} = 0.024SD_k\frac{f_{ck}}{f_y}$$

Whichever is the greater

where

| | | |
|---|---|---|
| $A_{sh}$ | = | area of cross-section of circular hoop; |
| S | = | pitch of spiral or spacing of hoops, in mm; |
| $D_k$ | = | Diameter of core measured to the outside of the spiral or hoops, in mm; |
| $f_{ck}$ | = | characteristic compressive strength of concrete; |
| $f_y$ | = | yield stress of steel (of circular hoops or - spiral); |
| $A_g$ | = | gross area of the column cross-section; and |
| $A_c$ | = | Area of the concrete core $\frac{\pi}{4}D_k^2$ |

**B-5.5.2** The total area of cross section of the bar forming rectangular hoop and cross ties, $A_{sh}$ to be used as special confining reinforcement shall not be less than

$$A_{sw} = 0.24Sh\left[\frac{A_g}{A_k} - 1.0\right]\frac{f_{ck}}{f_y}$$

Or

$$A_{sw} = 0.096Sh\frac{f_{ck}}{f_y}$$

Where

h = longer dimension of the rectangular confining

$A_k$ = hoop measured to its outer face; and area of confined core concrete in the rectangular hoop measured to its outer side dimensions

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# IRS Concrete Bridge Code Load Combination

LOADS TO BE TAKEN IN EACH COMBINATION WITH APPROPRIATE $Y_{fL}$
(Clauses 11.2 and 11.3)

| LOAD | | LIMIT STATE | $Y_{fL}$ TO BE CONSIDERED IN COMBINATION 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|---|---|
| Dead weight of concrete | | ULS<br>SLS | 1.25<br>1.00 | 1.25<br>1.00 | 1.25<br>1.00 | 1.25<br>1.00 | 1.25<br>1.00 |
| Superimposed dead load | | ULS<br>SLS | 2.00<br>1.20 | 2.00<br>1.20 | 2.00<br>1.20 | 2.00<br>1.20 | 2.00<br>1.00 |
| Wind | During erection | ULS<br>SLS | -<br>- | 1.25<br>1.00 | -<br>- | -<br>- | -<br>- |
| | with dead and superimposed dead loads only and for members primarily resisting wind loads. | ULS<br>SLS | -<br>- | 1.60<br>1.00 | -<br>- | -<br>- | -<br>- |
| | With dead plus superimposed dead plus other appropriate combination 2 loads. | ULS<br>SLS | -<br>- | 1.25<br>1.00 | -<br>- | -<br>- | -<br>- |
| | Relieving effect of wind | ULS<br>SLS | -<br>- | 1.00<br>1.00 | -<br>- | -<br>- | -<br>- |
| Earth quake | During erection | ULS<br>SLS | -<br>- | 1.25<br>1.00 | -<br>- | -<br>- | -<br>- |
| | With dead and superimposed dead loads only | ULS<br>SLS | -<br>- | 1.60<br>1.00 | -<br>- | -<br>- | -<br>- |
| | With dead plus superimposed dead plus other appropriate combination 2 loads. | ULS<br>SLS | -<br>- | 1.25<br>1.00 | -<br>- | -<br>- | -<br>- |
| Temperature | Restraint against movement except frictional | ULS<br>SLS | -<br>- | -<br>- | 1.50<br>1.00 | -<br>- | -<br>- |
| | Frictional restraint | ULS<br>SLS | -<br>- | -<br>- | -<br>- | 1.50<br>1.00 | -<br>- |
| | Differential temperature effect | ULS<br>SLS | -<br>- | -<br>- | 1.15<br>0.80 | -<br>- | -<br>- |
| Differential settlement | | ULS<br>SLS | As specified by engineer | | | | |
| Earth Pressure | Fill retained and or live load surcharge | ULS<br>SLS | 1.70<br>1.00 | 1.70<br>1.00 | 1.70<br>1.00 | 1.70<br>1.00 | -<br>- |
| | relieving effect | ULS | 1.00 | 1.00 | 1.00 | 1.00 | - |
| Erection temporary loads (when being considered) | | ULS | - | 1.30 | 1.30 | - | - |
| Live load on foot path | | ULS<br>SLS | 1.50<br>1.00 | 1.25<br>1.00 | 1.25<br>1.00 | -<br>- | -<br>- |
| Live load | | ULS<br>SLS | 1.75<br>1.10 | 1.40<br>1.00 | 1.40<br>1.00 | -<br>- | -<br>- |
| Derailment loads | | (As specified by bridge rules for combination 5 only) | | | | | |

NOTE 1-ULS : Ultimate limit state SLS : serviceability limit state
NOTE 2- Superimposed dead load shall include dead load of ballast, track, ballast retainer, precast footpath, wearing course, hand rails, utility services,kerbs etc
NOTE 3- Wind and earth quake loads shall not be assumed to be acting simultaneously.
NOTE 4- Live load shall also include dynamic effect, forces due to curvature exerted on track, longitudinal forces, braking forces and forces on parapets.

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Part 2

# IRS Substructure Rules

सत्यमेव जयते

GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS
(Railway Board)

INDIAN RAILWAY STANDARD

**CODE OF PRACTICE FOR THE DESIGN OF SUB-STRUCTURES AND FOUNDATIONS OF BRIDGES**

(BRIDGE SUB-STRUCTURES & FOUNDATION CODE)

ADOPTED –1936
FIRST REVISION -1985
SECOND REVISION -2013

(Incorporating Correction Slip Upto ACS 9 dt 06.07.2020)

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

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RESEARCH DESIGNS AND STANDARDS ORGANISATION
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# IRS Substructure & Foundation Rules

## *4.8 Clearance, 4.9 Free Board*

**4.8 CLEARANCE (C)**

**4.8.1** The minimum clearance for bridges excluding arch bridges, syphons, pipe culverts and box culverts from the water level of design discharge (Q) including afflux shall be in accordance with Table below **(ACS 1 dt 17.4.14)**

| Discharge in cumecs | Vertical clearance (mm) |
|---|---|
| 0-30 | 600 |
| 31-300 | 600-1200(Pro-rata) |
| 301-3000 | 1500 |
| Above 3000 | 1800 |

**4.9 FREE BOARD (F)**

- 4.9.1. The free-board from the water level of the design discharge (Q) to the formation level of the Railway embankment or the top of guide bund shall not be less than 1m. In cases where heavy wave action is expected, the free-board shall be increased suitably.

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# IRS Substructure & Foundation Rules

## *5.7 EARTH PRESSURE (EP)*

**5.7.1** All earth retaining structures shall be designed for the active pressure due to earth fill behind the structure. The general condition encountered is illustrated in (Fig.2)

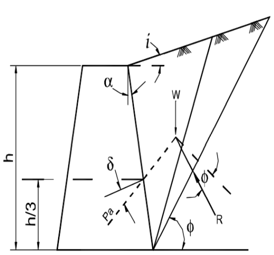


**Fig -2**

The active pressure due to earth fill shall

$P_a$ = ½ $Wh^2K_a$ where :-

$P_a$ = Active earth pressure per unit length of wall.

W = Unit weight of soil.

h = height of wall.

$\phi$ = angle of internal friction of back fill soil.

$\delta$ = angle of friction between wall and earth fill where value of $\delta$ is not determined by actual tests, the following values may be assumed.

(i) $\delta$ = 1/3 $\phi$ for concrete structures.

(ii) $\delta$ = 2/3 $\phi$ for masonry structures.

i = angle which the earth sirface makes with horizontal behind the earth retaining structure.

$\propto$ = angle which the back surface of earth retaining structure makes with vertical.

$K_a$ = Coefficient of static active earth pressure condition.

$$K_a = \frac{Cos^2(\phi-\alpha)}{Cos^2\alpha Cos(\alpha+\delta)\left[1+\sqrt{\frac{Sin(\phi+\delta)Sin(\phi-i)}{Cos(\alpha+\delta)Cos(\alpha-i)}}\right]^2}$$

$K_a$= Coefficient of Active Earth Pressure

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# IRS Substructure & Foundation Rules

## 5.7 EARTH PRESSURE (EP)

**5.7.1.6** The passive pressure $P_p$ due to the soil shall be calculated in accordance with the formula :

$$P_p = \frac{1}{2} Wh^2 K_p$$

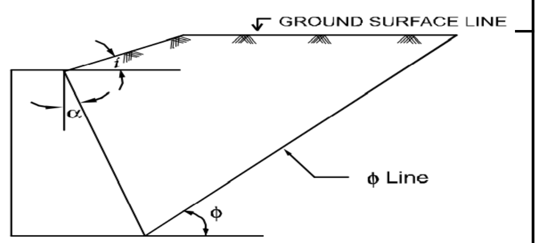
Where,

$P_p$ = Passive earth pressure per unit length of wall

W = Unit weight of soil

h = height from the base of the wall to the top surface of the soil.

$K_p$ = Coefficient of static passive earth pressure.




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

If at-rest earth pressure coefficient can be estimated by the following equation for normally consolidated soils:

Ko = 1 – sin φ



# IRS Substructure & Foundation Rules

## 5.7 EARTH PRESSURE (EP)

**5.7.1.8** Where such tests are not done, values of ϕ for granular soil may be assumed as given in Table -1.

**TABLE 1**

| Material | Loose state | Dense State |
|---|---|---|
| (a)Sand Coarse | 33 Degrees | 45 Degrees |
| (b) Sandy gravel | 35 " | 45 " |
| (c) Silty and fine sand | 30 " | 35 " |

## IRS Substructure & Foundation Rules
## 5.8 EARTH PRESSURE DUE TO SURCHARGE

5.8.1 Earth pressure due to surcharge on account of live load and dead loads (i.e. track, ballast etc.) shall be considered as equivalent to loads placed at formation level and extending up to the front face of ballast wall.

The surcharge due to live loads for the different standards of loading is indicated in Table-3.

**TABLE-3**

| Standard of loading | Surcharge,S (Kg/m) | Width of uniform distribution at formation Level, B (m) |
|---|---|---|
| DFC Loading (32.5t axle load) | 16,300 | 3.0 |
| 25t Loading-2008 | 13,700 | 3.0 |
| HM Loading - 1995 | 15,800 | 3.0 |
| Modified BG-1987 | 13,700 | 3.0 |
| Modified MG-1988 | 9,800 | 2.1 |
| MGML | 9,800 | 2.1 |
| NG'A' Class | 8,300 | 1.8 |



## IRS Substructure & Foundation Rules
## 5.8.2 Earth Pressure Due To Surcharge On Abutments

- The horizontal active earth pressure P due to surcharge, dead and live loads per unit length on abutment will be worked out for the following two cases.

Case-1 : When depth of the section h is less than (L-B).

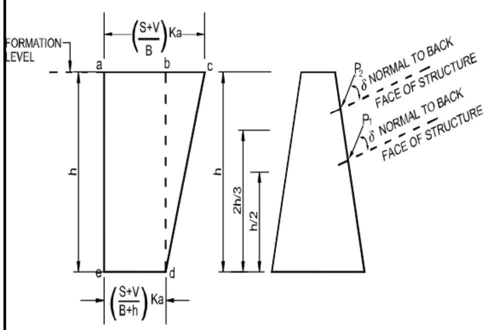
Case-2 : When depth of the section h is more than (L-B) .

Where : L= Length of the abutment; B= Width of uniform distribution of surcharge load at formation level; and h= Depth of the section below formation level.

# IRS Substructure & Foundation Rules
## 5.8.2 Earth Pressure Due To Surcharge On Abutments

**Case-1 : h $\leq$ (L-B)**

The active earth pressure diagrams are as under :




ACTIVE EARTH PRESSURE DIAGRAM

POINT OF APPLICATION AND DIRECTION OF ACTIVE EARTH PRESSURE

Whereas -

S = Live load surcharge per unit length

V = Dead load surcharge per unit length

$P_1$ =Force due to active earth pressure on 'abde'

$P_2$ = Force due to active earth pressure on 'bcd'.

$P_1 = \frac{(S+V)}{(B+h)} h.K_a$, acting at $\frac{h}{2}$ from section under consideration

$P_2 = \frac{(S+V)h^2 K_a}{2B(B+h)}$, acting at $\frac{2h}{3}$ from section under consideration.



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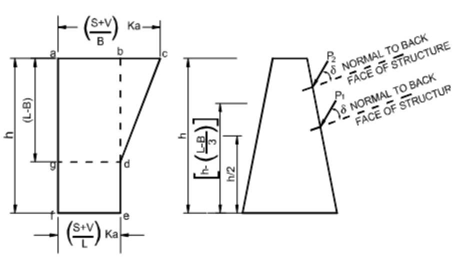
# IRS Substructure & Foundation Rules
## 5.8.2 Earth Pressure Due To Surcharge On Abutments

**Case-2 : h >(L-B)**

The active earth pressure diagrams are a under :




ACTIVE EARTH PRESSURE DIAGRAM

POINT OF APPLICATION AND DIRECTION OF ACTIVE EARTH PRESSURE

$P_1$ = Force due to active earth pressure on "abdefg"

$P_2$ = Force due to active earth pressure on "bcd"

$P_1 = \frac{(S+V)}{L} K_a h$

acting at $\frac{h}{2}$ from section under consideration

$P_2 = \frac{(S+V)(L-B)^2 K_a}{2BL}$ acting at

$\left[h-\left(\frac{L-B}{3}\right)\right]$ from section under consideration.

Where,

S = Live load surcharge for unit length

V = Dead load surcharge for unit length.

h = Height of fill.



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# IRS Substructure & Foundation Rules
# 5.8.2 Earth Pressure Due To Surcharge On Abutments

## Example for Case-2: h≥ (L-B)

*Earth Pressure Due to Surcharge on Abutments*

*(Static Condition)*

| | | |
|---|---|---|
| h = Depth of section below formation level | = | 8.00 m |
| L = Length of Abutment | = | 9.20 m |
| Angle of Internal Friction of Backfill Materials, φ | = | 30.00 deg. |
| Live Load Surcharge, S | = | 13,700 kg/m |
| B = Width of uniform distribution of surcharge load at formation level | = | 3.00 m |

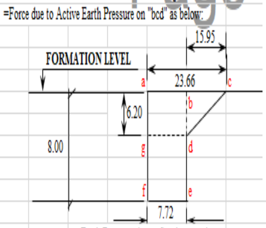
*h ≥ (L - B) : CASE-2*

**Calculation of Dead Load Surcharge, V**

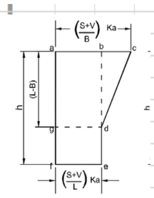
| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Weight of Rails: | | = | 60 | Kg/m | | | = | 7.06 Kn |
| Weight of Sleepers: 2 Nos. | | | | | | | | |
| Size (mm): | 2754 | x | 220 | x | 250 | | = | 14.30 Kn |
| Weight of Ballasts: | | | | | | | | |
| Size (mm): | 4800 | x | 1000 | x | 400 | | = | 60.29 Kn |
| | | | | | | Total: | w= | 81.65 Kn |
| Total Ballast width, Z= | | | | | | | = | 4.80 m |
| Longitudinal Distribution | | | | | | | | |
| Angle of internal friction | | | | | | | = | 30.00 deg |
| 90 - 45 + φ/2 | | = | 30.0 deg | | | | | |
| b= Width of the Sliding Wedge= tan(90 - 45 + φ/2)*h | | | | | | | = | 4.62 m |
| So, Dead Load Surcharge per meter Run, V=wb/z | | | | | | | = | 78.57 Kn/m |
| Live Load Surcharge per Unit Length, S from table | | | | | | | = | 134.40 Kn/m |
| Co-eff., Active Earth Press. (Ca) | | | | | | | = | 0.333 |

$P_1$ = Force due to Active Earth Pressure on "abdefg" as below:

$P_2$ =Force due to Active Earth Pressure on "bcd" as below:




Earth Pressure due to Surcharge only




ACTIVE EARTH PRESSURE DIAGRAM

**Foces due to Dead and Live Load Surcharge**

| | | | |
|---|---|---|---|
| Active Earth Pressure, Rectangular Part, = (S+V)/(L)*Ca | = | 7.72 | $kn/m^2$ |
| Active Earth Pressure at top, = (S+V)/B*Ca | = | 23.66 | $kn/m^2$ |
| Force due to Active Earth Pressure, Rectangular Part, $P_1$ = (S+V)/L*Ca*h | = | 61.73 | Kn/m |
| Acting at h/2 from section under consideration at base | = | 4.00 | m |
| Active Earth Pressure, Triangular Part | = | 15.95 | $kn/m^2$ |
| Force due to Active Earth Pressure, Triangular Part, $P_2$ = ((S+V)*(L-B)$^2$*Ka)/2BL | = | 49.43 | Kn/m |
| Acting at (h-((L-B)/3)) from section under consideration | = | 5.93 | m |

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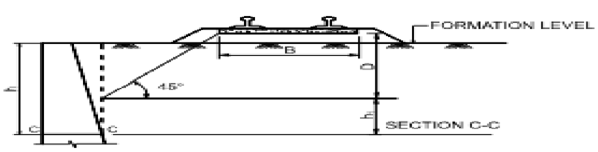
# IRS Substructure & Foundation Rules
# 5.8.3 Earth Pressure Due To Surcharge On Wing Wall

**5.8.3 Earth Pressure due to Surcharge on Return Walls** : The earth pressure due to surcharge on return walls of BOX type abutments may be assumed to be dispersed below the formation level at a slope of one horizontal to one vertical. The pressure due to live load and dead load surcharge shall be calculated by the formula:

$$P_1 = \frac{(S + V)h_1K_a}{(B + 2D)}$$

This pressure will be assumed to be acting at a distance of $h_1/2$ above the section considered as shown in Fig.5(a)

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# IRS Substructure & Foundation Rules
## 5.8.3 Earth Pressure Due To Surcharge On Wing Wall

***Earth Pressure Due to Surcharge on Return Walls***

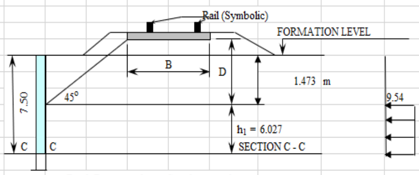
***Calculation of Dead Load Surcharge***

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Weight of Rails: | | | = | 60 | Kg/m | = | 3.53 | Kn |
| Weight of Sleepers: 2 Nos. | | | | | | | | |
| Size (mm): | 2754 | x | 220 | x | 200 | = | 5.72 | Kn |
| Weight of Ballasts: | | | | | | | | |
| Size (mm): | 4800 | x | 1000 | x | 300 | = | 22.61 | Kn |
| | | | | | Total: | = | 31.86 | Kn |
| ~~So, Dead Load Surcharge per meter Run~~ | | | | | | ~~=~~ | ~~28.74~~ | ~~Kn/m~~ |

Longitudinal Distribution

| | | | | | |
|---|---|---|---|---|---|
| Angle of internal friction | | | = | 30.00 | deg |
| 90 - 45 + φ/2 | = | 30 deg | | | |
| Width of the Sliding Wedge | | | = | 4.33 | m |
| So, Dead Load Surcharge per meter Run | | | = | 28.74 | Kn/m |

***Pressure Intensity due to Surcharge with both DL & LL***




Earth Pressure due to Surcharge only

| | | | |
|---|---|---|---|
| Live Load Surcharge per Unit Length, S | = | 134.40 | kn/m |
| Height of the Section below the Formation Level, h | = | 7.50 | m |
| Angle of Internal Friction of Backfill Materials, φ | = | 30.00 | deg. |
| Clear Distance between the Return Walls | = | 5700 | mm |
| Top of Rail in the Track | = | 10.690 | m |
| Formation Level / Top of Abutment / Top of Return Wall | = | 9.967 | m |
| Height of Rail | = | 0 | mm |
| Thickness of Sleeper | = | 0 | mm |
| Length of the Sleeper, B | = | 2754 | mm |
| Total Depth of Surcharge | = | 723 | mm |
| Dead Load Surcharge per Unit Length, V | = | 28.74 | kn/m |
| Co-eff., Active Earth Press. ($C_a$) | = | 0.333 | |
| Thickness of Cushion below the Sleeper | = | 723 | mm |
| Depth at which Surcharge Pressure acts from bottom of Sleeper, D | = | 1473 | mm |
| The Depth over which the Surcharge acts, $h_1$ | = | 6027 | mm |
| **The DL and LL Surcharge Pressure Intensity = (S+V)/(B+2*D)*Ca** | = | 9.54 | kn/m |
| **Acting at ($h_1$/2) from section C-C** | = | 3.014 | m |

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# IRS Substructure & Foundation Rules
## 5.9. FORCES DUE TO WATER CURRENT (WC)

5.9.2.1On piers parallel to the direction of water current the water pressure shall be calculated by the formula **:**

$P = KAV^2$ ,

P = Total pressure in kg due to water current.
A = Area in square metres of elevation of the part exposed to the water current.
V = The maximum mean velocity of current in metre per second
K = A constant having values for different shapes of piers as given in Clause 5.9.2.2.

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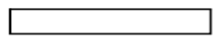
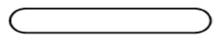
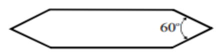
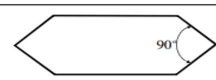
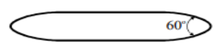
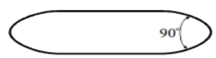
# IRS Substructure & Foundation Rules

## 5.9. FORCES DUE TO WATER CURRENT (WC)

TABLE 4

| S.No | Description | Figure | Value of K |
|---|---|---|---|
| 1. | Square-ended piers. |  | 79 |
| 2. | Circular piers or piers with semi-circular ends. |  | 35 |
| 3. | Piers with triangular cut-and ease-waters, the angle included between the faces being 60 degrees. |  | 37 |
| 4. | Piers with triangular cut-and ease-waters, the angle included between the faces being 90 degrees. |  | 47 |
| 5. | Piers with cut-and ease- waters of equilateral arcs of circle at 60 degrees. |  | 24 |
| 6. | Piers with arcs of the cut and ease waters intersecting at 90 degrees. |  | 26 |



# IRS Substructure & Foundation Rules

## 5.12 Seismic

- 5.12.1.1 Slab, box and pipe culverts need not be designed for seismic forces.
- For design of substructures of bridges in different zones, seismic forces may be considered as given below :-
- Zone I to III : Seismic forces shall be considered only in case of bridges of overall length more than 60m or spans more than 15m.
- Zone IV and V: Seismic forces may be considered for all spans.

## IRS Substructure & Foundation Rules

***5.12 Load Combination***

- (a) Combination I –
The worst possible combination of Dead load **(DL),** Live load (LL), Dynamic augment (I), Longitudinal forces (LF), Forces due to curvature and eccentricity of track (CF), Earth Pressure (EP), Forces due to water current (WC) and Buoyancy (B), Temperature effects where considered (TMP) and Effects due to resistance of expansion bearings to movements (EXB)

(b) Combination II - The worst possible combination of forces mentioned in combination I along with Wind pressure effect (WL).





## IRS Substructure & Foundation Rules

***5.12 Load Combination***

(c) Combination III - In case of bridges for which seismic forces have to be considered as per clause 5.12.1.1, the worst possible combination of forces in combination I plus forces and effects due to earth quake Seismic forces (SF). Wind pressure effect need not be taken into account when seismic effect is considered

(d) Combination IV - The worst possible combination of all loads and forces and effects which can operate on any part of the structure during erection. For bridges for which seismic forces have to be considered as per clause 5.12.1.1, either wind pressure effect or seismic effect whichever gives the worst effect need only be considered.

## IRS Substructure & Foundation Rules

**_5.15 PERMISSIBLE INCREASE IN STRESSES_**

***5.15.1 For combination of loads stated in clause 5.13, the permissible stresses on substructures shall be increased as follows : -***

Combination I ...... Nil

Combination II &III.... 33 %

Combination IV ...... 40%

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## IRS Substructure & Foundation Rules

5.15 PERMISSIBLE INCREASE IN Allowable Bearing Pressure

6.7 Permissible Increase In Allowable Bearing Pressure :
6.7.1 When the foundations are checked for combinations of loads as stated in clause 5.13, the allowable bearing pressures on foundations may be increased as follows :

Combination I - Nil
Combination II &III - 33 %
Combination IV - 40%

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Part 3

IRS Steel Bridge Code

PRB – 230
2000

सत्यमेव जयते

**Government of India**
**Ministry of Railways**
**(Railway Board)**

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

*(Steel Bridge Code)*

**ADOPTED – 1941**

**REVISED - 1962**
(Reprinted incorporating Addendum & Corrigendum Slip Nos.1 to21)

2017

**ISSUED BY**
**RESEARCH DESIGNS AND STANDARDS ORGANISATION**
**LUCKNOW – 226011**



## IRS Steel Bridge Code

Scope

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

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

# IRS Steel Bridge Code

## 3. LOADS, FORCES AND STRESSES

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

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# IRS Steel Bridge Code

## 3.2 Combination of Loads and Forces-

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

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

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

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# IRS Steel Bridge Code

## *.3 Primary and Secondary Stresses*

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

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



# IRS Steel Bridge Code

## *3 Primary and Secondary Stresses*

**TABLE – 1 TOTAL VARIATION IN ALLOWABLE STRESSES**

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

## IRS Steel Bridge Code

**3.8 Prestressing**

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

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



## IRS Steel Bridge Code

**3.6 Fluctuations of Stress (fatigue)**

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

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

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

# IRS Steel Bridge Code

## *3.6 Fluctuations of Stress (fatigue)*

### 14.6. Damage equivalence factors

**14.6.1** The damage equivalent factor for railway bridges should be determined from:

$\lambda = \lambda_1 * \lambda_2 * \lambda_3 * \lambda_4$

subject to the condition that $\lambda \leq \lambda_{max}$ where $\lambda_{max} = 1.4$

Where,

$\lambda_1$ is a factor that takes into account the damaging effect of traffic and depends on the base length of the longest loop of the influence line diagram

$\lambda_2$ is a factor that takes into account the annual traffic volume in million tons

$\lambda_3$ is a factor that takes into account the design life of the bridge in years

$\lambda_4$ is a factor to be taken into account when the bridge structure is loaded on more than one track

$\lambda_{max}$ is the maximum $\lambda$ value taking into account the fatigue limit

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# IRS Steel Bridge Code

## *3.6 Fluctuations of Stress (fatigue)*


**Table 14.6.2 (1): $\lambda_1$ for 25 T Loading**

| Span (m) | Train-1 | Train-2 | Train-3 | Train-4 | Train-5 | Train-6 | Train-7 | Train-8 | Train-9 | Train-10 | Train-11 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **0.50** | 1.30 | 1.34 | 1.45 | 1.28 | 1.45 | 1.44 | 1.48 | 1.53 | 1.36 | 1.09 | 0.88 |
| **1.00** | 1.29 | 1.32 | 1.43 | 1.28 | 1.43 | 1.43 | 1.48 | 1.53 | 1.35 | 1.08 | 0.89 |
| **1.50** | 1.28 | 1.31 | 1.42 | 1.27 | 1.42 | 1.42 | 1.47 | 1.52 | 1.35 | 1.07 | 0.90 |
| **2.00** | 1.27 | 1.30 | 1.40 | 1.27 | 1.40 | 1.41 | 1.47 | 1.52 | 1.35 | 1.06 | 0.90 |
| **2.50** | 1.26 | 1.29 | 1.38 | 1.26 | 1.37 | 1.39 | 1.46 | 1.51 | 1.34 | 1.05 | 0.91 |
| **3.00** | 1.24 | 1.28 | 1.36 | 1.25 | 1.34 | 1.38 | 1.45 | 1.50 | 1.34 | 1.04 | 0.92 |
| **3.50** | 1.22 | 1.26 | 1.34 | 1.23 | 1.32 | 1.36 | 1.44 | 1.49 | 1.33 | 1.03 | 0.93 |
| **4.00** | 1.20 | 1.25 | 1.32 | 1.22 | 1.30 | 1.34 | 1.43 | 1.47 | 1.31 | 1.02 | 0.94 |
| **4.50** | 1.18 | 1.23 | 1.30 | 1.21 | 1.28 | 1.31 | 1.42 | 1.46 | 1.30 | 1.00 | 0.95 |
| **5.00** | 1.17 | 1.21 | 1.28 | 1.19 | 1.26 | 1.27 | 1.41 | 1.45 | 1.28 | 0.99 | 0.96 |
| **6.00** | 1.12 | 1.17 | 1.24 | 1.15 | 1.23 | 1.21 | 1.39 | 1.42 | 1.24 | 0.96 | 0.97 |
| **7.00** | 1.08 | 1.14 | 1.19 | 1.11 | 1.18 | 1.16 | 1.37 | 1.39 | 1.19 | 0.93 | 0.98 |
| **8.00** | 1.05 | 1.12 | 1.17 | 1.09 | 1.16 | 1.12 | 1.38 | 1.37 | 1.14 | 0.91 | 1.00 |
| **9.00** | 1.03 | 1.11 | 1.15 | 1.07 | 1.13 | 1.08 | 1.38 | 1.36 | 1.09 | 0.90 | 1.02 |
| **10.00** | 0.96 | 1.03 | 1.12 | 1.15 | 1.08 | 1.10 | 1.37 | 1.37 | 1.08 | 0.89 | 0.99 |
| **12.50** | 0.89 | 0.98 | 1.05 | 1.06 | 1.03 | 1.02 | 1.32 | 1.32 | 1.00 | 0.86 | 1.01 |
| **15.00** | 0.87 | 0.92 | 1.00 | 1.01 | 1.05 | 1.00 | 1.30 | 1.31 | 0.99 | 0.83 | 1.02 |
| **17.50** | 0.82 | 0.86 | 0.94 | 0.93 | 1.04 | 0.94 | 1.24 | 1.24 | 0.93 | 0.78 | 0.98 |
| **20.00** | 0.83 | 0.86 | 0.99 | 0.94 | 1.07 | 0.89 | 1.13 | 1.09 | 0.89 | 0.79 | 1.01 |
| **25.00** | 0.76 | 0.86 | 0.93 | 0.85 | 1.08 | 0.87 | 1.13 | 1.11 | 0.87 | 0.75 | 0.99 |
| **30.00** | 0.77 | 0.82 | 0.84 | 0.80 | 1.09 | 0.88 | 0.98 | 1.10 | 0.87 | 0.69 | 0.96 |
| **35.00** | 0.73 | 0.75 | 0.78 | 0.76 | 0.87 | 0.86 | 0.93 | 1.09 | 0.86 | 0.66 | 0.90 |
| **40.00** | 0.66 | 0.68 | 0.70 | 0.73 | 0.85 | 0.78 | 0.84 | 1.07 | 0.78 | 0.63 | 0.75 |
| **45.00** | 0.64 | 0.66 | 0.68 | 0.77 | 0.81 | 0.77 | 0.82 | 1.01 | 0.78 | 0.63 | 0.65 |
| **50.00** | 0.62 | 0.63 | 0.65 | 0.77 | 0.80 | 0.75 | 0.79 | 0.91 | 0.76 | 0.62 | 0.66 |
| **60.00** | 0.59 | 0.60 | 0.62 | 0.77 | 0.77 | 0.74 | 0.77 | 0.78 | 0.74 | 0.60 | 0.67 |
| **70.00** | 0.58 | 0.59 | 0.60 | 0.75 | 0.76 | 0.75 | 0.77 | 0.78 | 0.75 | 0.61 | 0.64 |
| **80.00** | 0.56 | 0.58 | 0.59 | 0.66 | 0.76 | 0.74 | 0.76 | 0.77 | 0.75 | 0.60 | 0.63 |
| **90.00** | 0.56 | 0.57 | 0.58 | 0.64 | 0.76 | 0.72 | 0.75 | 0.76 | 0.75 | 0.58 | 0.63 |
| **100.00** | 0.55 | 0.56 | 0.58 | 0.61 | 0.76 | 0.73 | 0.74 | 0.75 | 0.74 | 0.55 | 0.63 |

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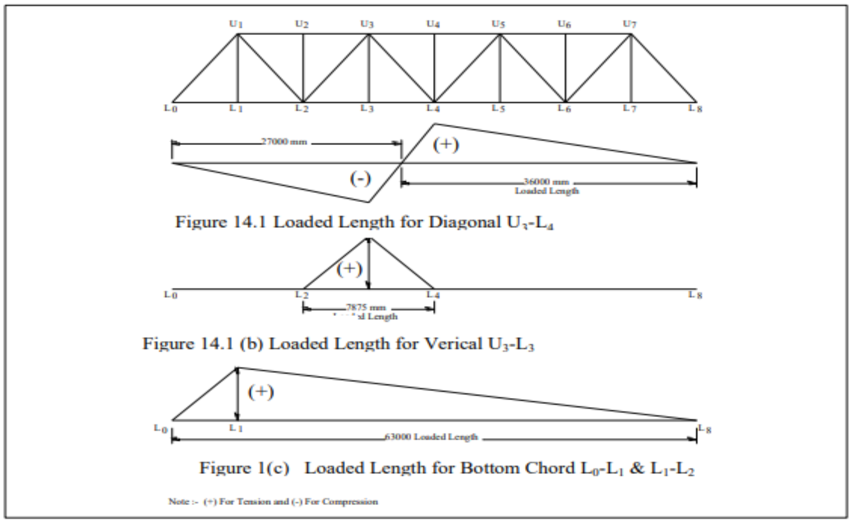
# IRS Steel Bridge Code

## 3.6 Fluctuations of Stress (fatigue)



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# IRS Steel Bridge Code

## 3.6 Fluctuations of Stress (fatigue)

**14.6.4** The value of $\lambda_2$, in terms of the annual volume of traffic may be obtained from the following expression where $T_a$ is the annual volume of traffic expressed in million tons:

$$\lambda_2 = 0.5193 * T_a^{0.2036}$$

**14.6.5** The value of $\lambda_3$, in terms of the design life may be calculated from the following expression where $L_D$ is the design life in years:-

$$\lambda_3 = 0.3899 * L_D^{0.2048}$$

**14.6.6** The value of $\lambda_4$, assuming 15% of the total traffic on both tracks crosses whilst on the bridge, shall be obtained from

$$\lambda_4 = 0.7926 * a^2 - 0.7280 * a + 0.9371$$

Where $a = \Delta\sigma_1 / \Delta\sigma_{1+2}$

$\Delta\sigma_1$ = Stress range at the section being checked due to train on one track.

$\Delta\sigma_{1+2}$ = Stress range at the same section due to train load on two tracks.

The values of $\lambda_4$ may be calculated for other proportions of crossing traffic from

$$\lambda_4 = \sqrt[5]{n + (1-n)\left[a^5 + (1-a)^5\right]}$$

Where, n is the proportion of traffic that crosses simultaneously on the bridge.



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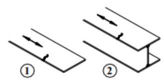
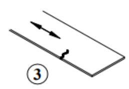
# IRS Steel Bridge Code

## 3.6 Fluctuations of Stress (fatigue)

**Table G-II.1 Non-welded details**

| Detail category | Constructional Detail | Description | Requirements |
|---|---|---|---|
| 160 | NOTE: The fatigue strength curve associated with category 160 is the highest. No detail can reach a better fatigue strength at any number of cycles.  | Rolled and extruded products:<br>1)Plates and flats<br>2)Rolled sections | Details 1) to 2)<br>Sharp edges, surface and rolling flaws to be improved by grinding until removed and smooth transition achieved. |
| 125 |  | 3) Sheared or gas cut plates:<br>Machine gas cut or sheared material subsequently dressed to remove all edge discontinuities. | 3) All visible signs of edge discontinuities to be removed. The cut areas are to be machined or ground and all burrs to be removed.<br>Any machinery scratches for example from grinding operations can only be parallel to the stresses.<br>Detail 3<br>- Re-entrant corners to be improved by grinding (slope < ¼) or evaluated using the appropriate stress concentration factors.<br>- No repair by weld refill. |

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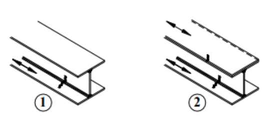
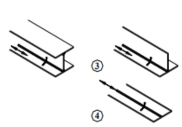
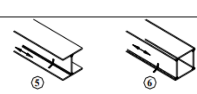
# IRS Steel Bridge Code

## 3.6 Fluctuations of Stress (fatigue)

**Table G-II. 2 Welded built-up sections**

| Detail category | Constructional Detail | Description | Requirements |
|---|---|---|---|
| 125 |  | Continuous longitudinal welds:<br>1) Automatic butt welds carried out from both sides.<br>2) Automatic fillet welds Cover plate ends to be checked using detail 5) or 6) Table G-II.5. | Details 1) to 2)<br>No stop/start position is permitted except when the repair is performed and inspection is carried out to verify the proper execution of the repair. |
| 112 |  | 3) Automatic fillet or butt weld carried out from both sides but containing stop/start positions.<br>4) Automatic butt welds made from one side only, with a continuous backing bar, but without stop/start positions. | 4) When this detail contains stop/start positions category 100 to be used. |
| 100 |  | 5) Manual fillet or butt weld.<br>6) Manual or automatic butt weld carried out from one side only, particularly for box girders. | 6) A very good fit between the flange and web plates is essential. The web edge to be prepared such that the root face is adequate for the achievement of regular root penetration without break-out. |

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# IRS Steel Bridge Code

## 3.6 Fluctuations of Stress (fatigue)

4.3.1 Detail category: The designation given to a particular structural detail for a given direction of stress fluctuation to indicate which fatigue strength curve is applicable for fatigue assessment. This is denoted by a number which represents the magnitude in N/mm2 of the stress range which is associated with an endurance of 2 million cycles for that particular category.

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

$$\gamma_{Ff} = 1.00$$

$$\gamma_{Mf} = 1.15$$

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## Example: Fatigue Stress Calculation.

| Fatigue stress Calculation | | | | Reference |
|---|---|---|---|---|
| **For Cross Girder** | | | | |
| Calculation of Fatigue according to the IRS Code | | | | |
| Connection Category | = | 125 | N/mm2 | Table G-12 |
| N | = | 2000000 | Cycle | |
| Stress range , ΔσR | = | 125 | N/mm2 | |
| Damage Factor, λ=λ1*λ2*λ3*λ4 | | | | CL 14.6.1 |
| λ1 for 32.5 t loading and span 20 m | = | 1.18 | | Table 14.6.2 (2) |
| | | | | 32.5t Loading |
| $T_0$= GMT= 2.08 Per Train @ 24 Train/ Day | = | 50.0 | GMT | |
| λ2=0.5193*To^0.2036 | = | 1.1517 | | 14.6.4 |
| | | | | Cl. 3.6.5 |
| Design Life, LD | = | 100 | Years | correction slip 18 |
| | | | | GMT will be 50 |
| λ3=0.3899*LD^0.2048 | = | 1.0 | | 14.6.5 |
| λ4 | = | 1 | For Single Track | 14.6.6 |
| Damage Factor λ=λ1*λ2*λ3*λ4 | = | 1.361 | | |
| λmax | = | 1.4 | | |
| Partial Factor | = | 1.15 | Claus Clause 7.4 , Apendix G | |
| OK | | | | |
| **λ will be taken not more than 1.40** | = | 1.361 | | |
| Allowable Fatigue stress= $\frac{\text{Stress Range (125)}}{\text{Damage Factor (1.361) x Fartial Factor Strees (1.15)}}$ | | | | |
| | = | 79.88 | N/mm2 | |
| For Plate Thickness, t | = | 35 | mm | |
| Stress will be modified | | | | |
| $\Delta\sigma R_t$=ΔσRt25*(25/t)^0.2 | = | 74.68 | N/mm2 | CL. 10.2.2.3 |
| **Allowable fatigue stress** | = | 74.68 | Mpa | |

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## 2.5 FORCES DUE TO CURVATURE AND ECCENTRICITY OF TRACK

4.16 Camber

4.16.1 Beams and plate girder spans up to and including 35 m (115 ft) need not be cambered.

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

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

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## IRS Steel Bridge Code

***4.16 Camber***

4.16 Camber

4.16.1 Beams and plate girder spans up to and including 35 m (115 ft) need not be cambered.

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

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

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# IRS Steel Bridge Code

## 4.17 DEFLECTION

4.17 Deflection- For permanent installation other than foot-over-bridges the ratio of deflection to length of the girder shall not exceed 1/600.

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

Note:-

With the specific sanction of the Board, the limit of 1/600 may be exceeded for girders in permanent installations.





# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

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

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

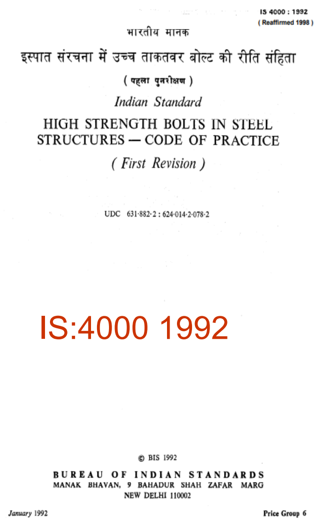
7.12.2 Joints with HSFG bolts shall be designed only as friction type joints. Bearing type joints with HSFG bolts shall not be provided.

7.12.3 Short/long slotted holes shall not be allowed.

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

# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)


IS 4000 : 1992
( Reaffirmed 1998 )

भारतीय मानक

इस्पात संरचना में उच्च ताकतवर बोल्ट की रीति संहिता

( पहला पुनरीक्षण )

*Indian Standard*

HIGH STRENGTH BOLTS IN STEEL STRUCTURES — CODE OF PRACTICE

*( First Revision )*

UDC 631·882·2 : 624·014·2·078·2

IS:4000 1992

© BIS 1992

BUREAU OF INDIAN STANDARDS
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI 110002

*January* 1992

Price Group 6

**Grade of HSFGB**

- 10.9 Grade means Ultimate strength 1000 mPa and Yield Strength 90% of Ultimate Strength 900 mPa.
- 8.8 Grade means Ultimate strength 800 mPa and Yield Strength 80% of Ultimate Strength 640 mPa.

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# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

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

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

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

7.12.3 Short/long slotted holes shall not be allowed.

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

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# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

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

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

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

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

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

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

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

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

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# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

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

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

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

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# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)

7.12.6.2 For joints subjected to shear and tensile force.

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

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

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

(a) 1.4 for normal loads

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

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



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# IRS Steel Bridge Code

## 7.12 HIGH STRENGTH GRIP BOLT (HSFGB)


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

**Table XIII**



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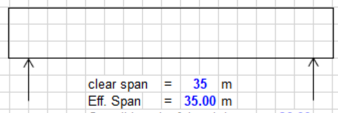
## Example: Design of Stringer (open Deck) Cont.

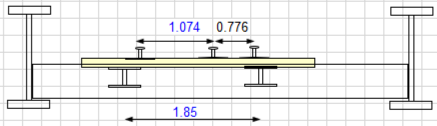
Subject : Design of Semi Through Girder - 35m Span (Broad Guage) with Footpath

Calculation of Stringer (Open Deck) | Reference

1 **Design of Stringer**



clear span = 35 m
Eff. Span = 35.00 m
Overall length of the girder = 36.00 m




| | | | |
|---|---|---|---|
| Spacing of the cross girders | = | 2.625 | m |
| Effective Span of the stringer | = | 2.625 | m |
| Dead load of stringer | | | |
| ( Track Load + Self wt.) ( 0.65 + 0.13 ) t/m | = | 0.78224 | t/m |
| Dead Load on the stringer | = | 2.05338 | t |
| % of Load on the stringer due to Meter Guage Loading | = | 70.973 | % |
| coefficient of Impact load ( 1.5 L) | = | 0.955 | |

| Loads due to Live Load | B.M(W) | Shear(P) | |
|---|---|---|---|
| **LL for MMG-1988 -EUDL (T)** | 32.3 | 43.28 | t |
| LL per Girder = 70.97 % of one track load | 22.924 | 30.717 | t |
| **Live Load for 32.5t DFC load** | 65 | 83.42 | t |
| LL per Girder = 50% of one track load | 32.5 | 41.71 | t |
| **B.G Governs in the Design** | | | |

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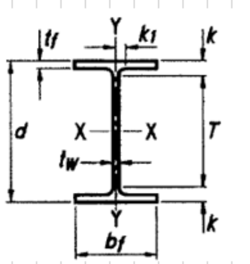
## Example: Design of Stringer (open Deck) Cont.

| | | | |
|---|---|---|---|
| **B.G Governs in the Design** | | | |
| Impact Load per stringer | 31.04 | 39.83 | t |
| Dead Load of the stringer | 2.053 | 2.053 | t |
| Number of Stringer | 2.000 | | |
| Total Load on the Stringer | **65.59** | **83.60** | t |
| Max. Bending Moment | **21.52** | | tm |
| Max. Shear Force | | **41.80** | t |
| Max. Bending Moment for live load only | 20.849 | | tm |
| Max. Shear force for live load only | | **40.77** | t |

<u>Sectional Properties of the Member</u>




From AISC Manual

| | | | | | |
|---|---|---|---|---|---|
| Member | W | 16 | x | 89 | |
| Yield Stress | 50 | ksi | | 345 | Mpa |
| Weight | 89 | lb/ft | | 132 | Kg/m |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Area | | | | | = | 169.032 | $cm^2$ |
| Overall Depth | | | | | = | 42.545 | cm |
| Web Thickness | | | | | = | 1.3335 | cm |
| Flange width | | | | | = | 26.3271 | cm |
| Flange Thickness | | | | | = | 2.2225 | cm |
| Mom. of Inertia | Ixx | | | | = | 54110.1 | $cm^4$ |
| Ybs | = | 21.3 | cm | Zbs | = | 2543.66 | $cm^3$ |
| Yts | = | 21.3 | cm | Zts | = | 2543.66 | $cm^3$ |
| Mom. of Inertia | Iyy | | | | = | 6784.57 | $cm^4$ |

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## Example: Design of Stringer (open Deck) Cont.

**Permissible Stresses :** Compressive stresses

| | | | | |
|---|---|---|---|---|
| Bending Compression Stress considering lateral Torsional Buckling | | | Cs Values | cl: 3.9 of IRS Steel Bridge Code |
| case | 1 | Flanges have equal moment of Inertial about y axis | A | |
| case | 2 | Moment of Inertia of the compression flange about y -y axis exceeds that of Tension flange | $A+k_2B$ | |
| case | 3 | Moment of Inertia of the Tension flange about y-y axis exceeds that of compression flange | $(A+k_2B)Yc/Yt$ | |

where

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

Yc = Distance from N.A of girder to extreme fibre in compression

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

Yt = Distance from N.A of girder to extreme fibre in Tension

k2 = A coefficient to allow for inequality of tension and compression flanges
te = Effective Thickness of Flange

| | | | | | |
|---|---|---|---|---|---|
| Moment of Inertia of the Top flange about YY axis | | = | 3379.63 | $cm^4$ | |
| Moment of Inertia of the Bottom flange about Yyaxis | | = | 3379.63 | $cm^4$ | |
| It comes in Case 1,Cs = A | | | | | IRS SBC |
| Effective Length of the compression Flange | | = | 2625 | mm | cl: 5.4.4 |
| Radius of the Gyration about YY axis | ry | = | 63.35 | mm | |
| Overall Depth of the Section | D | = | 425.45 | mm | |
| Effective Thickness of the Flange K1 = 1 | te | = | 22.225 | mm | Table V cl:5.3 |
| | l / ry | = | 41.4365 | | |
| | D/te | = | 19.1429 | | |
| | A | = | 173.24 | $kg/mm^2$ | |
| | B | = | 155.931 | $kg/mm^2$ | |
| Ratio M.I of Comp. flange to M.I of comp + tension flange | Rm | = | 0.5 | | |
| | k2 | = | 0 | | Table VI |
| | Cs | = | 173.236 | $kg/mm^2$ | |





## Example: Design of Stringer (open Deck) Cont.

| | | | | | |
|---|---|---|---|---|---|
| | | | | | IRS SBC |
| Allowable Bending Compressive stress | Pbc | = | 21.2 | $kg/mm^2$ | Table VIII |
| for equivalent steel considered | **IS 2062 : Grade E350 (Fe490 W)** | | | | |
| Max. Permissible Bending Stress | (if thickness is 20mm or less) | = | 22.5 | $kg/mm^2$ | Table II |
| | (if thickness is more than 20mm | = | 21.9 | $kg/mm^2$ | of IRS SBC |
| Max. Permissible Shear Stress | (if thickness is 20mm or less) | = | 13 | $kg/mm^2$ | Table II |
| | (if thickness is more than 20mm | = | 12.7 | $kg/mm^2$ | of IRS SBC |
| **Stresses with Fatigue Consideration** | | | | | |
| fmin / fmax | = 2.05338 / 83.60 | = | 0.025 | | |
| Class of Connection | = Category 160 No of Cycles | = | 2 Million | | Appendix "G' |
| Allowable Fatigue Stress in Compression | | = | 18.1 | $kg/mm^2$ | of IRS SBC |
| Allowable Fatigue Stress in Tension | | = | 10.13 | $kg/mm^2$ | |
| Considering all above | | | | | |
| Actual Bending Stress | Compression | = | 8.46116 | $kg/mm^2$ | |
| Allowable Bending Stress | | = | 18.1 | $kg/mm^2$ | |
| | | | O.K | | |
| Aactual Bending Stress | Tension | = | 8.46116 | $kg/mm^2$ | |
| Allowable Bending Stress | | = | 10.13 | $kg/mm^2$ | |
| | | | O.K | | |
| Actual Shear Stress | | = | 8.227 | $kg/mm^2$ | |
| Permissible shear stress | | = | 13 | $kg/mm^2$ | |
| | | | O.K | | |

## Example: Design of Stringer (open Deck) Cont.

| | | | |
|---|---|---|---|
| **Design of Connections** | | | |
| **Design of Cleat Angle to Stringer Beam** connection is by HSFG Bolts | | | IS4000 |
| Propose Cleat anlge 150 x 150 x 9.53 mm 5.9 x 5.9 x 0.375 in | | | |
| Dia of the Bolt Proposed to use | = 24 | mm | |
| Dia of the hole | = 26 | mm | |
| Grade of the Bolt | = 8.8 | | |
| **Considering Bearing Type of joint** | | | Table II |
| Max. Permissible force in the bolt | = 73.2 | Kn | |
| Limitation capacity of the bolt fy = 350 t = 10 mm | = 96.0 | Kn | |
| Capacity of the Bolt | = **73.2** | Kn | |
| No of bolts required | = **6** | No's | |
| Hence Provide 6 No's 24 mm dia HSFG Bolts | | | |
| **Considering Friction Type of joint** | | | |
| Friction coefficient | = 0.3 | | |
| No of Interfaces | = 2 | | |
| Factor of safety | = 1.4 | | |
| Minimum Bolt Tension | = **212** | Kn | |
| Permissible force in the bolt | = **90.8571** | Kn | |
| Limitation capacity of the bolt fy = 350 t = 10 mm | = 96.0 | Kn | |
| Capacity of the Bolt | = **90.9** | Kn | Table III |
| No Bolts required | = **5** | No's | |
| Hence Provide 5 No's 24 mm dia HSFG Bolts | | | |
| **Considering Fatigue for bolts** | | | |
| Category | = 100 | Mpa | Table 9.1 |
| Material safety | = 1.15 | | of IRS SBC |
| λmax | = 1.4 | | |
| Maximum Allowable stress per bolt | = 62.11 | MPa | |
| Capacity of the Bolt | = 28.10 | kN | |
| Shear force for live load | = 40.77 | t | |
| Nos of face | = **2** | | |
| Nos of bolts required | = **8.00** | Nos. | |
| Number of bolts provided in each row | = **4** | Nos. | |
| Number of rows | = **2** | Nos. | |
| Provide Edge distance from C.G of the Bolt = 40 mm O.K | | | |
| = 60 mm O.K | | | |
| Length of the Cleat Angle = ( 40 + 180 + 40 ) | = 260 | mm | |



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## Example: Design of Stringer (open Deck)

| | | | |
|---|---|---|---|
| **Design of cleat Angle to Cross Beam** | | | |
| Force on the Member | | = 41.80 | t |
| Eccentricity of the force to the Member | | = 0.05 | m |
| Moment on the Member | | = 2.09 | tm |
| Dia of the Bolt Proposed to use | | = **24** | mm |
| Dia of the hole | | = 26 | mm |
| Grade of the Bolt | | = **8.8** | |
| **Considering Bearing Type of joint** | | | |
| Max. Permissible force in the bolt | | = 73.2 | Kn |
| Limitation capacity of the bolt fy = 350 t = 10 mm | | = 96.0 | Kn |
| Capacity of the Bolt | | = **73.2** | Kn |
| No of bolts required | | = **6** | No's |
| Hence Provide 2 - 8.00 No's 24 mm dia HSFG Bolts on either side of stringer | | | |
| Shear Force on the each Bolt | V | = 26.1243 | Kn |
| Tensile force on the Bolt due to moment | Pt | = 40.1912 | Kn |
| Permissible Tensile force of the Bolt | Pto | = 127 | Kn |
| By condition V / Vo + Pt / Pto | | = 0.22752 | **<=1, O.K** |
| **Considering Friction Type of joint** | | | |
| Friction coefficient | | = 0.3 | |
| No of Interfaces | | = 1 | |
| Factor of safety | | = 1.4 | |
| Minimum Bolt Tension | | = **212** | Kn |
| Permissible force in the bolt | | = **45.43** | Kn |
| Limitation capacity of the bolt fy = 350 t = 10 mm | | = 96.0 | Kn |
| Capacity of the Bolt | | = **45.4** | Kn |
| No Bolts required | | = **10** | No's |
| Hence Provide 16 No's 24 mm dia HSFG Bolts | | | |
| Calculated Shear on the bolt | V | = 26.1243 | Kn |
| Calculated Tension on the Bolt | Pt | = 40.1912 | Kn |

As per the expression $\dfrac{\text{Calculated shear}}{\text{slip factor} \times \text{N.O.I}} \leq \dfrac{(\text{Proof load} - \text{Cal.Tension} \times F)}{\text{Factor of safety}}$

$$\frac{26.1243}{0.3 \times 1} \leq \frac{212 - 40.191 \times 2.0}{1.4}$$

87.081 ≤ 94.0125 Kn **O.K**



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## Table II

TABLE II -- BASIC PERMISSIBLE STRESSES IN STRUCTURAL STEEL

| Description | Mild steel to IS: 226 and IS: 2062 with yield stress of | | | | High tensile steel grade 58-HTC to IS: 961 with yield stress of | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 26 kg/ mm² | 16.5 ton/in² | 24 kg/ mm² | 15.2 ton/in² | 36 kg/ mm² | 22.9 ton/in² | 35 kg/ mm² | 22.0 ton/in² | 33 kg/ mm² | 21.0 ton/in² |
| 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 |
| **Parts in Axial Tension** On effective sectional area ... | 15.4 | 9.8 | 14.2 | 9.0 | 21.3 | **13.5** | 20.7 | 13.1 | 19.5 | 12.4 |
| **Parts in Axial Compression** on Effective gross section ... | | | | | **See** | **Clause** | 3.8 | | | |
| **Parts in bending (Tension or Compression).** On effective sectional area for extreme fibre stress – (i)For plates, flats, tubes, rounds, square and similar sections. | 17.0 | 10.8 | 15.7 | 10.0 | **23.5 See** | **14.9 also** | 22.8 **Clause3.9** | 14.5 | 21.6 | 13.7 |
| (ii) For rolled beams, channels, angles and tees, and for plate girders with single or multiple webs with $d_1$/t not greater than 85 for steel to IS:226 and IS:2062 $d_1$/t not greater than 75 for steel to IS:961. | 16.3 | 10.4 | 15.0 | 9.5 | 22.5 **See** | **14.3 also** | 21.9 **Clause 3.9** | 13.9 | 20.6 | 13.1 |
| ii) For plate girder with single or multiple webs with : $d_1$ /t greater than 85 for steel to IS:226 and IS:2062, d/t greater than 75 for steel to IS:961. | 15.4 | 9.8 | 14.2 | 9.0 | 21.3 **See** | **13.5 also** | 20.7 **Clause 3.9** | 13.1 | 19.5 | 12.4 |

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

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## Table II

TABLE II – (Contd.)

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

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

Contd....

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## Table II

Table II (Contd…)

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

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

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



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

TABLE VI – VALUES OF $K_2$

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

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

TABLE V – VALUES OF $K_1$

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

## Table V, VI, VII

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

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

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

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

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



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TABLE VIII – ALLOWABLE WORKING STRESS $P_{bc}$.
FOR DIFFERENT VALUES OF CRITICAL STRESS $C_s$.
(SEE ALSO CLAUSE 3.7 TABLE II)

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

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

## Table VIII

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# Questions?
# Thank you

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