Government of India
Ministry of Railway
(Railway Board)

No. 2012/Proj./HMR/30/4 New Delhi, dated 11 January, 2013

Managing Director,
Hyderabad Metro Rail Limited,
Metro Rail Bhawan, Saifabad,
Hyderabad-500 004.

Sub: Hyderabad Metro Rail Project – Design Basis Report.

Ref: MD/Hyderabad Metro Rail Limited letter No. 402/MD/CGM/HMR/2012/91 dated 10.10.2012

Design Basis Report Revision-H forwarded by Hyderabad Metro Rail Limited vide letter referred above has been examined in consultation with RDSO and the same has been approved by the Competent Authority.

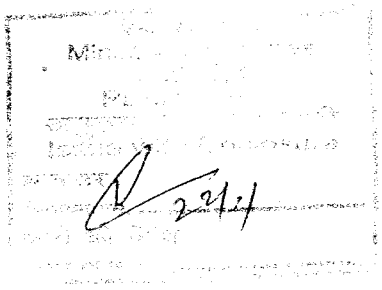
O/C 11.1.13

(Mohit Lila)
Jt. Director / Works Plg.
Railway Board

Encl: Copy of approved DBR.

Copy to: ED/UT&HS, RDSO, Lucknow w.r.t. his letter No. UTHS/63 dated 21.11.2012 along with a copy of approved DBR.

D.T.S 22/1/13

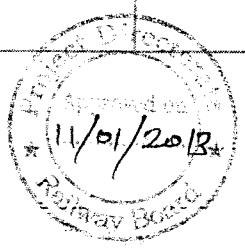


Hyderabad Metro Rail

Design Basis Report

Section 2: Viaduct

REVISION	DATE	PREPARED & CHECKED	REVIEWED BY	APPROVED BY
A	3rd March 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
B	19th March 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
C	11th April 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
D	13th April 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
E	14th June 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
F	25th July 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
G	02nd Sept 2011	Jatinder Singh Pahuja	GVR Raju	David E. Race
H	28th Sept. 2012	M. Sundara Reddy	Pradeep Mahajan	David E. Race



Hyderabad Metro Rail

Design Basis Report

Section 2 : Viaduct

Revision Record Sheet

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A	3rd March 2011	All	First Issue	David E. Race
B	19th arch 2011	All	L&TMRHL comments incorporated Second Issue	David E. Race
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D	13th April 2011	All	L&TMRHL comments incorporated Fourth Issue	David E. Race
E	14th June 2011	All	IE's comments & necessary changes incorporated Fifth Issue	David E. Race
F	25th July 2011	All	IE's comments & necessary changes incorporated Sixth Issue	David E. Race
G	02nd Sept 2011	All	IE's comments & necessary changes incorporated Seventh Issue	David E. Race
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1 INTRODUCTION

1.1 Project Description

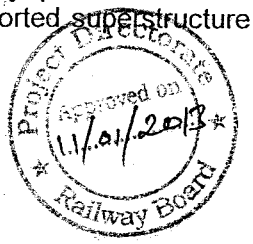
1.1.1 Hyderabad Metro Rail System Consists of three lines:

- Line 1 Miyapur–LB Nagar for a length of 29.318 km with 27 Stations
- Line 2 JBS–Chrminar–Falknuma for a length of 14.89 km with 16 Stations
- Line 3 Shilparamam–Nagole for a length of 26.742 km with 23 Stations.

1.1.2 Rake interchangeability planned between Corridors I & II at Ameerpet and between Corridors II & III at Parade grounds.

1.2 Aim of this section of the Design Basis Report

1.2.1 This Report is being submitted highlighting the proposed design parameters and methodology to be adopted for the project. All design works shall be performed taking into consideration this Design Basis Report. Checks for standard superstructures and special (continuous) structure during construction including temporary load effects during construction shall be required to be detailed separately depending on the construction scheme opted for the same. This design basis report does not cover design requirements for any cable supported superstructure for which supplementary part to this design basis report is required to be made (if any cable supported superstructure is proposed) based on exact configuration of superstructure.



2 PROPOSED STRUCTURAL SYSTEMS

2.1.1 The work of whole stretch can be broadly classified into two parts namely, Viaduct & Ramp.

2.2 Superstructure System of Viaduct

2.2.1 The superstructure of a large part of the viaduct comprises of simply supported spans. However at major crossing / over or along existing bridge, special continuous bridges / steel girder – concrete deck composite unit will be provided.

2.2.2 The super structure on the main lines will be accommodating the two tracks situated at 4.15 m c/c throughout both in straight and curved alignments.

2.2.3 The following types of practical options of Superstructures are considered for design

- Post tensioned Segmental Single Cell Box Girder (for dual track) with internal prestressing
- Pre/Post tensioned I Girder with cast-in-situ deck slab (may be used at pocket track or Rake interchange zones)

2.3 Bearing System

2.3.1 Considering the span configuration and safety aspects of the structural system (in normal and seismic condition), it is proposed to adopt elastomeric bearing placed underneath Box girder for transfer of vertical forces and shear key (protruding above the pier head) for transfer of in-plane forces.

2.3.2 In such case, all horizontal longitudinal loads (traction / braking loads and longitudinal seismic loads) are taken by restraining device which is a combination of horizontal tie-bars connecting the deck with the concrete shear key. Such arrangement of tie the superstructure will be only done at one end of span while allowing unrestrained longitudinal movement at the other end of the span by adequate sliding surfaces in contact with the deck.

2.3.3 The shear key at both end of span would also take the transverse loads (transverse seismic, centrifugal, wind...). The shear key would anyway be also designed at each pier head to prevent any large movements of deck due to unexpectedly larger seismic loading (in both directions).

2.3.4 Elastomeric bearing shall be designed as per UIC-772-R.

2.3.5 In case loads / movements are high and elastomeric bearings cannot be designed, only then pot bearings shall be used. All the pot bearings, if any, will be designed as per IRC: 83 Part-III.

2.3.6 If found necessary, a hold-down device (with active force) connecting the deck and the pier head shall be placed in order to prevent the deck from overturning. The hold-down device may be integrated in the pot – bearing system or be a separate system constituted of bars embedded in pier-cap and the viaduct with appropriate details permitting translation/rotation. Other systems can also be foreseen.

2.3.7 Due to the lack of appropriate guidelines in Indian codes, the design criteria for hold-down device (upward force limit requiring hold-down device, design

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formulas,) shall be taken from the latest international practice (AASHTO, MOTC codes).

2.3.8 While finalizing the proposed bearing system, it shall be kept in mind that accessibility and replacement of each part of bearing are of paramount importance as the design life of bearings is shorter than that of the structure. Keeping in view the above cited criteria, all the bearings and the pier caps shall be detailed for replacement of bearings in the future. Elastomeric bearing will have a design life of 10years while pot bearing (if adopted) will have a design life of 25 years.

2.4 Proposed Substructure System

2.4.1 Generally viaduct superstructure is supported on single cast-in-place RC pier.

2.4.2 For the standard spans with Box Superstructure, the pier gradually widens at the top to support the bearing under / close to the box webs. Preferably pier cap shall be so profiled and detailed that it can be cast along with pier shaft in one go.

2.4.3 However if I Beam with Cast in situ slab superstructure is provided, hammer head shaped pier cap shall be provided. Such hammer head pier cap shall be either cast-in-situ or solid precast member with hole in centre (for in-situ connection with pier). Post tensioning shall be accomplished partially in casting yard and partly in-situ depending on design requirements.

2.4.4 Slenderness effect in Piers shall be duly considered as stipulated in IRS-CBC or alternatively P-δ analysis shall be carried out..

2.4.5 To prevent the direct collision of vehicle to pier, a Jersey shaped crash barrier of 1.0m height above existing road level may be provided all around the pier with a gap of 25mm between the crash barrier and outer face of pier.

2.4.6 Size of the pier may be required to be increased to control the stresses / deformations based on case to case basis. As far as possible, pier shall be elongated along longitudinal direction. Only in special cases pier elongation along transversal direction shall be adopted depending on structural requirement.

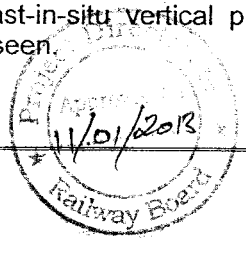
2.4.7 The space between the elastomeric bearings shall be utilized for placing the lifting jack required for the replacement of elastomeric bearing. An outward slope of 1:200 shall be provided at pier top for the drainage due to spilling of rainwater, if any.

2.4.8 Where ever plan alignment of the elevated guideway is not matching with central median, cantilever pier shaped (reinforced / prestressed) pier or portal beam with piers resting on central median / footpath shall be provided. Such portal shaped beams shall generally be monolithic with piers at its both ends.

2.5 Proposed Foundation System

2.5.1 Major stretch of all three corridors comprises of Hard/soft disintegrated rocky strata with overlying soil strata of 2m to 5m. Open foundations or pile foundations (with toe socketed in rock) shall be adopted depending on depth of rock from ground level. The same shall be finalized on a case-by-case basis.

2.5.2 Some part of alignment, where the stretch comprises of soil, guideway shall be supported on suitable diameter bored cast-in-situ vertical piles (viaduct). For typical piers, a pile group of four (4) is foreseen.



2.5.3 Initial pile load tests at required locations shall be conducted. The intended methodology and equipment for pile installation shall be employed while carrying out the initial pile load tests.

2.6 Parapets

2.6.1 Aesthetic appealing parapets shall be provided with provision of sound barrier on both side of elevated guideway. Since frontal evacuation of walkway is planned in case of emergency, parapet need not to have any provision of walkway at top. However parapet shall have provision that in case additional sound absorber materials are required at specific location to limit noise within allowable limit, the same can be mounted at its top. Since OHE mast system of traction has been planned the same shall be mounted on precast concrete parapet (connected to deck by cast-in-situ connection) of small length to support the same. Structure provided for parapet shall also be able to carry the cables along the guideway.

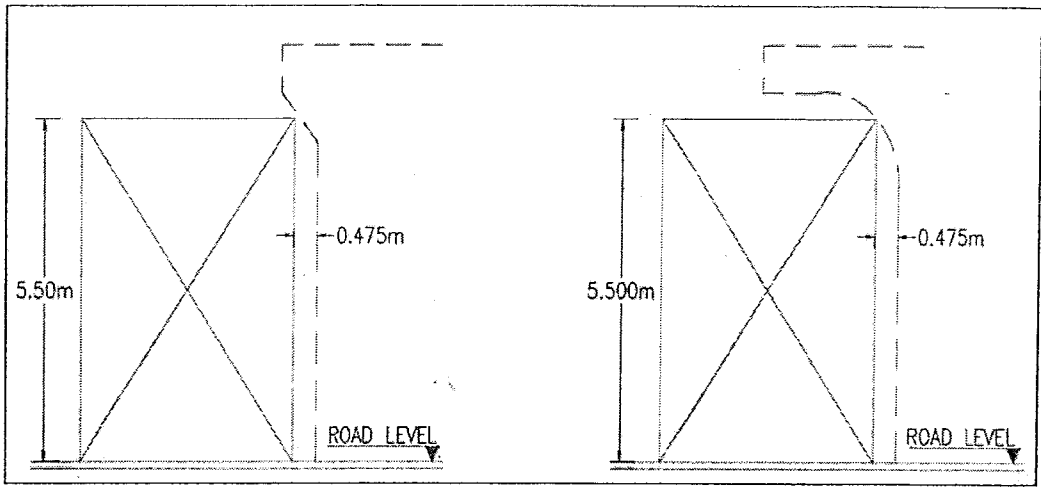
2.6.2 Alternatively, a precast concrete parapet (connected to deck by in-situ connection) shall be provided on both sides with local thickening of the same at OHE mast location.

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3 CLEARANCES

3.1.1 Clearance for Road Traffic

3.1.2 The shape of pier cap / hammer head piercap shall be so dimensioned that a required clearance of 5.5m is always available on roadside beyond vertical plane drawn on outer face of crash barrier i.e. 0.475m (0.45m (width of the 1m-high Jersey-type crash barrier) + 0.025m (clearance between crash barrier and pier shaft)) from pier shaft outer line.



3.2 Clearances for Rolling Stock

Vertical Clearance

3.2.1 The minimum plinth thickness is assumed as 200mm (without considering the extra depth of plinth required to accommodate the vertical curve).

3.2.2 The distance between top of rail and top of plinth is assumed as 220mm.

Horizontal Clearance

3.2.3 For horizontal clearance to any structure or horizontal clearance to any OHE mast, Schedule of Dimension shall be referred.

3.2.4 Deleted.

3.2.5 For Car Specifications S.O.D shall be referred

3.3 Clearances for Railway traffic

3.3.1 When the Metro viaduct crosses the Indian Railway track, minimum horizontal and vertical clearances as per IRS Schedule of Dimensions shall be followed.



4 STRUCTURAL MATERIALS

4.1 Units

4.1.1 The main units used for design shall be: [t], [m], [mm], [kN], [kN/m2], [MPa], [°C], [rad]

4.2 Concrete

Instantaneous modulus

4.2.1 E is given as § 5.2.2.1 of IRS- CBC-1999:

- For fck = 60 MPa E = 36,000 MPa (given in IRS-CBC)
- For fck = 50 MPa E = 34,000 MPa (given in IRS-CBC)
- For fck = 45 MPa E = 32,500 MPa (interpolated)
- For fck = 35 MPa E = 29,500 MPa (interpolated)

(where fck represents 28 days characteristic cube strength of concrete)

Conversion between cubic strength and cylinder strength

4.2.2 For purpose of calculations conversion between cubic strength and cylinder strength can be performed using internationally recognized Design Codes, for instance Eurocode EN 1992-1-1:2004, Table 3.1 as reproduced below:-

f_{ck}(cylinder) (Mpa)	12	16	20	25	30	35	40	45	50	55
f_{ck} (cube) (Mpa)	15	20	25	30	37	45	50	55	60	67

Modular Ratio

4.2.3 Modular ratio for all concrete grades shall be taken as

- For tensile R/f = 280/fck and
- For Compression R/f = 420/fck

(as per cl.5.2.6 in corrigendum slip no. 12 of IRS-CBC)

Compressive Strength

4.2.4 Following durability provisions for structures as per "moderate" conditions of environment in accordance with IRS CBC: 1997, clause 5.4 (also refer Correction Slip No-12 dated 03.06.2009) and as per "Durability of Concrete Structure", ref. Report N°. BS-14 (Revised), published by the Government of India (Ministry of Railways) in January 2001.

4.2.5 Keeping the durability and structural requirement, the minimum strength of various elements of structure shall be as follows:

- Superstructure f_{ck} = 45 MPa

(Segmental Box(Internal Prestressing or Segmentalor I- Beams with cast-in-situ slab)

- for pier (shaft & pier cap) fck = 40 MPa
- for cantilever pier & portals fck = 45 MPa
- for bearing pad (mortar) fck = 75 MPa
- for foundations (Open or, piles & pile cap) fck = 35 MPa
- Other Miscellaneous structure (Cable trough, parapet) fck = 30 MPa

4.2.6 Concrete characteristics as detailed above might need to be improved for foundation if the structure environment is found to be particularly aggressive (soil or water). This shall be assessed on case-by-case basis.

Density

4.2.7 Following density of various types of concrete shall be adopted for calculation of self weight of concrete members

- 25 kN/m3 prestressed concrete
- 25 kN/m3 for reinforced concrete

Poisson's Ratio

4.2.8 Poisson's ratio for all concrete: 0.15

Thermal Expansion Coefficient

4.2.9 Thermal expansion coefficient shall be taken as

$$\varepsilon = 1.17 \times 10^{-5} /°C$$

4.3 Prestressing Hardwares

Prestressing Steel for Tendons

4.3.1 Prestressing steel will be conforming to IS: 14268, class 2 Low Relaxation uncoated stress relieved strands.

Young's Modulus

4.3.2 E= 195,000 MPa (same value for 1 strand alone or 1 tendon).

Prestressing steel type

4.3.3 All Prestressing steel units shall be of 0.6" strands type (Nominal dia =15.2mm, Area=140 mm2).

Prestressing Units

4.3.4 12K15, 19K15, (longitudinal units)

Ultimate Strength & 0.2% Proof Stress & 0.1%Proof Stress

- Ultimate strength of strand = 1860 MPa
- 0.2% Proof stress =90% of Ultimate Strength = 1674 MPa
- 0.1% proof stress =85% of minimum ultimate tensile strength)= 1581MPa

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Density:

4.3.5 Density = 78.5 kN/m3

Relaxation Properties

4.3.6 For Long term, relaxation losses in prestressing steel, 3 times the 1000 hours value given in IRC:18-2000 shall be used.

Sheathing

4.3.7 Sheathing type:

- For internal cables, Double Corrugated HDPE Duct shall be used.

4.3.8 Diameter of sheathing:

- For internal cables-107 mm & 85mm ID (E 1 mm tolerance) for 19k15 & 12k15 shall be used-To be confirmed from supplier. Centripetal displacement of strands within internal dia of sheathing will be taken into account while designing the prestressed structure.

4.3.9 Thickness of wall for Sheathing:

- For Internal Prestressing: 2.3 + 0.3mm as manufactured and minimum 1.5mm after loss in the compression test, for duct size upto 160mm OD

4.3.10 Following coefficient shall be taken for short term loss calculations of prestress:

Friction (wobble)	0.002 m^{-1}
Friction (curvature)	0.17 rad^{-1}

Anchorages

4.3.11 For Internal prestressing: Anchorages conforming to BS:4447 shall be used

4.3.12 Anchorage set-in of 6 mm shall be considered at stressing ends

4.4 Structural Steel (for Composite Bridges & other structures if any)

4.4.1 Structural steel shall be used for special composite bridges and for miscellaneous use such as railing, supporting utilities, coverings etc.

Structural Steel for Composite Bridges

4.4.2 The connections between steel members shall be bolted using HSFG bolts and required torque shall be also applied. Bolts shall be tightened as per RDSO Guidelines or IS 4000. The bolts in serviceable limit state shall be designed as friction grip with appropriate slip factor and factor of Safety. The bolts shall be checked in ultimate limit state as bearing bolts.

Submerged Arc Welding (SAW) shall be used for shop welds as far as possible. Mandatory tests applicable to welds shall be conducted.

Structural steel conforming to Grade Fe 410W as per IS: 2062 shall be adopted and the grade shall be B0.

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4.4.3 Structural steel conforming to Grade Fe 540 as per IS: 2062 shall be adopted in case high strength steel is required and the grade shall be B0.

Tensile Strength And Yield Strength

4.4.4 Structural Steel shall conform to IS:2062:2006 with following properties.

a) For Fe 410W grade steel:

Tensile strength shall be	410 Mpa;
Yield strength shall be	250Mpa (for t<20mm),
	240Mpa (for 20mm < t < 40mm)
	230Mpa (for t > 40mm)

b) For Fe 540 grade steel:

Tensile strength shall be	540 Mpa;
Yield strength shall be	410Mpa (for t<20mm),
	390Mpa (for 20mm < t < 40mm)
	380Mpa (for t > 40mm)

Young's Modulus

4.4.5 E = 205,000 MPa

Steel Grade

4.4.6 Steel grade Fe 410 or Fe 540 conforming to IS 2062:2006 shall be followed. All chemical and mechanical properties of the steel shall also conform to IS 2062:2006.

Density

4.4.7 Density = 78.5 kN/m3

Poisson's Ratio

4.4.8 Poisson's Ratio = 0.30

Thermal Expansion Coefficient

4.4.9 $\varepsilon = 1.2 \times 10\text{-}5\ /°C$

4.5 Structural Steel for Miscellaneous Use

4.5.1 Two types of structural steel are proposed as per the following standards:

a) IS: 4923 "Hollow steel sections for structural use with Yst 310"

b) IS: 2062 "Steel for General Structural Purposes (Grade B-Designation 410B)"

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4.5.2 The hollow steel sections would be square (SHS) or rectangular (RHS). Other traditional rolled sections like plates, angles, channels, joists would also be used where necessary.

4.5.3 The base connections and connection with concrete shall be effected by internally threaded bolt sleeves (hot dipped galvanized @ 300 gm/ sqm) manufactured from

4.5.4 IS: 2062 Grade B mild steel. The sleeve shall receive hexagon-head bolt M20 Class 8.8 as per IS: 1364 (Part 1) with galvanized spring washer.

4.5.5 The connections within the steel structure would be effected essentially by direct welding of members with/ without gusset plates. The minimum thickness of metal for SHS/RHS sections for main chord members as well bracings shall be 4mm as applicable for steel tubes in clause 6.3 of IS: 806.

Young's Modulus

4.5.6 E= 200,000 MPa

Tensile Strength / Yield Strength :

4.5.7 For Hollow steel sections (conforming to IS: 4923)

Tensile strength shall be 450 MPa;

Yield strength shall be 310Mpa.

Density

4.5.8 Density = 78.5 kN/m3

Poisson's Ratio

4.5.9 Poisson's Ratio = 0.30

Thermal Expansion Coefficient

4.5.10 ε = 1.2x10-5 /°C

4.6 Reinforcement Steel (Rebars)

4.6.1 Only Thermo-mechanically treated reinforcement bars of grade 500 conforming to IS: 1786 will be adopted.

Young's Modulus

4.6.2 E= 200,000 Mpa

Yield Stress

4.6.3 fy = 500 MPa.

Diameters

4.6.4 Shall be in [mm] of the following size : 6, 8, 10, 12, 16, 20, 25, 28, 32, & 36.

Density

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4.6.5 Density = 78.5 kN/m^3



5 LOADS TO BE CONSIDERED

5.1.1 Following elementary loads shall be taken into account for design of structural component of viaduct:

Elementary load			
Dead loads	DL	Self Weight maxi	D_{max}
		Self Weight mini	D_{min}
		Overhead Line Equipment	OLE
	Shrinkage & Creep		SC
	Prestress		PR
Super Imposed Loads			SIDL
Live load	LL	Train Weight	TW
		Dynamic Impact	I
		Force due to curvature or Transverse eccentricity	CF
		Longitudinal Force (traction, braking)	LF
	Live Load on Foot Path		LFP
Derailment Load			DER
Overall temperature effect			OT
Differential Temperature			DT
Long welded rail forces			LWR
Nosing forces			NF
Forces on parapets			PP
Wind pressure effect :	WL	Longitudinal Direction	WL_x
		Transverse Direction	WL_z
Earthquake	EQ	Longitudinal direction	EQ_X
		Transverse direction	EQ_z
		Vertical direction	EQ_Y
Differential settlement (Applicable for continuous units only)			DS
Vehicle Collision load on Piers			VCL
Buffer Load			BL

5.2 Dead Loads (DL)

5.2.1 Self weight of all structural element shall be worked out based on the actual cross-section area and unit weight as defined in the previous section of this report.

5.3 Shrinkage & Creep (SC)

5.3.1 For calculation of shrinkage & creep, IRS:CBC shall be used but other codes could also be used depending on humidity ratio and notional thickness of the member. In such case, shrinkage calculation shall be done assuming average humidity ratio of 70%.

5.4 Prestressing Force (PR)

5.4.1 As per IRC:18:2010 - Jacking force in strand is limited to 90 percent of 0.1% Proof stress,(IRC 18-2000) = 0.9 * 0.85 of U.T.S = 76.5% Of U.T.S

5.4.2 Incidentally, as per IRS CBC: 1997 cl. 16.8.1, the maximum seating force in strand is limited to 70% of the characteristic strength of steel for post-tensioned tendons and to 75% of the characteristic strength of steel for pre-tensioned tendons. However the jacking force in tendon will be permitted up to 80% of characteristic strength of strand.

5.4.3 Following above-mentioned approach it is proposed to limit jacking force to 76.5% of ultimate tensile strength of strand for pre-tensioned steel / post-tensioned steel

Provision for replacement/future prestressing

5.4.4 For Internal Prestressing

- None of the prestress tendons shall be designed to be replaceable
- All the ducts shall be grouted with cementitious grout material
- For pre-cast post-tensioned segmental construction, each and every prestressing tendon duct is visible for inspection before pre-stressing. Moreover, grouting of cable duct shall be done only after confirmation of strand elongation as per design requirement. Hence, contingency tendons are not required for pre-cast segmental post-tensioned pre-stressed bridge.
- However, provision of installation of future cables in the form of external cables in all types of girders shall be provided as per IRS-CBC:1997 .
- In the case of post-tensioned full spans cast-in-situ structures, contingency tendons shall be considered as per the IRS Bridge Rules.

5.5 Superimposed Dead Loads (SIDL)

5.5.1 For calculation of Superimposed dead load, following assumptions shall be taken :

Variable type

- rails + pads 0.30 t/m
- cables 0.07 t/m

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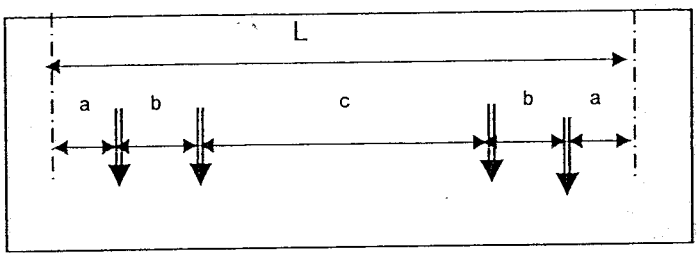
- cable trough cell 0.74 t/m
- cable trays 0.01 t/m
- plinth 2.80 t/m (suitable increase to be made for curved superstructure based on the cant value)
- miscellaneous (OCS, signalling...) 0.40 t/m

Fixed type

- parapet (on both side of deck) (2.2 t/m) taking account of running as well as locally thickened parapet at OHE mast also (To be verified as per detailed design)
- Total Suiperimposed Dead Load = 6.5t/m For tangent track (to be
- verified as per detailed design)

5.6 Vertical Train Live Load (TW)

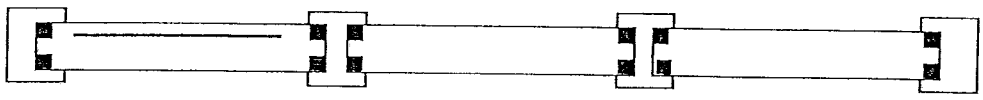
5.6.1 Each component of the structure shall be designed / checked for all possible combinations of these loads and forces. They shall resist the effect of the worst combination:



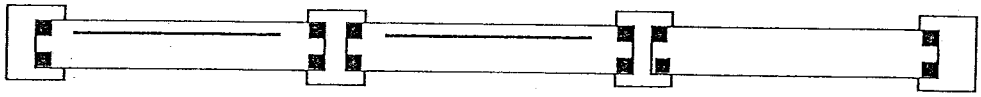
- Axle loads = 17 tons
- Maximum number of successive cars in a train = 6
- Wheel Configuration
  - i. a = 2150mm (Overhang)
  - ii. b = 2200mm (Wheel base in a bogie)
  - iii. c = 12400mm (Distance between Axle-2 and Axle-3 in the car)
- Total Length of one car L = 2a +2b +c = 21100mm (Length of a car)

5.6.2 Maximum number of axles shall be loaded on the superstructure to arrive at maximum longitudinal force, max shear and max BM. Substructure shall be checked for one track loaded condition as well as both tracks loaded condition. Where both the tracks are supported by single box superstructure, the bearings shall also be checked for one track loaded condition as well as both tracks loaded condition. Live load envelope cases shall be based on all possible configuration of train such as one car, two cars, three cars, four cars, five cars & six cars.

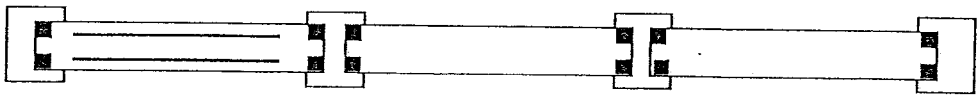
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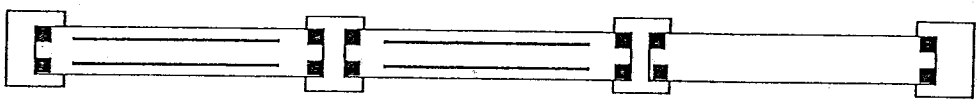
LL1: used for deck torsion, bearing compression, uplift, shaft check, foundation check



LL2: used for shaft check, foundation check



LL3: used for deck check, bearing compression check,uplift, shaft check, foundation check



LL4: used for shaft check, foundation check & shear key check

5.7 Coefficient of Dynamic Impact (I)

5.7.1 Impact factor for longitudinal analysis shall be as per IRS Bridge rules. The suitable corrections for speed and two tracks being supported on single section shall be applied. However a minimum value of longitudinal CDA shall be 1.2. Sample calculation is enclosed as Annexure – 1. For transverse analysis CDA shall be calculated as per IRS Bridge rules. The span (dispersion load length) shall be worked out as per Clause No. 2.3.4.2 of IRS Bridge rules. The calculation is attached as Annexure – 2.

5.7.2 Deleted

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5.8 Centrifugal Force (CF)

5.8.1 Centrifugal Force shall be as per IRS Bridge Rules.

5.8.2 Maximum operating speed of 90km/hr will be considered for computation of centrifugal force. In sharp radius, following speed shall be considered as per rolling stock characteristics and Schedule of Dimensions. Pl note that values

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given below are for radius of centerline of track and not the center line of alignment.

Plan Radius of Track (m)	Maximum Operating speed (km/hr)
Upto R= 400	90
R=300	80
R=250	70
R=200	60
R=175	55
R=150	50
R=125	45
R=120	45

5.9 Braking and Traction (LF)

5.9.1 Braking load shall be taken as 18% of the un-factored vertical loads.

5.9.2 Traction load shall be taken as 20% of the un-factored vertical loads.

5.9.3 Tractive force of one track and braking force of another track shall be taken in the same direction to produce worst condition of loading.

5.9.4 As per IRS Bridge Rules Cl 2.8.5, when considering seismic forces, only 50% of gross tractive effort / braking force shall be considered. However IRS Bridge Rules Cl 2.8.3.4 specifies that dispersion and distribution of longitudinal forces are not allowed for new bridges.

5.9.5 The provision of UIC-774-3 in relation to rail structure interaction is well known and is being used in many rail based structure. Rail structure studies shows that several piers of the guideway participate in resisting the braking and tractive force. Hence after distribution and dispersion of longitudinal forces through rail, bearing forces in a given span is less than braking/tractive forces transferred from wheels located in the same span. Hence our recommendation is to adopt the international practice of distributing the braking/ tractive forces by conducting a rail-structure interaction analysis..

5.10 Footpath Live Load (LFP)

5.10.1 Footpath live load shall be adopted as 490 kg/m2. As there is no walkway, footpath live load shall not to be considered with carriageway live load.

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5.11 Derailment Load (DER)

5.11.1 Check for derailment loads shall be made as per RL Loading of BD 37/01.

5.12 Overall Temperature (OT)

5.12.1 Following guidelines given in IRC:6:2010, cl 215.2, For Hyderabad city

- Highest Maximum Bridge Temperature = 45°C (as per Fig.8)
- Lowest Minimum Bridge Temperature = 5°C (as per Fig.9)
- Total Variation of Temperature = 45 - 5 = 40°C
- Mean Variation of temperature = 40 / 2 = 20°C

5.12.2 Bridge temperature to be assumed when the structure is effectively retrained (as per cl 218.2 of IRC-6:2000). For Concrete Structure:

= ± [Mean variation of temperature + 10°C]

= ± (20°C + 10°C)

= ± 30°C

5.12.3 Hence an overall differential temperature of ±30°C (taking account of difference between construction temperature and maximum/minimum coming temperature) shall be considered.

5.13 Differential Temperature (DT)

5.13.1 For closed sections differential temperature shall be considered as defined in IRC: 6-2000, Clause 215 with a modification that average thickness of 150mm wearing coat shall be assumed as the deck top is covered by track plinth, cable trough. To take account of such effect, temp gradient shall be modified as given in BD 37/01. Thermal stresses in superstructure shall be evaluated based on the half of the elastic instantaneous modulus of elasticity of concrete.

5.13.2 Above-mentioned check shall be also performed for beam slab superstructure system also.

5.13.3 Closed section shall be also checked for difference in temperature between outside and inside of box girder of 5°C.

5.14 Long Welded Rail Forces (LWR)

5.14.1 Continuous welded rails are proposed to be used for the deck. Therefore there will be an interaction between the rail and superstructure deck resulting in forces in the rail as well as forces in the deck. Rail structure interaction (RSI) studies analyzes the normal stress variation in the long welded rail generated by

- Differential expansion between the supporting structure and the rail
- Rotations at deck end due to track flexure
- Acceleration / braking horizontal loads

5.14.2 The UIC-774-3 is internationally recognized and used for Rail / Structure interaction studies. It details the calculation methodology, allowable stress and displacement limits to comply with, validation criteria for spreadsheets and FEM

Models. These recommendations have been used for years in most of railway projects where long welded rails have been used.

5.14.3 Please note that Rail structure interaction is not a single load case in itself. By the studies, the interaction between rail and deck under acceleration /braking, temperature variation of the deck/rail, end rotation of deck and rail fracture are taken into account. Therefore the effect of RSI is included in LL, LF, OT, DT load cases. In addition case of rail fracture shall be also needed to be considered to happen at any time during the service period of the structure.

5.14.4 Various parameters to be considered for such studies are

- UIC 60 Rail type to used
- Bilinear curve of Force / displacement will be considered (as per UIC-774-3)
- Toe load per rail seat (2.0t) —To be verified at detailed design stage based on the actual track fastening system adopted on the project.
- Slip resistance is 50% of toe load —To be verified at detailed design stage based on the actual track fastening system adopted on the project.
- Spacing of rail seat (0.65m)
- In the rail, the stress free temperature which is set to balance the stresses in the rail at high and low temperature is 33°C ± 5°C. At this temperature there is no additional stress in rail. Considering a rail temperature difference of 4°C to 62°C in Hyderabad, the maximum temperature variation in the rail shall be taken as ± 34°C (calculated below).

  When rail will be installed at low temperature: (62 – (33.0 – 5)) = 34.0°C

  When rail will be installed at higher temperature: (4 – (33.0 + 5)) = - 34.0°C

- In concrete superstructure deck–half of rail variation i.e +/-17^0 C shall be taken.
- Radial effect of long welded rails in curved structure shall be also considered while designing the substructure and foundation.
- Rail fracture condition (for one rail breaking at a given time) shall also need to be considered.
- Maximum allowable additional stress in rails due to RSI effects shall be as per UIC 774-3.

  Maximum compression: 92.0 MPa

  Maximum tension: -92.0 Mpa

The span configuration/ pier locations on the viaduct portion where turnouts are located shall be so planned such that the expansion joint between two girders (with one girder having moveable end) shall not fall either in the switch portion or crossing portion of the turnout. However the expansion joint can be in the lead portion of the turnout. In case it is completely unavoidable (expansion joint falling either on switch portion or crossing portion), both the girders adjacent to expansion joint shall have fixed bearings.

5.14.5 Stage opening of the corridors shall also be taken into consideration while designing the viaduct (at terminals) for the following:

a) Dispersion of the Longitudinal forces at the end of the viaduct at each stage of opening

b) Long welded rail forces at end spans of the viaduct at each stage of opening

c) The Buffer loads at end spans(turn back facility) of the viaduct at each stage of opening

5.15 Nosing Forces (NF)

5.15.1 Nosing Force shall be taken as single concentrated force of 100kN acting horizontally at the top of rails in either direction at right angles to the centre line of the track and at such a point in the span as to produce maximum effect in the element under consideration.

5.15.2 It shall be applied on both the straight and curved track. For element supporting more than one track a single load as specified shall be taken.

5.15.3 This force shall always be combined with the vertical train load. However It shall be neglected in combination with seismic loading.

5.16 Forces on Parapets (PP)

5.16.1 In addition to self wt, parapets shall be designed to resist a lateral horizontal and vertical force of 150kg/m applied simultaneously at the top of the railing or parapet. Additional noise barrier In addition to above aerodynamic actions from passing of train shall be taken into account based on the design speed of the train, aerodynamic shape of the train , the shape of parapet structure and the position of parapet structure.

5.17 Wind Force (WL)

5.17.1 Wind Loads (longitudinal & Transverse) as given in IRC:6-2010 shall be used. However the height of vehicle shall be as given in Schedule of Dimensions for Rolling Stock above the rail level. Cl 209.3.7 of IRC 6 shall not apply.

5.18 Seismic Force (EQ)

5.18.1 The purpose of this section is to summarize the methodology and the assumptions that shall be used for the seismic analysis.

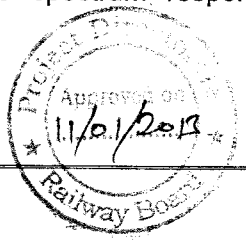
General Principle

5.18.2 Seismic analysis of viaducts shall be conducted according to the proposed modifications in Indian standard IRC 6:2010, clause 219.

5.18.3 Therefore, the seismic actions are calculated by a 2-steps process:

- Single mode analysis to obtain the fundamental vibration period of the viaduct
- Estimation of seismic forces using the spectrum response, defined hereafter.

Fundamental Vibration Period Calculation



5.18.4 The fundamental period calculation is performed according to the table C-3.3.1 of IRC 6:2010. Each pier is considered as a single degree of freedom oscillator with mass placed at the Centre of Gravity (COG) of the deck.

5.18.5 Expression given in Appendix A of the clause 22 of IRC 6:2010 can be also used for period computation. This is:

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

D = Appropriate dead load and live load in KN as defined is "Mass" section

F = Horizontal force in kN required to be applied at the centre of mass of the superstructure for one mm horizontal deflection at the top of the pier along the considered direction of horizontal force.

a) Masses shall be as defined in Annexure - D of IRC.6.2010.

5.18.6 25% of Train mass shall be considered while evaluating time period / forces due to seismic in transverse direction. This percentage is only for working out the magnitude of seismic force. Train mass shall not be considered when acting in the direction of traffic i.e. longitudinal direction. In both the seismic conditions (longitudinal as well as transverse), for calculating the stresses due to vertical effect of live load, 50% of the design live load shall be considered at the time of earthquake.

5.18.7 As per IRS Bridge Rules, correction slip no.22 dated 17 / 1 / 1994, in transverse/ longitudinal seismic condition, only 50% of gross tractive effort / braking force shall be considered.

a) Stiffness

- Stiffness shall be calculated with the uncracked section characteristics and with the concrete instantaneous modulus of elasticity, for all structural elements.
- Wherever pile foundations are provided, effect of the foundation system (pile-cap + piles + soil) in the flexibility of the substructure is considered by a set of equivalent springs added simulating pile-soil interaction (for details refer to pile stiffness calculation).

Response Spectrum Definition

5.18.8 All numerical values mentioned in this chapter are based on a 5% damping ratio, which will be used for the design.

a) Basic Design Response Spectrum

Response spectrum used for seismic calculation shall be as per (IS 1893 (part1) 2001), reproduced in IRC 6:2010.

b) Seismic Acceleration

c) Sa/g being computed by the relevant IRC:6 seismic acceleration can then be calculated by:

Ah = (Z/2) * (I/R) * (Sa/g), where:

Ah = horizontal seismic coefficient to be considered in design (Seismic acceleration = Ah*g)

Z = Zone factor = 0.10 (Hyderabad region = zone II)

I = Importance factor = 1.5

d) R = Response Reduction Factor shall be as per Table 8 of IRC.6.2010.

e) Above factor is based on the assumption that ductile detailing shall be followed for all structural component.

Vertical Seismic

- The vertical seism shall be taken as two third of the horizontal seismic coefficient (A_h).

$$A_v = 2/3 * A_h$$

- Since Hyderabad is in zone-II, vertical seismic to be considered only for prestressed superstructure. Vertical seismic is not to be considered for substructure & foundation.

Seismic Combinations

- As per IRC 6: 2010, the following seismic combinations shall be considered:

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

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

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

- r_1 : Seismic force calculated by Ah in X direction (x, axis of the project)
- r_2 : seismic force calculated by Ah in Z direction (Z , transverse direction)
- r_3: Vertical seismic calculated by Av

5.18.9 For design of foundation, the seismic loads shall be taken as 1.25 times the forces transmitted to it by substructure, so as to provide sufficient margin to cover the possible higher forces transmitted by substructure arising out of its overstrength.

5.18.10 For calculation of displacement of any element of bridge or bridge as a whole due to seismic, response reduction factor of 1.0 shall be taken.

5.19 Differential Settlement (DS)

5.19.1 Differential settlement between two adjacent viaduct piers shall be:

- 5mm (min.) if the foundation rest on highly weathered disintegrated rock (to be verified by Geotechnical Specialist)
- No differential settlement will be considered if the foundation rest on disintegrated hard rock or hard rock (to be verified by Geotechnical Specialist)
- Differential settlement is to be considered only in the design of continuous structures, if any.

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- While deriving the effect of differential settlement, long term modulus of concrete of superstructure (half the instantaneous modulus of concrete) shall be taken.

5.20 Vehicle Collision Loads on Piers (VCL)

5.20.1 Based on the proximity of piers supporting guideway structure from edge of road carriageway, any collision load to be decided case to case basis.

5.20.2 The effect of collisions loads shall not be considered on pier located 4.5m from the edge of carriageway.

5.20.3 Piers not conforming with above-mentioned provisions need to be designed for vehicle collision load as given below:

	Load normal to carriageway (t)	Load parallel to carriageway (t)	Point Of application on bridge support
Main Load	50	100	At the most severe point between 0.75m and 1.5m above carriageway level
Residual Load	25	50	At the most severe point between 1m and 3m above carriageway level

5.20.4 The nominal loads given in table above shall be considered to act horizontally as vehicle collision loads. Supports shall be capable of resisting the main and residual loads component acting simultaneously. Loads normal to the carriageway and loads parallel to the carriageway shall be considered to act separately and shall not be combined.

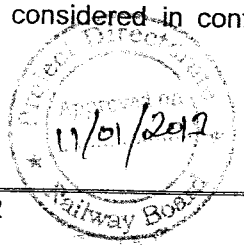
5.20.5 The piers shall be designed for the residual load component only if protected with suitably designed barrier/fencing system taking into account its flexibility, having a minimum height of 1.5m above the carriageway level. In such a case the crash barrier as mentioned in section 2.4.5 need not be provided.

5.20.6 While checking for vehicle collision load, the principal live load on the guideway, seismic or wind need not to be considered.

5.20.7 Vehicle collision load to be checked for 1.0 x VCL in SLS (stress check) only and 1.25 x VCL in ULS.

5.20.8 Construction loads (CL)

- For superstructure constructed span by span, distributed construction live load of (1.0kN/m^2) shall be taken in addition to any other equipment loads (such as temporary prestressing frames and bars) at the top of superstructure to be erected. Any reaction at top of erected structure from the LG girder during erection of span segments or self hauling of launching girder shall also be considered.
- For superstructure constructed by balance cantilever construction, following construction loads shall be considered in conformity with EN 1991-1-6.



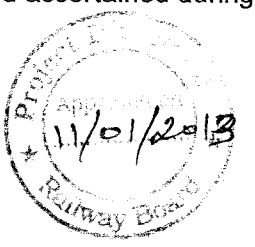
- Construction live load (qc) :- A distributed load (q_{ca}) of 1.0kN/m2 shall be taken as a construction live load (workers on top of erected superstructure). In addition to above a distributed load (q_{cb})of 0.2kN/m2 shall be also to be taken into account for the allowance of miscellaneous small sized machinery and light equipment. To take account of storage of any material or construction equipment at tip of cantilever a load (F_{cb})of 100kN shall be also taken into account.
- Form Traveller / Lifter Gantry (Qcc):- Actual load of form traveller / Lifter Gantry shall be taken into account . Such load shall be lowered or enhanced by a factors of 0.96 or 1.06 depending on whether the load effect is favourable or unfavourable.
- Movable heavy machinery and equipment Loads (Qcd):- Loads from any construction equipment moving over the part / fully erected structure such as movement of launching girder , beam and winches , trucks delivering the segments
- Accidental Release of Form Traveller/ Lifter Gantry (Ac) :- In case of accidental release of any empty form traveller during erection / casting of segment, dynamic response of the same shall be also taken in addition to removal of form traveller. Such dynamic response shall be 100% of form traveller weight. It means upward force equivalent to twice the form traveller weight shall be applied for the accidental release of form traveller.

5.21 Buffer Load (BL)

5.21.1 Provision of Buffers is contemplated at the end of temporary terminal stations during stage opening of the Corridors , at Pocket track ends and at the terminal stations of the corridors (at the end of turn back/stabling lines)Such buffers will be of friction type . These Buffers will be designed to have the following stopping performance:

- A fully loaded 6-car train (having a mass of 408 tonnes) shall stop from 15 km/h in a maximum distance of 15m (including the length of buffer stop and its components- i.e. total occupancy distance) without damage to the train or buffer
- A fully loaded 6-car train (having a mass of 408 tonnes) shall stop from 25 km/h in a maximum distance of 15m (including the length of buffer stop and its components- i.e. total occupancy distance) regardless of damage to the train or buffer.

5.21.2 Viaduct elements need to be designed for such Buffer load. The exact Buffer loads need to be interfaced and ascertained during the Detailed Design.



6 LOAD COMBINATIONS

6.1.1 Load Combinations for RCC substructure & foundation & prestressed structure with internal prestressing

6.1.2 In each of SLS and ULS cases, 5 basic load combination groups, as indicated in the table shall be considered. Due to the SHEAR-KEY, withstanding the horizontal forces, combination group IV can be eliminated if rail structure interaction study is made. However Group IV shall be considered in the case of POT – PTFE Bearings.

Limit State	Loads	Symbol	G I	G II (***)		G III	G V
			Cracking + Stress	G II a Stress	G II b Stress	Stress	Stress
SLS COMBINATIONS	Dead Loads	DL	1.00	1.00	1.00	1.00	1.00
	Shrinkage & Creep	SC	1.00	1.00	1.00	1.00	1.00
	Prestressing	PR	1.00	1.00	1.00	1.00	1.00
	Super Imposed Loads	SIDL (fixed)	1.00	1.00	1.00	1.00	1.00
		SIDL (Variable)	1.20	1.20	1.20	1.20	1.20
	Earth quake	EQ		1.00	1.00		
	Wind			1.00	1.00		
	Overall T	OT				1.00	
	Differential Temperature	DT				0.80	
	Differential settlement	DS	1.00				
	Live Load	LL	1.10		1.00	1.00	
	Derailment Loads	DR					1.00
ULS COMBINATIONS	Dead Loads	DL	1.25	1.25	1.25	Not to be considered, Per IRS: CBC Cl 10.3.1	1.25
	Prestressing	PR	1.15/0.87 (*)	1.15/0.87 (*)	1.15/0.87 (*)		1.15/0.87 (*)
	Super Imposed	SIDL (fixed)	1.25	1.25	1.25		1.25

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	Loads	SIDL (Variable)	2.0	2.0	2.0		2.0
	Earth quake	EQ		1.60	1.25		
	Wind			1.60	1.25		
	Live Load	LL	1.75		1.40		
	Derailment Load						As per BD-37

- (*) 1.15/0.87 : according to IRS CBC art. 11.3.3., when the Prestressing PR increases the section capacity vs. shear then PR is multiplied by 0.87. When the Prestressing PR decreases the section capacity vs. shear then PR is multiplied by 1.15.
- (***) Wind and Earthquake loads shall not be assumed to be acting simultaneously.

6.1.3 50% LL effects (LL + LFP) have to be considered along with G II & GIII.

6.1.4 Structure shall be checked with appropriate Prestressing value, i.e. at construction and at "infinity" stage (i.e. Year 100).

6.1.5 Following SLS & ULS Load combinations shall be followed for checks to be made during construction stage.

LOAD COMBINATIONS DURING CONSTRUCTION

Limit State	Actions		Symb	SLS-1	SLS-2				
SLS COMBINATIONS	Dead Loads		G	1.0	1.0				
	Shrinkage & Creep		SC	1.0	1.0				
	Prestressing		PR	1.0	1.0				
	Construction Loads	Construction Live Load	qca	1.0	1.0				
		Light Equipments	qcb & Fcb	1.0	1.0				
		Form traveller / Lifting gantry / Launching Girder	Qcc & Qcd	1.0	1.0				
	Earthquake		EQ		0.5				
	Overall T		OT	1.0					

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	Wind		W	1.0					
ULS COMBINATIONS	**Actions**		**Symb**	**ULS-1**	**ULS-2**	**ULS-3**	**ULS-4**	**ULS-5**	**ULS-6**
	Dead Loads		G	1.35/1.25	1.05/0.95	1.35/1.25	1.05/0.95	1.0	1.0
	Prestressing		PR	1.0	1.0	1.0	1.0	1.0	1.0
	Construction Loads	Construction Live Load	qca	1.35	1.35	1.35*0.2	1.35*0.2		1.0
		Light Equipments	qcb & Fcb	1.35	1.35	1.35	1.35		1.0
		Form traveller / Lifting gantry /Launching Girder	Qcc & Qcd	1.35	1.35	1.35	1.35	1.0	1.0
		Accidental	Fa	---	---	---	---	1.0	---
	Earthquake		EQ		---		---	---	1.0
	Wind		WL	1.35 * 0.2	1.5*0.2	1.35	1.5	---	---

- Special Notes for special structures constructed by balance cantilever construction.

  a) In addition to check for strength of structure for Load Combination ULS-1 to ULS-6, check for loss of equilibrium shall be also performed and it shall be ensured that no uplift in bearing (temporary or permanent) shall occur. Such condition will require additional stabilizing arrangement and vertical prestressing to prevent any uplift in the bearing.

  b) While performing ULS combination ULS-1 to ULS-4, unbalance load due to different numbers of segment on either side of pier (n segments on LHS side and n+1segments on RHS side) shall be considered. In addition to above, Live load (vertical downward on RHS side) and Wind load (vertical upward on LHS side & Vertical downward on RHS side) shall be also applied. Form Traveller / Lifting Gantry load shall be also applied at tip of LHS side and RHS side).

  c) Different Value of partial load factor to dead load (1.35 /1.25 or 1.05 /0.95) shall be applied on RHS (higher value for destabilizing effect) of structure & LHS (lower value for stabilizing effect) of structure.



7 DESIGN METHODOLOGY

7.1 Design Code

7.1.1 The Indian Railway's Concrete Bridge Code shall be followed for design for RCC sub-structure and foundations and for prestressed concrete girders with internal prestressing. For superstructure with external prestressing with dry joints, international codes such as BD 58/84, AASHTO 1996, shall be followed.

7.2 Permissible Stresses in SLS Case

7.2.1 Allowable stresses in superstructures for SLS check for different types of structures are given below:-

For Precast Segmental Simple Spans with Internal Prestressing

s. No	Load Combination	Allowable compressive strength	Reference	Allowable Tensile stress	Reference
At transfer and/or Construction stage					
1	DL + *DS + App. PR	0.5 fci but <0.4 fck	Cl 16.4.2.2(b) of IRS CBC 1997	No tension anywhere	
2	SLS Combination as per cl 6.1.4	0.5 fci but <0.4 fck	Cl 16.4.2.2(b) of IRS CBC 1997)	No tension anywhere	
During Service					
3	SLS GI	0.4 fck	Cl. 16.4.2.2 (a) of IRS CBC 1997	No tension anywhere	Note 2 under Table 11, IRS CBC 1997, and cl 17.3.3)
4	SLS GII	0.4 fck	Cl. 16.4.2.2 (a) of IRS CBC 1997	No tension anywhere	Note 2 under Table 11, IRS CBC 1997, and cl 17.3.3)
5	SLS GIII	0.4 fck	Cl. 16.4.2.2 (a) of IRS CBC 1997	No tension anywhere	Note 2 under Table 11, IRS CBC 1997, and cl 17.3.3)



For Cast-in Situ / Precast Post-Tensioned Hammer Head Pier Cap (For supporting I-beam) / Precast Pretensioned / Postensioned I-girder with cast-situ slab

No	Load Combination	Allowable compressive strength	Reference	Allowable Tensile stress	Reference
At transfer and/or Construction stage					
1	DL + *DS + App. PR	0.5 fci but <0.4 fck	Cl 16.4.2.2(b) of IRS CBC 1997	-1.0 mpa	Cl 16.4.2.2(b) of IRS CBC 1997
2	SLS Combination as per cl 6.1.4	0.5 fci but <0.4 fck	(Cl 16.4.2.2(b) of IRS CBC 1997)	-1.0 mpa	(Cl 16.4.2.2(b) of IRS CBC 1997)
During Service					
3	SLS GI	0.4 fck	(Cl. 16.4.2.2 (a) of IRS CBC 1997	No tension anywhere	Note 2 under Table 11, IRS CBC 1997, and cl 17.3.3)
4	SLS GII	0.4 fck	(Cl. 16.4.2.2 (a) of IRS CBC 1997	No tension anywhere	Note 2 under Table 11, IRS CBC 1997, and cl 17.3.3)
5	SLS GIII	0.4 fck	(Cl. 16.4.2.2 (a) of IRS CBC 1997	No tension anywhere	Note 2 under Table 11, IRS CBC 1997, and cl 17.3.3)

7.3 Check for ULS

7.3.1 Requirements for Ultimate Limit State check for Superstructure with internal prestressing as given below:-

Ultimate Limit State for Superstructure with internal prestressing

7.3.2 Ultimate Limit State check for flexure as required in IRS Concrete Bridge Code, 1997, cl. 16.4.3. shall be made. Appropriate formulae or software may be used. Shear and torsion shall be checked in accordance with IRS CBC 1997, cl 16.4.4 while calculating maximum shear stress as per clause 6.4.4.5. "d" is the distance from the compression face to the centroid of the actual steel area in tension zone. However "d" should not be less than 0.8 times the overall depth of the member.

7.3.3 The ultimate flexural & shear capacity calculated for internally prestressed segmental structure shall be multiplied by a factor of 0.95 & 0.9 respectively.

7.4 Crack Width

7.4.1 Crack width in reinforced concrete members shall be checked for SLS combination G I. Crack width shall be as per § 15.9.8.2 of IRS CBC. Crack width shall not exceed the admissible value based on the exposure conditions defined in section 2.2.1.2.

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7.4.2 For crack control in columns, cl.15.6.7 shall be modified to the extent that actual axial load shall be considered to act simultaneously.

7.5 Fatigue Check

General

7.5.1 Fatigue phenomenon needs to be analyzed only for those structural elements that are subjected to repetition of significant stress variation (under traffic load). Thus generally the fatigue needs to be regarded only for deck structural part.

PC Structures

7.5.2 Fatigue check for prestressed concrete (PC) structures does not need to be performed as long as the whole section (from top to bottom fiber) remains under compression under SLS GI (OK by definition) both in the longitudinal and transverse directions.

RC Structures

7.5.3 Fatigue check for reinforced concrete (RC) structures does not need to be performed unless a RC main structure member (i.e. the deck) supports the traffic.

Steel Structures

7.5.4 Fatigue check is needed for steel or steel/concrete composite structures if any shall be as per Correction slip 18 of IRS Steel Bridge Code.

7.5.5 Requirements of the IRS-CBC, § 13.4 shall be followed for reinforcement bar welding.

7.5.6 Lap welding & welding in part of deck slab subjected to concentrated loads shall not be allowed.

7.5.7 Serviceability Requirements And Criteria For The Calculations Of Deformations.

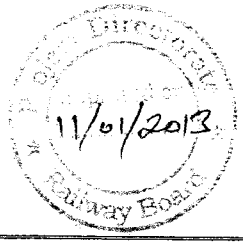
Vertical Deflection At Mid Span

7.5.8 As per IRS: CBC Cl 11.3.4, vertical deck deflection is to be checked only for GI load combination for SLS case. This total vertical deck deflection shall be sum of vertical deflection due to longitudinal flexure of the girder and transverse flexure of slab of girder and flexure of any supporting arrangement such as portal girder.

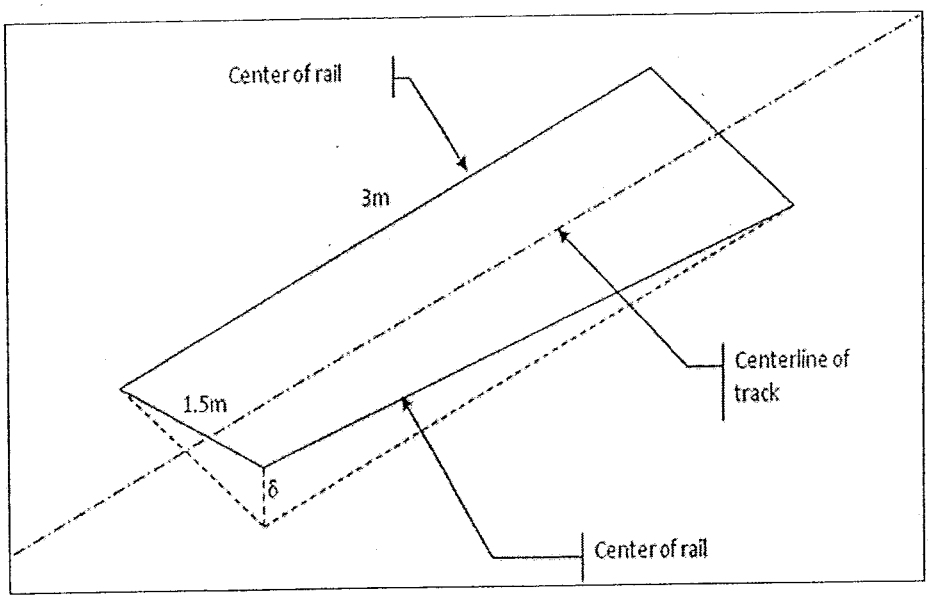
7.5.9 Vertical deck deflection at mid span shall be limited to

7.5.10 L/600 for deck supporting double track (LL including impact) based on robustness requirement of the superstructure.

7.5.11 From passenger comfort point of view the total deflecion shall be limited as defined in Figure A2.3 of Eurocode EN1990 for railway bridges with 3 or more successive simply supported spans corresponding to a permissible acceleration of bv =1.0m/s^2 (very good level of comfort) for a speed of 120km/hr for various span configuration.



Track Twist



7.5.12 Twist (δ mm/3m) in a track shall not exceed 4.5mm (δ) due to live load.

7.5.13 Twist (δ mm/3m) in a track shall not exceed 7.5mm (δ) taking into account the total twist due to permanent deformation and deformation due to live load.

7.5.14 Such deformation check is required to be performed for all SLS load combinations.

Deformations At Deck End

7.5.15 Following deformations at deck end is limited to prevent excessive stresses in continuous rail. These checks need to be performed for SLS load combination GI only.

a) Vertical

- Relative vertical displacement at deck end w.r.t end of adjacent deck shall not exceed 3mm.

b) Transversal

- Relative transversal displacement at deck end w.r.t end of adjacent deck shall not exceed 5mm. Max horizontal deformation of decks shall be limited as per UIC-776-3R for speed range of 120 km/hr.

c) Longitudinal

- The gap between ends of two adjacent deck shall be kept so that pounding of the structure does not happen in seismic condition (after taking account of creep, shrinkage & temperature movement).

d) Rotation

- The maximum change in rotation at deck end of two adjacent superstructures shall be limited to 0.002radian.

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7.6 Durability

7.6.1 All components of viaduct structures shall be designed for a design life of 100 years. Following below-mentioned parameters as given in IRS:CBC , 100years design life can be ensured (refer cl 16.1.3 of IRS:CBC) in non-coastal /non-sea areas.

7.6.2 Following specifications are intended to meet the durability requirements:

- Complete and adequate drainage;
- Sufficient concrete cover;
- Limiting crack width
- Appropriate concrete mixture design and good pouring, acceptable permeability and surface finishing (IRS-CBC § 5.4)

7.6.3 Considering moderate condition of exposure for exposed structure i.e superstructure, pier & foundation, following values of concrete cover to outermost reinforcement shall be used as per IRS:CBC.

a) For Superstructure

- Precast segmental Superstructure 35mm
- Cast-in-situ superstructure 40mm
- For Pier/ Pier Cap 50mm

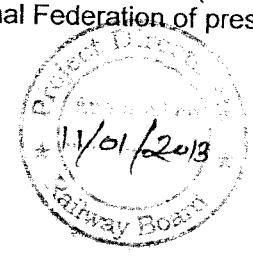
b) Reinf Cover to pile, pile cap & open foundations has to reviewed based on aggressiveness of soil on case to case basis

- For Open foundation 50mm
- For Pile cap 50mm
- For Pile 50mm
- Concrete cover to prestressing duct 75mm

W/C ratio (for RCC & Prestressed Structure) restricted to 0.4.

7.6.4 For prestressed concrete superstructure, Cement conforming to IRS T-40 or Grade-53 cement (IS:12269) shall be used.

7.6.5 For internal bonded prestressed segmental superstructure, the mating surface of both adjoining segments shall be applied with epoxy (of about 1mm on each surface) and then subject to axial prestress equivalent to 0.3mpa (uniformly over the complete cross-section) within 70% of open time. Epoxy to be used shall meet requirements of relevant provision of FIB (International Federation Of concrete, previously "FIP-International Federation of prestressed Concrete").



8 ISSUES RELATING TO FOUNDATION DESIGN

8.1 General

8.1.1 Where ever it is practical to excavate, shallow open foundation system founded on soil / weak rock/ moderately weak rock /hard rock shall be used or else pile foundation shall be used with toe socketed in weak rock / moderately weak rock / hard rock.

8.1.2 For determining the width of open foundations, it shall be ensured that an unobstructed barricading width of 9m shall be provided and one should be able to excavate to final founding level with unsupported vertical edges. If necessary, additional shoring arrangement to vertical edges shall be required to be envisaged as a safety measure.

8.2 Design Assumptions for Open Foundation

8.2.1 Bearing pressure underneath open foundation shall be worked out based on Serviceability Limit State (SLS) load combination GI, GII & GIII and it shall be ensured that maximum bearing pressure shall be less than allowable bearing pressure. Coefficient of Dynamic Augment (CDA) on live load shall not be taken into account for comparing the maximum bearing pressure vis-à-vis allowable bearing capacity. It is also not to be taken into account for structural design of open foundation.

Shallow Open Foundation on Soil / Weak rock / Moderately weak rock

8.2.2 Open foundation on soil is permitted provided required bearing pressure & settlement criteria shall be met with. Settlement / rotation under LL shall be checked and deformation limits of track as defined elsewhere in this report shall be met with. Plate load tests shall be performed for typical cases to conform the settlement. The sizes of open foundations shall be so proportioned that resultant of all forces on the base of the foundation shall fall within the middle third. It means that all parts of the foundation shall remain under compression under all SLS load combinations.

Shallow Open Foundation on Hard Rock

8.2.3 The dimensions of the open foundations shall be so that the resultant of all forces on the base of foundation shall fall within the middle half. It means that 75% of the area of the foundation shall remain in compression under all SLS load combinations.

Structural Design of Open Foundation

8.2.4 Open foundation shall be checked as RCC member in SLS only with stress check for all load combinations & crack width check for G1 load combination only Methodology as given in IRC: 78-2000 clause 707 "Open Foundation" shall be followed for structural design of Open foundation. However verification of equilibrium is required to be performed in SLS combination as per IRS:BRIDGE SUB-STRUCTURE & FOUNDATION Code.

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Case	Combination No	Factor of Safety Against Overturning	Factor Of safety Against Sliding
SLS combination as per cl 6.1.1	Combination G1 & GIII	2.0	1.5
	Combination GII	1.5	1.25
SLS Combination as per cl 6.1.4	SLS-1 & SLS-2	1.5	1.25

Allowable Bearing Pressure

8.2.5 Allowable Safe Bearing capacity for open foundation shall be assessed by the geotechnical specialist following the recommendations given in Section- 6 of IRS-Code of Practice for the Design of Substructure and Foundation of Bridges.

8.2.6 For SLS load combination GII, allowable bearing pressure shall be enhanced by 33%.

8.3 Design Assumptions for Pile Foundation

8.3.1 The pile cap/piles system supported by horizontal and vertical soil spring reactions is idealized as a space frame. The forces applied by the pier are transferred to the bottom of the pile cap for this purpose.

8.3.2 For piles and pile caps, the load combinations GI GII and GIII shall be considered, which is a part of the IRS CBC 1997, Table 12: Combinations I, II& III. CDA on live load shall not be taken into account while comparing the actual pile load vis-à-vis pile capacity. It is also not required to be considered for structural design of pile.

8.3.3 The various specific assumptions made for the pile and pile cap design are as follows:

a) Large diameter bored cast-in-situ vertical group of piles / monopiles have been contemplated for the foundation of piers.

b) The vertical load capacity of the pile in soil shall be based on static formula given in IS: 2911 (Part-1/Section-2). For pile carrying capacity, the SLS check (Comb I, II & III) only will be considered and no reference will be made to ULS combinations. The lateral load capacity of pile in soil shall be evaluated by using empirical formulae given in IS:2911 (Part-1/Section-2) by limiting the lateral deflection to 1% the pile diameter of the pile considering it as fixed headed pile under combination I. The capacity so evaluated will be used purely for the purpose of arriving at the upper bound of lateral load capacity. This deflection limitation will not be applicable in load combinations with seismic conditions for which the resulting stresses and capacity of the section would be the governing criterion. The permissible increase in vertical load capacity of pile in comb II would be taken as 33%.

c) The following limiting values shall not be exceeded for computation of safe vertical load of pile in soil:

i. Results of sub-surface investigation will be used for adopting the value of angle of internal friction ∅ and cohesion of soil, c.

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ii. Angle of wall friction δ shall be taken as equal to ∅ deg.

iii. Co-efficient of earth pressure "K" shall be taken as 1.5 generally for the type of soil likely to be encountered.

iv. Maximum overburden pressure at bottom of pile tip for calculation of shaft resistance and end bearing resistance shall correspond to pile length equal to 15 times the diameter of the pile. The maximum depth shall be considered from the existing ground level.

v. The entire overburden shall be assumed fully submerged (wherever applicable) for the purpose of calculation of safe load.

vi. Factor of safety to arrive at working load = 2.5 (for pile in soil).

vii. Bulk density corresponding to 100% saturation (wherever it is applicable) shall be calculated and used for working out submerged density of soil.

d) Initial load tests (not on working pile) shall be conducted first for determination of safe vertical and lateral loads. The safe load shall be taken as least of (a) load arrived at from the initial load test and (b) the calculated safe load based on static formula. Initial test shall be conducted for a load of 2.5 times the safe vertical load based on static formula.

e) Soil stiffness for lateral loads shall be taken from IS: 2911 (Part I/ Section 2 Appendix C). Unconfined compressive strength shall be calculated base on the results (from in-situ samples collected) from Geotechnical Investigation Report. Cohesion as calculated using unconsolidated undrained test with required modification of angle of internal friction will be used for working out unconfined compression strength.

f) For working out vertical load capacity of pile socketted in rock, IRC-78 –Amendment No-68 shall be followed. Various factors to be adopted for such purpose shall be assessed by Geotechnical Specialist. However, for calculation of socket friction capacity, 0.3 m of top zone of socket shall be ignored as per IRC: 78 (Notification no-68, Appendix-5, cl. 9.1, Note 3). For working out horizontal / moment carrying capacity of socketed pile, IRC: 78 (Notification no-68, Appendix-5, cl. 9.2) shall be followed. In case of pile passing through combination of soil and embedded in weak rock /moderately weak rock / hard rock , the method adopted by Reese and Matlock based on P-Y curve model shall be used following cautionary measures in terms of installation of pile and testing of pile shall be also taken as given below:-

i. Initial load tests shall be performed on test piles (using the same methodology and type of equipment to be employed for permanent pile) for working out vertical load carrying capacity of pile (well in advance of working pile of permanent structure) shall be conducted to verify the theoretical calculated values. The number of initial load tests shall be determined taking into consideration of the borelog.

ii. Routine load test shall be conducted again to confirm the allowable load.

g) Design criteria of monopile as a foundation for viaduct pier are not covered in IRS, IRC or IS codes.

- Hence, for design of monopile (if adopted), guidelines given in AASHTO-LRFD for fulfil the requirements of Geotechnical limit

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state, Structural strength limit state and serviceability limit state shall be followed.

- For performing the lateral load capacity of monopile in layered soil in combination of completely weathered rock / hard rock, a non-linear approach shall be followed taking account of

i. Modelling of pile shall be performed using beam on elastic foundation model using non-linear analysis and P-Y curves obtained for different layers of soil at different depths. The P-Y curves should account for the soil properties applicable for cyclic loading. It should also account for sensitive and reliability studies taking a range of parameters to be adopted for the design. Influence of water table if any shall be accounted-for suitably. Wherever monopiles are adopted, In-situ field pressuremeter test shall be performed to corroborate the assumed P-Y curves. A geotechnical specialist shall be involved in assessing the above-mentioned geotechnical tests & parameters.

ii. Modelling of nonlinearity of material (concrete and steel) of pier and pile system along its depth.

iii. Taking account of construction tolerances (tilt and shift as per IRC-78) in installation of pile. Above-mentioned effect shall be taken in the worst direction.

iv. Effect of any nearby existing utility must be ascertained and effect of the same shall be incorporated, including those planned utility under implementation.

v. Design of monopile shall be carried out taking account of second order effect of combined system of pile and pier.

vi. Slenderness of pier shall account for the combined system of pile and pier.

vii. Deformations at rail level shall be worked out and shall be ensured that all deformations (in SLS Load Combination-I) are within limits as defined elsewhere in this report under different combination of loads.

h) Centre to centre spacing between the piles shall generally be not less than 3 times the diameter of pile in soil and 2 times the diameter when founded on rock.

i) Computation of bending moments along the pile length due to forces applied at the pile cap level shall be based on a space frame model with actual stiffness of piles restrained by springs simulating the soil stiffness

j) Pile cap shall be designed based on truss analogy for a pile group of upto five (5) piles. For other pile groups, bending theory using the space frame model indicated above shall be employed for pile cap design. No support from soil below pile cap shall be considered.

k) The top of pile cap shall be kept about 500mm below the existing ground level and weight of the earth cover will be applied on top of pile cap when unfavourable. The earth cover on pile cap for any favourable effect (stability, soil horizontal capacity) shall be neglected

l) Pile & pile cap foundations shall be with a minimum grade of Concrete of M35.

m) The structural design of the pile and pile cap shall be checked in SLS and ULS conditions. IRS CBC 1997 cl 15.6 shall be used for the piles. However, for crack control in piles, cl 15.6.7 shall be modified to the

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extent that actual axial load shall be considered to act simultaneously. For the pile cap reference shall be made to cl. 15.4 and cl 15.8.3.

n) For pile carrying capacity, check shall be performed without any factorization of loads.



9 MISCELLANEOUS ISSUES

9.1 Pier Cap

9.1.1 Pier cap shall be designed as corbels if shear span to effective depth ratio is less than 1. In other case, it shall be designed as flexural member.

9.2 Drainage of Deck / Solid Pier

Solid pier

9.2.1 The drain pipe of MS/GI with water collecting box at top with cover made up of mesh/jali of MS/GI shall be located within the solid pier to avoid aesthetics problems.

Deck

9.2.2 The top of deck slab of box girder shall be profiled so as collect the run-off water at one points (in the center of deck) by providing a cross inward slope of 2.5%. Runner pipe shall be provided (hung from deck slab) inside the box girder to collect run off through the drain chamber (to be provided in every alternate segment) which shall pass through opening of box girder at one end and then drain off in the water collecting box provided at pier top.

9.3 Minimum thickness of members

9.3.1 Desirable minimum thickness of any concrete member shall be as below:

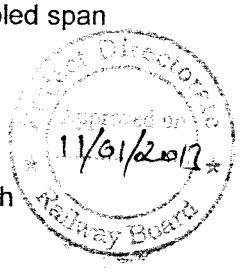
- Deck – 200 mm
- Web of T-beam – 250 mm
- Web of pre-stressed girders – 150 mm + d
- If there are 2 cables at any level – 150 mm + 3d
- (Where d is the outside diameter of the cable duct)
- Box Girders: minimum thickness of member:
- Deck slab – 200 mm
- Web – 250 mm

9.3.2 For superstructure as box girder then the clear height inside the box girder to be minimum 1.0 m to facilitate inspection, or as required by IRS Concrete Bridge Code, whichever is greater.

9.4 Tolerances for finished segments of Pre-cast box Girder

9.4.1 The Tolerances shall not exceed the following:

- Length of match-cast segment (not cumulative): +10.4mm/m, 25mm max.
- Length of totally assembled span : ±12.5 mm
- Web Thickness : ±9.5mm
- Depth of Bottom Slab : ±9.5mm
- Depth of Top flange : ±9.5 mm
- Overall Top Flange Width : ±5.2 mm/m, 25mm max.



- Diaphragm Thickness : ±12.5 mm
- Grade of form edge and soffit : ±1 mm/m
- Tendon hole Location : ±3.2 mm
- Position of Shear Keys : ±6.3 mm

9.5 Allowable Range Of Noise Level

9.5.1 The allowable range of noise levels for different land uses are:

- Residential : 50-70 dbA
- Business And Commercial : 75 dbA
- Hospitals : 60 dbA
- Rural : 45-50 dbA

9.5.2 In addition to parapet which shall form as sound barrier, additional noise barrier shall be provided in lengths of viaducts passing through sensitive residential or hospital zones. The choice of barrier type and their disposition along the parapet /railing shall be closely related to aesthetics of the structures.

9.6 Detailing Aspects

Laps of Rebars

9.6.1 For lapping of bars, guidelines given in IRC:21:2000 shall be followed.

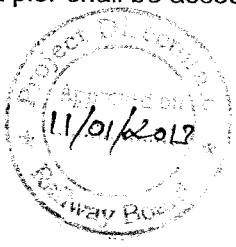
Mechanical coupling of bars

9.6.2 Mechanical couplers are permitted to be used however it should be able to meet the testing requirements as given in MORTH. Mechanical coupling devices shall be arranged so that as small a number as possible affect a single section. They should, in addition, be placed outside the most highly stressed sections. Wherever the same shall be adopted, not more than 20% of bars shall be coupled at one location and splices shall be staggered by 600mm.

9.6.3 Mechanical joint including its connecting elements shall develop in tension /compression atleast 125% of the characteristic strength.

9.7 Provisions for Skywalk / Footover bridges

9.7.1 Forces and Loads due to any planned skywalk / footover bridges (underneath viaduct) supported on permanent pier shall be accounted for.



10 LIST OF DESIGN CODES AND STANDARDS

10.1.1 The following codes will be used in various stages of works (non-exhaustive list):

IRS Codes

- IRS Substructure & Foundation Code 2003 (Include. CS 23 To CS 28)
- IRS Bridge Rules- 2008 (Include. CS 40)
- IRS Concrete Bridge Code 2003 (Include. CS 8 To CS 13)

IRC Codes

- IRC: 5:1998 Standard Specification & Code Of Practice For Road Bridges-General Features Of Designs (Sixth Revision)
- IRC-6-2010 (incl notification no-67) Standard Specification & Code Of Practice For Road Bridges- Loads And Stresses (Fifth Revision)
- IRC: 18-2000 Design Criteria For Prestressed Concrete Road Bridges (Post Tensioned Concrete) (3rd Revision)
- IRC: 21-2000 Standard Specification & Code Of Practice For Road Bridges -- Cement Concrete (Plain & Reinf) (2nd Revision)
- IRC:78-2000 Standard Specifications & Code Of Practice For Road Bridges--Section Foundations & Sub-Structure. (1st Revision)
- IRC-83-1999 Standard Specifications & Code Of Practice For Road Bridges, Part-I Metallic Bearings
- IRC: 83-1987 Standard Specifications & Code Of Practice For Road Bridges, Part-II Elastomeric Bearings
- IRC: 83-2002 Standard Specifications & Code Of Practice For Road Bridges, Part-III Pot, Pot-Cum-PTFE, Pin And Metallic Guide Bearings
- IRC:SP:64-2005 Guidelines For Design Of Construction Of Segmental Bridges
- IRC:SP:67-2005 Guidelines For Use of External And Unbonded Prestressing Tendons In Bridge Structures
- IRC:SP:70-2005 Guidelines For The Use Of high Performance Concrete In Bridges
- IRC:SP:71-2006 Guidelines For Design And Construction Of Precast Pre-tensionioned Girder For Bridges

IS Codes

- IS: 269-1976 Specs for Ordinary and Low Heat Portland cement
- IS: 383-1970 Specs for coarse and fine aggregate from natural sources for concrete
- IS: 432-1982 Specs for Mild Steel & medium tensile steel bars (Part 1)
- IS: 455-1976 Specifications for Portland Slag Cement

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- IS: 456-2000 Code of Practice for Plain and Reinforced Concrete - based essentially on CP-110
- IS: 800-2007 Code of Practice for General construction in steel
- IS: 875-1987 Code of Practice for Design Loads Parts 1,2,3,4 & 5 (other than Earthquake) for Building and structures
- IS: 1080-1985 Design and Construction of shallow foundations in soils (Other than Raft, Ring & Shell)
- IS: 1364-1992 Hexagon Head Bolts, screws & nuts of product grades A & B Part 1.
- (Part 1) Hexagon Head Bolts (size range M1.6 to M64)
- IS: 1489 (Part 1) Specifications for Portland Pozzolana Cement (Flyash based)
- IS: 1786-1985 Specs. for High Strength Deformed steel bars and wires for concrete reinforcement
- IS: 1893(Part 1) 2002 Criteria for Earthquake Resistant Design of structures
- IS: 1904-1986 Design and Construction of Foundation in soils: General Requirements
- IS: 2062-2006 Hot Rolled Low, Medium and High Tensile Structural Steel
- IS: 2502-1963 Code of Practice for Bending and Fixing of Bars for

...Foundations

...Practice for Earthquake Resistant ...Construction of Buildings

- IS: 4923-1997 Hollow steel sections for structural use -specif
- IS: 8009-1976 Calculation of settlement of shallow foundation
- IS: 8112-1989 Specification for 43 Grade Ordinary Portland c
- IS: 9103-1999 Specifications for Admixtures for Concrete
- IS: 12070-1987 Code of Practice for Design and Constru... Shallow Foundations on Rocks
- IS: 12269-1987 Specifications for 53 Grade Ordinary Portland
- IS: 13920-1993 Ductile Detailing of Reinforced Concrete St... subjected to Seismic Forces
- IS: 14268-1995 Uncoated Stress Relived Low relaxation Se... Strands for Prestressed Concrete
- IS: 14593-1998 Design And Construction Of Bored Cast-In-Si... Founded On Rocks-Guidelines

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International Codes

- BS: 4447-1983 Specifications for the Performance of Prestressing Anchorages for Post-Tensioned Construction
- BS: 4486 Specifications for High Tensile Steel bars used for Prestressing
- BS: 5400 Code Of Practice For Design Of Concrete Bridges Part-4-1990
- BS: 5400 Code Of Practice For Fatigue Part-10-1990
- BS: 8006 Code Of Practice For Strengthened Reinforced soils and other fills-1995
- BS: 8007-1987 Design of Concrete Structures for Retaining Aqueous Liquids
- BD 37/01 Loads for Highway Bridges
- BD 58/94 The Design of Concrete Highway Bridges and Structures with External and Unbonded Prestressing
- ACI 358.1R-92 (American Concrete Institute) for assessment of dynamic impact for transit Guideways.
- AASHTO-1996 AASHTO LRFD Bridge Design Specifications–Second Edition 1998
- UIC 772-R Code For The Use Of Rubber Bearing For Rail Bridges
- UIC 776-1R Loads to Considered In Railway Bridge Design
- UIC 776-3R Deformation Of Bridges
- UIC 774-3R Track/Bridge Interaction Recommendations For Calculations
- Eurocode 0 Basis Of Structural Design
- Eurocode 1 Actions On Structures-Part2: Traffic Loads On bridges
- Eurocode 2 Design Of Concrete Structures-Part 1-1 General Rules and Rules for Buildings
- Eurocode 2 Design Of Concrete Structures-Part 2 Concrete Bridges-Design and Detailing Rules

Others

- FIP Recommendations for the Acceptance of Post-Tensioning Systems
- MOST Specifications for Road and Bridge Works – 1994

Miscellaneous

- Any other codes & special publications as required and as mentioned in this report.




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ANNEXURE- IE'S COMMENTS RESOLUTION ON DBR – SECTION 2 - VIADUCT (Ref. Tracking Number – 06-CO-4-S1-R)



		Coating of the pile cap would leave the bottom of cap unprotected; as such other methods of improving the resistance of concrete are considered more effective in line with IRS:CBC provisions prevalent for Railway Bridges in the country.		
2	Cl. 5.1.1: Hunting force to be defined.	As per IRS practice these forces are defined as Raking forces and are used for design of local bracing members of steel bridges only. Cl.5.15.1 of DBR refers to these loads.	Hunting force is different phenomenon than "Raking force". Although as per IRS practice Raking force is only applicable to steel member, consideration should be taken during Detailed Design. Effect of "Hunting Force" should be analyzed at the time of Detailed Design. **(Response Level B)**	Cl. 5.15 has been amended in the DBR (Revision- G) to include nosing forces as per Euro code instead of raking forces as per IRS code. This force is excluded in the seismic combination as train loads are generally not considered in the seismic case in these codes.
3	Cl. 5.1.1: Buffer load at the end of track to be defined.	The Buffer Load has now been defined in Cl. 5.1.1 and a new clause No. 5.21 has been added in the DBR (Rev. –F) , which reads as follows ; **5.21 Buffer Load (BL)** Provision of Buffers is contemplated at the end of temporary terminal stations during stage opening of the Corridors , at Pocket track ends and at the terminal stations of the corridors (at the end of turn back/stabling lines)Such buffers will be of friction type . These Buffers will be designed to have the following stopping performance:	To be verified at the time of Detailed Design. **(Response Level B)** Project Directorate approved on 11/01/2012 Railway Board	Noted

		A fully loaded 6-car train (having a mass of 408 tonnes) shall stop from 15 km/h in a maximum distance of 15m (including the length of buffer stop and its components- i.e. total occupancy distance) without damage to the train or buffer A fully loaded 6-car train (having a mass of 408 tonnes) shall stop from 25 km/h in a maximum distance of 15m (including the length of buffer stop and its components- i.e. total occupancy distance) regardless of damage to the train or buffer. Viaduct elements need to be designed for such Buffer load. The exact Buffer loads need to be interfaced and ascertained during the Detailed Design.		
4	Cl. 5.3.1: Consider CEB-FIP method.	The use of any other code is already permitted in the DBR as per the Cl. 5.3.1.1 and CEB-FIP can be one of them.	To be verified at the time of Detailed Design and issue of Good for Construction Drawings. **(Response Level B)**	Noted
5	Cl.7.7.2.: Consider cover to rebar of the piles to be 75mm.	The provisions in the DBR are as per existing practice for moderate conditions of exposure as defined in MSS. However DBR clearly specifies that (Reference Clause 7.7.2 – section - Durability) "Reinforcement Cover to pile, pile cap & open foundations has to be reviewed based on aggressiveness of soil on case to case basis". Please also refer the response to Sl no. 1 above.	As per CL:304.3.2 of IRC: 21 minimum cover to the foundation should be 75mm. To be verified at the time of Detailed Design and issue of Good for Construction Drawings. **(Response Level B)**	The permissible stresses and the crack width provisions are in line with IRS codes. In our opinion the durability criteria should be in line with the same code instead of IRC code. The minimum cover for foundation will be verified and reviewed based on the aggressiveness of the soil at the detailed design stage.

Project Director
Approved
11/01/2012
Railway Board

ANNEXURE – 1

CDA – LONGITUDINAL

CDA for Longitudinal analysis has been calculated as per IRS – Bridge Rule – Clause 2.4.1.1 applying the formula.

$$CDA = 1 + \left[0.15 + \frac{(8)}{6+L}\right]$$ where L is the span length in Metres carrying the loaded train for

speed considered at 160 KM/Hr. Correction factor on a) Speed and b) double track on single structure is considered for calculating final CDA figure for spans.

- For Spans 28^m & above : Longitudinal CDA has been calculated as 1.20 as follows :

$$CDA=1 + \left[0.15 + \frac{(8)}{6+L}\right] = 1+ \left[0.15 + \frac{8}{6+26.2}\right] = 1+ \left[0.15 +0.248\right] = 1+ \left[0.398 \text{ X } 0.72\text{X}\frac{90}{125}\right] = 1.20$$

- Similarly for Spans 25^M and applying corrective factors CDA = 1.217

- For Station span of 13.4 ^{M} having a train operating speed of 60 KMPH and applying corrective factor CDA = 1.207

- For all other spans, Longitudinal CDA is calculated based on span length, and operating speed applying corrective factor, as per IRS-Bridge Rule, applicable as above.

- Superstructures have been designed adequately taking into account respective applicable CDAs as demonstrated above.



ANNEXURE – 2

CDA – Transverse

C/C of Axle in Bogie = 2.2 ^{m}
Depth of plinth including Rail = 0.55 ^{m}
Total dispersed length from rail top to Deck top

= 0.55 x 2 + 2.2
= 3.3 ^{m} - (1)

Additional Dispersion through slab as per clause no: 2.3.4.2.(b) of IRS bridge rules

= ¼ on each side x 2 x 4 (Span of Deck slab between two Webs)

= 2 m - (2)

Total (1) + (2) = 3.3 ^{M} + 2 ^{M} = 5.3 ^{M}

Applying in formula:

$$1 + \left\{0.15 + \frac{8}{6+L}\right\} = 1 + \left\{0.15 + \frac{8}{6+5.3}\right\}$$

$$= 1 + \{0.86\} = 1 + \left\{0.86 \times 0.72 \times \frac{90}{125}\right\}$$ (Considering reduced co-eff & speed effect)

= 1.45

∴ CDA (Transverse) = 1.45

11/01/2013
Railway Board