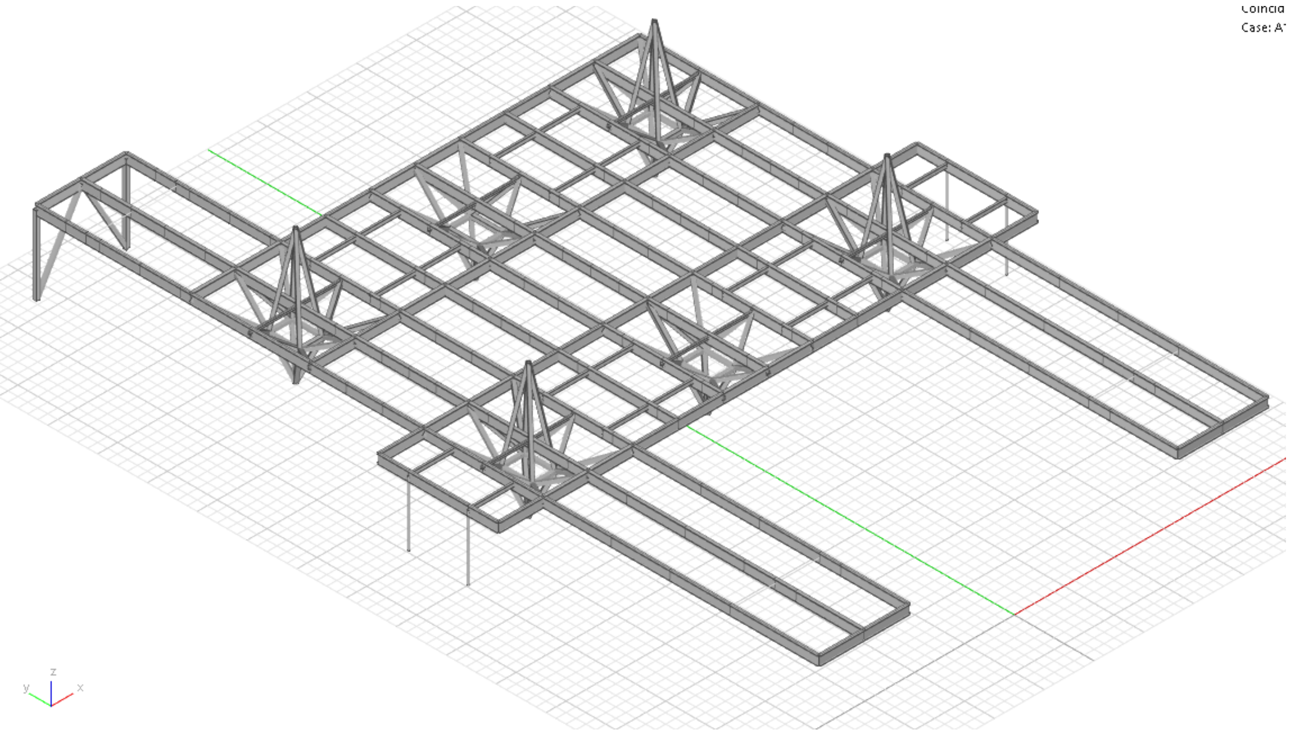
Rail Land Development Authority

Redevelopment of Safdarjung station

Structural Design Basis

Reference: 284197-ARP-ZZ-ZZ-RP-ZX-000003

Issue 05 | 05 December 2023



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Job number 284197-10

Arup India Private Limited
Reg No U29141MH2005PTC224526

Arup India Private Limited
5th Floor Jet Prime Suren Road
Off Western Express Highway
Andheri (East)
Mumbai 400 093
India
arup.com

ARUP

Document Verification

Project title	Redevelopment of Safdarjung station
Document title	Structural Design Basis
Job number	
Document ref	284197-ARP-ZZ-ZZ-RP-ZX-000003
File reference	Design-basis.docx

Revision	**Date**	**Filename**	IR Safdarjung Design Basis Rev01		
Issue 1	17-Feb-2023	**Description**	Issue for client review		
			Prepared by	**Checked by**	**Approved by**
		Name	Prerak Shah Eva Babic	Shrirang Agrawal Rahul Gehlot Francesco Banchini	Francis Archer
		Signature			
Issue 2	07-Apr-2023	**Filename**	IR Safdarjung Design Basis Rev02		
		Description	Issue for client review		
			Prepared by	**Checked by**	**Approved by**
		Name	Prerak Shah Eva Babic	Shrirang Agrawal Rahul Gehlot Francesco Banchini	Francis Archer
		Signature			
Issue 3	16-Aug-2023	**Filename**	IR Safdarjung Design Basis Rev03		
		Description	Revised issue following client review		
			Prepared by	**Checked by**	**Approved by**
		Name	Eva Babic Surya Raghu	Francis Archer Francesco Banchini	Francis Archer
		Signature			

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Revision	Date	Filename	IR Safdarjung Design Basis Rev04		
Issue 4	21-Aug-2023	Description	Revised issue following client review		
			Prepared by	Checked by	Approved by
		Name	Eva Babic Surya Raghu	Francis Archer Francesco Banchini	Francis Archer
		Signature			
Issue 5	05-Dec-2023	Filename	IR Safdarjung Design Basis Rev04		
		Description	Revised issue incorporations comments from Norther Railway – highlighted		
			Prepared by	Checked by	Approved by
		Name	Eva Babic	Francis Archer	Francis Archer
		Signature			
		Filename			
		Description			
			Prepared by	Checked by	Approved by
		Name			
		Signature			

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Abbreviations

General Definitions

CCTV	Closed Circuit Television
F&B	Food and Beverage
IR	Indian Railways
MCE	Multi-Criteria Evaluation
MEP	Mechanical, Electrical and Plumbing
PAVA	Public Address & Voice Alarm
PHT	Peak Hour Traffic
PV	Photovoltaics
OHE	Overhead Equipment
ToR	Terms of Reference
VCE	Vertical Circulation Equipment

Modularised components

FOB	Foot Over Bridges
ACC	Air Concourses
COP	Cover over Platform
THR	Through Roofs
EEB	Entry and Exit Blocks

Standards and Codes

ASHRAE	American Society for Heating, Refrigeration and Air-conditioning Engineers
COP	Code of Practice
ECBC	Energy Conservation Building Code
MSSR	Manual of Standards and Specifications for Railway Stations
NBC	National Building Code of India
IGBC	Indian Green Building Council
ISHRAE	Indian Society for Heating, Refrigeration and Air-conditioning Engineers
IRC	Indian Roads Congress
IRS	Indian Railway Standards by Ministry of Railways
IS	Indian Standards

Executive summary

This Report comprises requirements and assumptions utilised in the design of the primary structure of the Safdarjung Railway Station, New Delhi. This report outlines the design philosophy of the structure and identifies the fundamental load paths the that the system relies on, as well as specifies the basic design parameters, such as superimposed loads, serviceability criteria and ground conditions.

This document shall be read in conjunction with other relevant project reports, structural drawings, specifications and calculation plan.

1 Introduction

The report sets out the basic design parameters, applied loads and design philosophy for the primary frame.

The design of the substructure, all precast concrete elements and cast-in connections to steelwork have been designed and detailed by specialist subcontractors in accordance with the project performance specifications. Therefore, the details of their design parameters are not included within this report, however the design philosophy of how these behave within the global structure are considered and described herein.

2 Basis of Design

2.1 Codes of practice and standards

2.2 List of codes

The building structure shall be designed using the list of the relevant codes and standards are given below.

• IRS B1-2001 – Specification for fabrication and erection of steel girder bridges and locomotive turn-tables (Fabrication specification)

• IRS Welded Bridge Code 2001

• Indian Railways Bridge Manual 1998

• IRS – Code of practice for the design of steel or wrought iron bridges carrying rail, road or pedestrian traffic (Steel Bridge Code)- 1962

• IRS - Seismic Code for Earthquake Resistant design of Railway Bridges (2020)

• IRS - Code of Practice for Plain, Reinforced & Prestressed Concrete for General Bridge Construction -Second revision- 1997

• IS 875 (Part 1): 1987 - Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures- Unit Weights of Building Materials and Stored Materials

• IS 875 (Part 2): 1987 - Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures-Imposed Loads

• IS 875 (Part 3): 2015 - Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures-Wind Loads

• IS 875 (Part 5): 1987 - Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures-Special Loads and Load Combinations

• IS 1893 (Part 1): 2016 (Amendment 2) – Criteria for Earthquake Resistant Design of Structures- General Provisions and Buildings

• IS 1893 (Part 3): 2014 – Criteria for Earthquake Resistant Design of Structures-Bridges and Retaining Walls

• IS 432 (Part 1): 1982 – Specification of mild steel and medium tensile steel bars and hard drawn steel wire for concrete reinforcement – Mild steel and Medium Tensile Steel Bars

• IS 432 (Part 2): 1982 – Specification of Mild Steel and Medium Tensile Steel bars and Hard drawn Steel Wire for concrete reinforcement – Hard drawn Steel Wire

• IS 4326: 2013 - Code of Practice for Earthquake Resistant Design and Construction of Buildings

• IS 13920: 2016 - Ductile Design and Detailing of Reinforced Concrete structures subjected to lateral forces

• IS 456: 2000 - Code of Practice for plain and reinforced concrete

• IS 1786: 2008 - Specification for high strength deformed steel bars and wires for concrete reinforcement

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• IS 800: 2007 - Code for practice for general construction in steel

• IS 2911: 2010 – Code of Practice for Design and Construction of Pile Foundations

• IS 1892: 1979 – Code of Practice for subsurface investigation for foundations (first revision)

• IS 1343: 2012 – Prestressed concrete – code of practice (second revision)

• IS 1786: 2008 – High strength deformed steel bars and wires for concrete reinforcement – specification (fourth revision)

• IS 1498: 1970 - Classification and identification of soils for general engineering purposes

• IS 1904: 1986 - Code of practice for design and construction of foundations in soils: General requirements

• IS 1080: 1985 - Code of Practice for Design and Construction Of Shallow Foundations In Soils (Other Than Raft, Ring And Shell)

• IS 2950(Part 1): 1981 - Code for practice for design & construction of Raft foundation

• IS 3370 (Part 1): 2021 - Code of practice for concrete structure for storage of liquids – Reinforced concrete structures

• IS 814:2004 - Covered electrodes for manual metal arc welding of carbon and carbon manganese steel – specification (sixth revision)

• IS 1367:2002 - Technical supply conditions for threaded steel fasteners

• IS:1785-part1 - Specification for plain hard-drawn steel wire for prestressed concrete, part 1 – cold drawn stress-relieved wire, second revision)

• IS 2062:2011 - Hot rolled medium and high tensile structural steel – specification (seventh revision)

• IS 14593:1988 - Design and construction of bored cast-in-situ piles founded on rocks-Guidelines

• IS 16700: 2017 – Criteria for Structural Safety of Tall Concrete Buildings

• IS 8009 (Part 1): 1976 – Code of Practice for calculation of settlement of foundations

• IS 10262: 2009 – Code of Practice for Design and Construction of Raft Foundations

• IS 1642: 1989 – Indian Standard Code of Practice for Fire Safety of Buildings (General): Details of Construction

• IS 4326: 2013 – Code of Practice for Earthquake Resistant Design and Construction of Buildings

• IS 3414: 1968 – Code of Practice for design and installation of joints in buildings

• IS 10297: 1982 – Code of Practice for design and construction of floors and roofs using precast reinforced/ prestressed concrete

• IRC 6: 2017 – Standard specifications and code of practice for road bridges

• IRC 112: 2020 – Code of practice for concrete road bridges

• NBC 2016 - The National Building code of India, 2016

2.3 Other International Standards and References (wherever needed)

• ACI 318:19 – Building Code requirements for Structural concrete

• ASCE 7-10 – Minimum Design Loads for Buildings and other structures

• AS/NZS 1170.2:2002 – Australian/ New Zealand Standards; Structural design actions; Part 2 Wind actions (wherever needed)

• UBC 1997 – Uniform Building Code (seismic classification)

• ACI 435 – Control of deflection in Concrete Structures

• BS EN 1994-1-1 - Eurocode 4: Design of composite steel and concrete structures. General rules and rules for buildings

2.4 Other References

The following references are used for the structural design.

• Geotechnical investigation report

• SP 16 - Design Aids for Reinforced Concrete

• SP 34 - Handbook on Concrete reinforcement & Detailing

• SP 6 – Handbook for structural engineers – steel

2.5 Hierarchy of codes

The hierarchy of standards in generally to be taken as listed below:

1. IRS
2. IS
3. IRC
4. Eurocode/BS/International codes

Wherever a different approach has been taken, it will be stated in the basis of design or calculation plan.

Indian Railway Codes/(IRS) corrected up to date shall be given 1st preference. Other codes shall be referred for only for those clauses which are not specified in IRS codes. Structure shall be checked for critical Load combination specified in applicable IRS codes.

Sachin Kumar

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3 Description of structures

3.1 Air Concourse

An air concourse is a waiting space for departing passengers positioned above the platforms and tracks. It is linked to the platforms via vertical transportation, which includes stairs, escalators and lifts.

The bottom of air concourse is at least 7.3m above the top of rails (as per RDSO letter no TI/ OHE/ IRSDCL / 2017 dated 31.10.2017). A through roof continues across the concourse deck with enough overlap to provide protection from rain and sun.

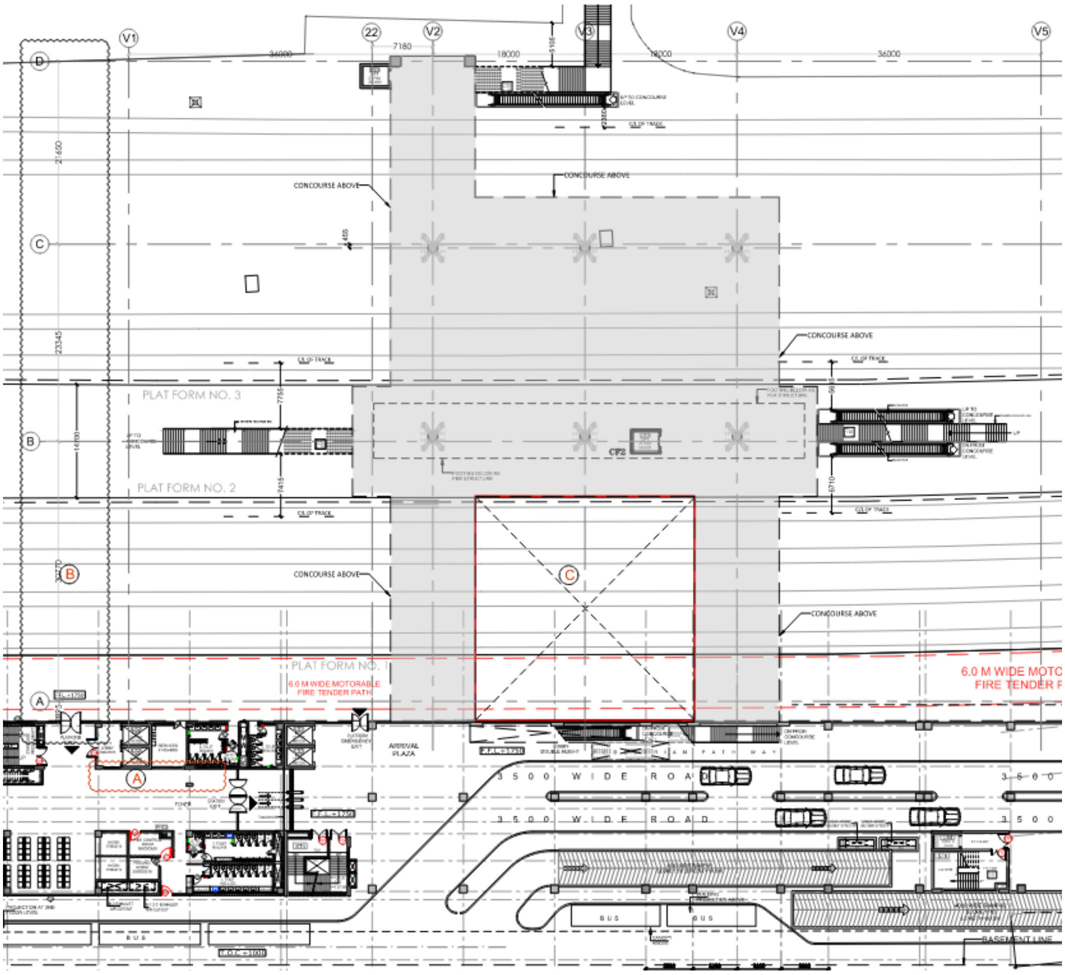

Figure 1: Air concourse level plan

3.2 Through Roof

The through roof steel structure consists of modules that fit together to form a continuous cover across all the platforms and tracks between them. The height of the roof relative to the platform FFL varies from 14.25m to 17.2m allowing for a generous space above the concourse deck.

The spacing of the roof columns along a platform must be sufficient to allow for the escalators and stairs between platform and concourse to have sufficient run off at the top and bottom. This dictates a choice of 36m spacing of roof columns along the length of platform. The spacing of the columns across the platforms depends instead on Safdarjung's specific platform layout. This has been decided based on the architectural intent. Between platform 2-3 and platform 4 (at GL C) the roof columns are spaced at 23.35m c/c.

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3.3 Foot Over Bridge

A Foot Over Bridges (FOB) is provided as the primary N-S connection from the air concourse to the station entrances.

The shorter FOB to the North spans ~15.5m. The two FOBs to the South of the Air Concourse are supported from the building frame and span ~27.7m

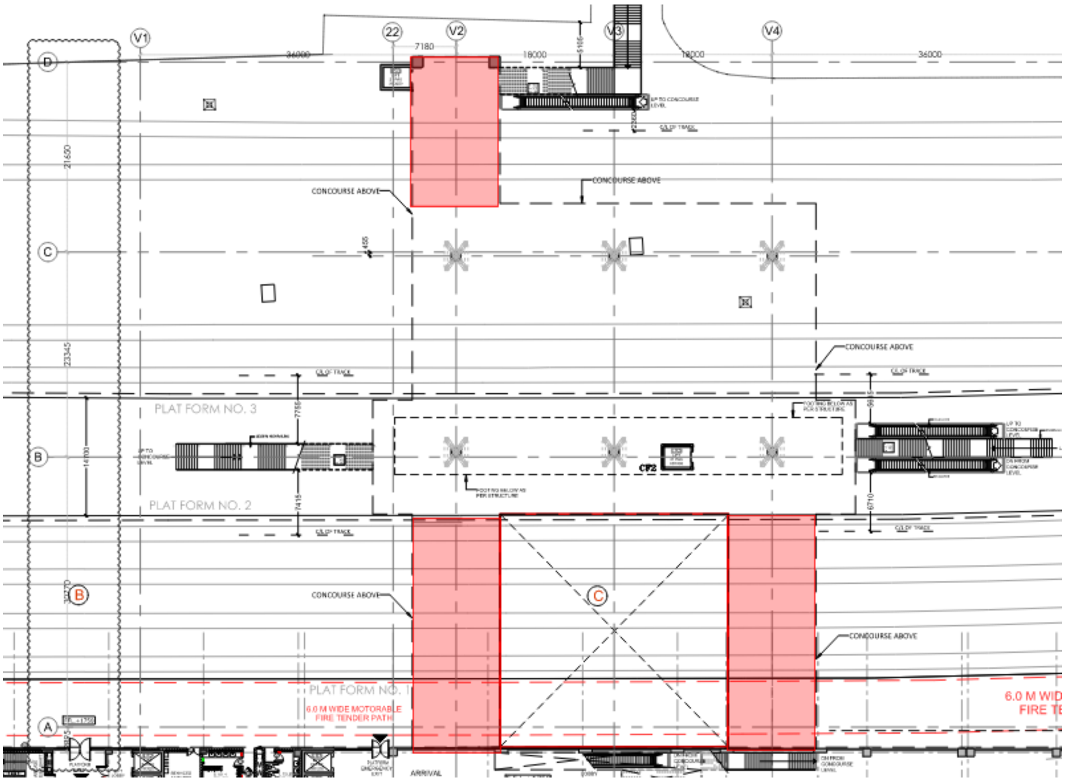

Figure 2: Foot Over Bridges in station setting

3.4 Platform canopies

Platform canopies serve to provide shelter to passengers. Since being in a close proximity to passengers, it is imperative that these canopies are aesthetically pleasing.

These steel structures cover entire width of the platform in Safdarjung we propose canopies supported by a central column or by a pair of columns.

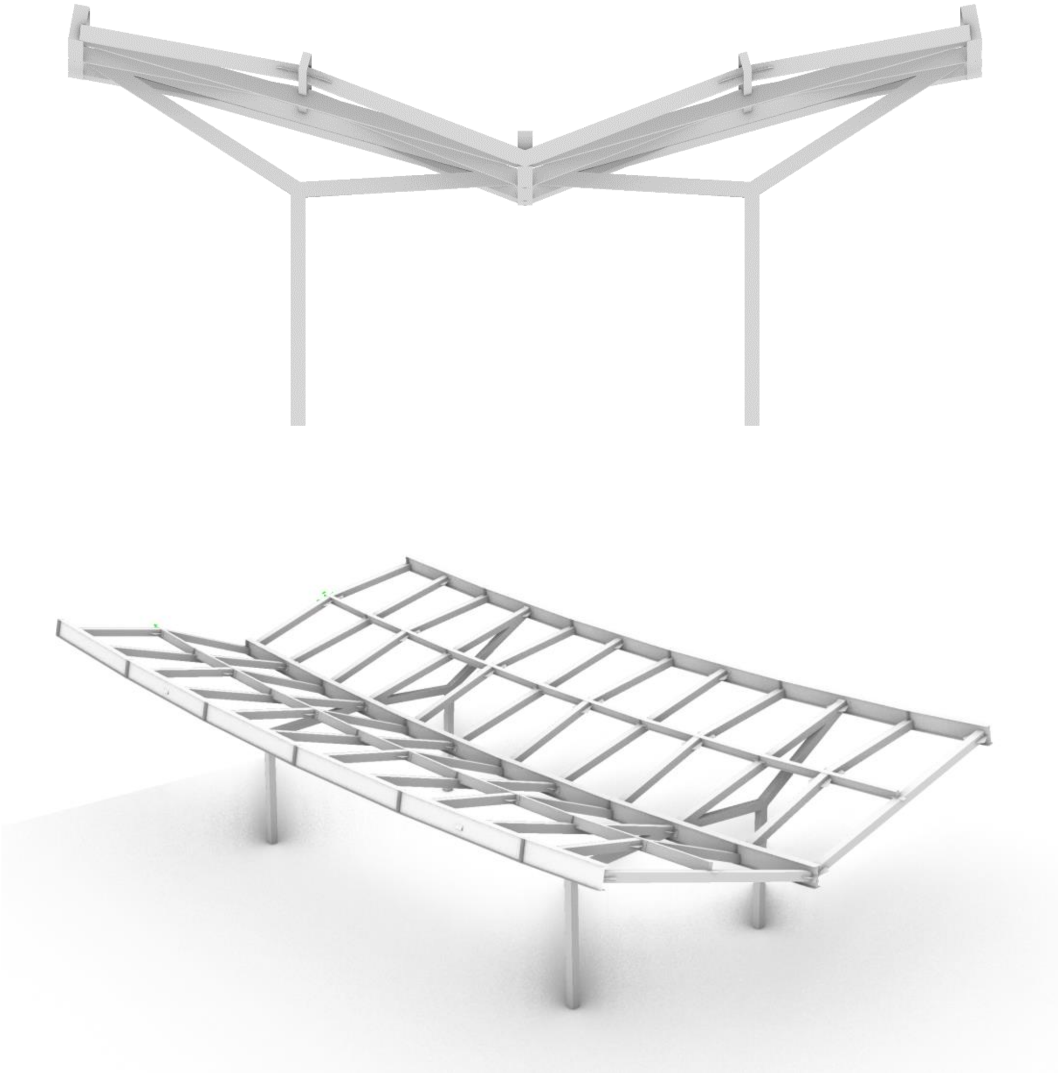

Figure 3: Platform canopy supported by a pair of inclined columns.

3.5 Movement joint strategy

There are no movement joint in the station.

4 Design life of structures

Design life of Air concourse, Through roof, FOB and Cover over platform is taken as 100 years as per relevant standards.

4.1 Air concourse, Through roof and Cover on platforms

The structures can be classified under "All general buildings and structures" as per Table-1 in IS875 (Part 3):2015. The mean probable design life according to the code is 50 years.

While the code allows to design for a 50 years lifespan we decided to design for 100 years working life instead to account the importance of the building.

4.2 Foot Over Bridges

The design working life of the foot over bridges shall be 100 years in accordance with IRC:112-2020 Cl. 5.8.1 and in line with Table 2.1 of EN 1990.

Non-replaceable elements must be designed for a design working life of 100 years:

- foundation structures
- concrete and steel structures. (Piers, deck)

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5 Loads

Characteristic loads shall be determined from the various parts of IS875 and IS1893, except were stated otherwise in this section.

5.1 Dead loads

The dead load of structure is primarily the self-weight of structural members which is calculated based on known unit weight of the materials in accordance with IS 875: 1987 Part 1.

A self-weight of reinforced concrete equal to 25kN/m^3 and a self-weight of steelwork equal to 78.5kN/m^3 will be considered in structural design.

In addition to above, a self-weight multiplier of 1.10 will be employed for steel structures to account for the weight of connection plates, angles, bolts and welds.

5.2 Superimposed dead loads

5.2.1 Air concourse

Table 1 - Superimposed dead loads on air concourse

Description	Superimposed dead load	Notes
Floor Finish	3.7 kN/m^2	As per approved thickness of max 160mm
Services	0.3 kN/m^2	
Total	**4.00 kN /m^2**	

5.2.2 Through Roofs

Table 2 - Super imposed dead load on through roofs

Description	Superimposed dead load	Notes
Cladding and ceiling	0.50 kN/m^2	
Metal deck (100mm)	0.10 kN/m^2	
Photovoltaic panels	0.25 kN/m^2	
Total	**0.85 kN /m^2**	**or as per actual as provided by roof manufacturer**

5.2.3 Foot Over Bridges

Table 3 - Superimposed dead loads on foot over bridges

Description	Superimposed dead load	Notes
Floor Finish	3.7 kN/m^2	As per approved thickness of max 160mm
Services	0.3 kN/m^2	
Total	**4.00 kN /m^2**	

5.2.4 Platform canopies

Table 4 - Super imposed dead load on platform canopies

Description	Superimposed dead load	Notes
Cladding & Ceiling	0.25 kN/m^2	
Solar Panel services	0.25 kN/m^2	
Total	**0.5 kN/m^2**	**or as per actual as provided by roof manufacturer**

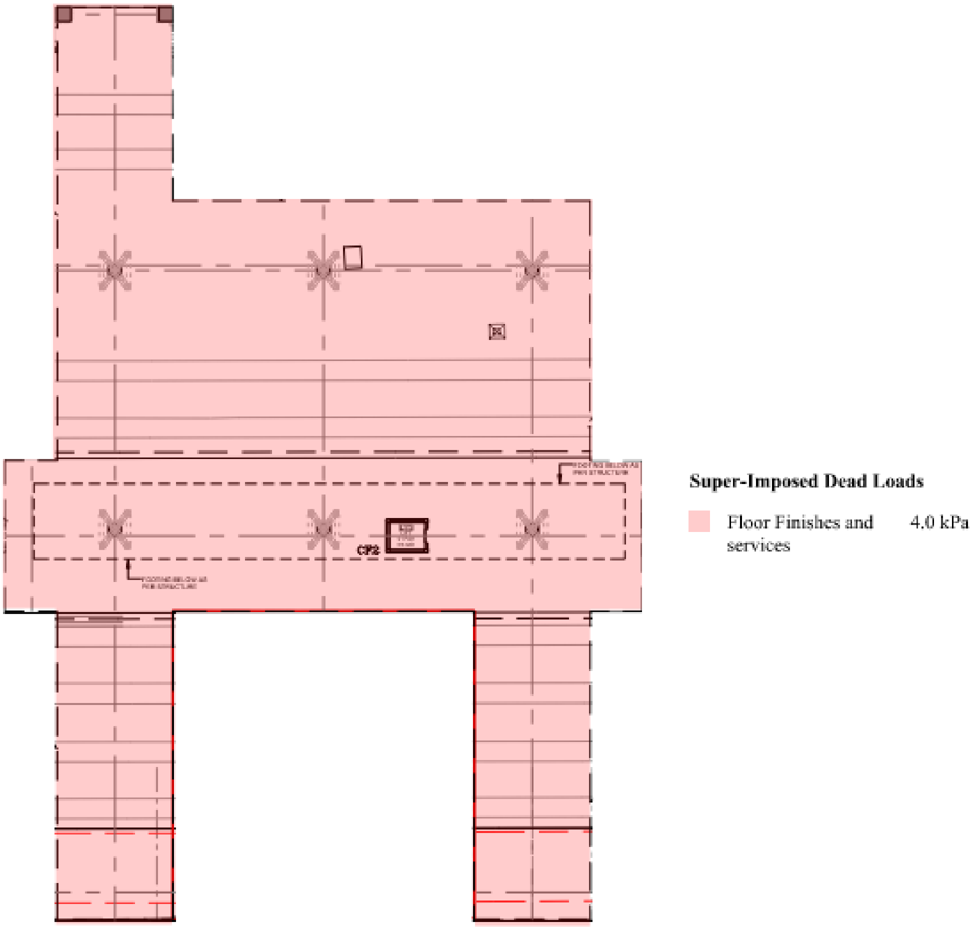

Figure 4 - Load plan for super-imposed dead loads.

5.3 Live/ Roof Live loads

5.3.1 Air concourse and foot over bridge

Permanent

As per IR policy, air concourse is treated as FOB as per IR policy, and therefore live load on air concourse and FOB is taken as 490 kg/m2 (4.8 kPa) as per clause 2.3.2 of IRS Bridge Rule.

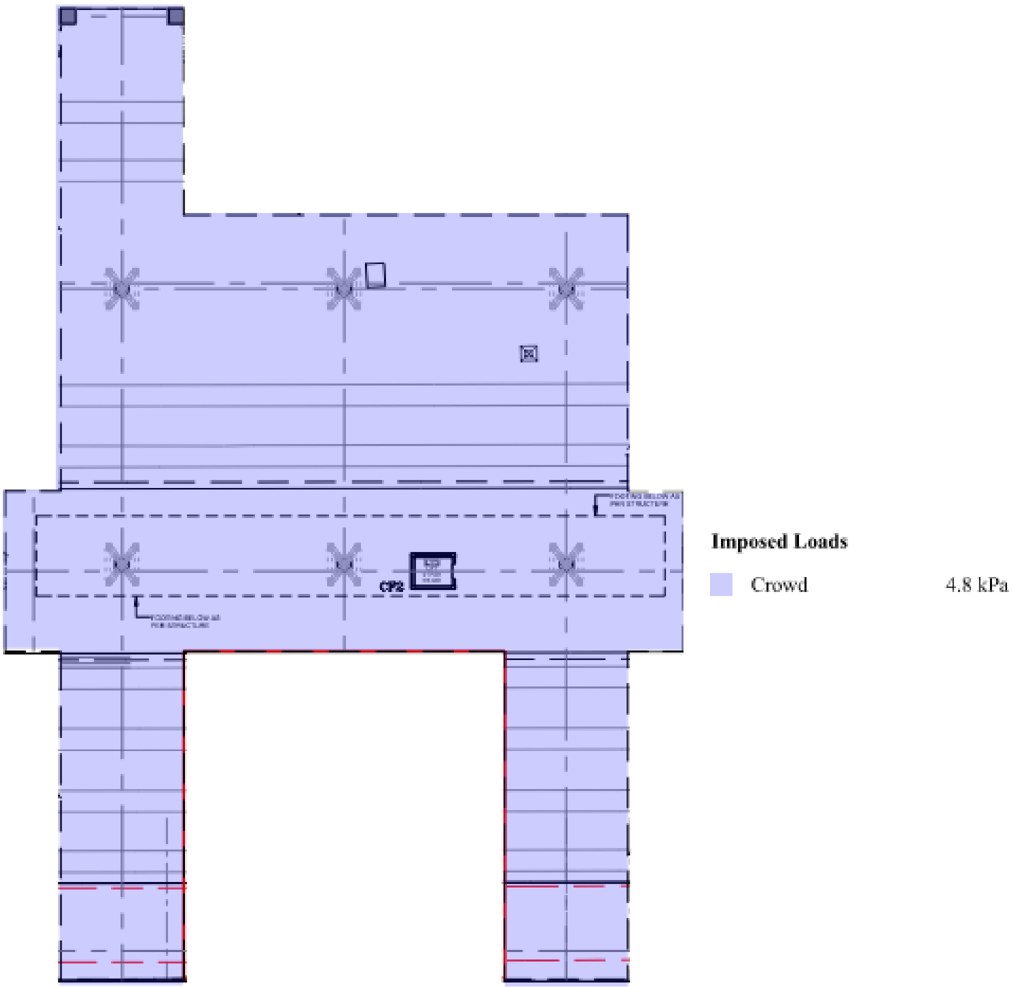

Figure 5 - Load plan for imposed loads.

Parapets

The pedestrian parapets are designed as Type 1 according to IRC:6 Cl. 206.5 (solid/partially filled parapets continuously cantilevering along full length of deck). A horizontal and vertical load of 1.50 kN/m acting simultaneously on the top level of the parapet is applied.

5.3.2 Through roofs

It is assumed that there will be no accumulation of water due to rain. Hence, load due to ponding effect of rain is not taken into consideration.

The roof will be non-accessible. Hence, according to IS 875 Part-2, Table-2, Sl. No. 1 (ii), the imposed load is 0.75 kN/m^2.

5.3.3 Platform canopies

As for through roofs.

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5.4 Wind actions

5.4.1 Design wind pressure according to IS 875

Wind load is determined in accordance with IS 875.

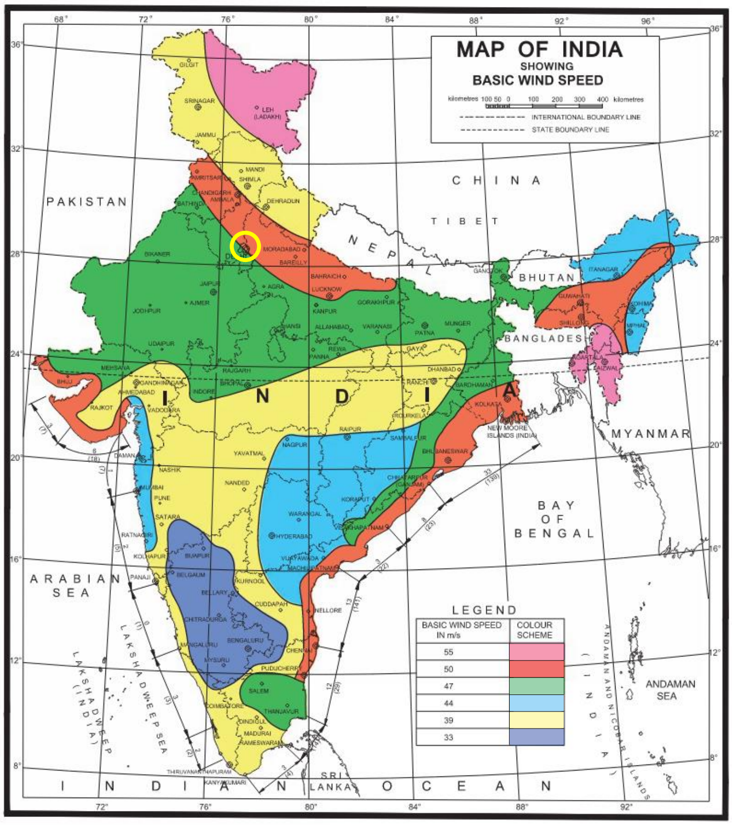

Figure 6: Basic wind speed in m/s based on 50-years mean return period at Safdarjung railway station (Fig. 1, IS 875-3)

The basic wind speed at 10m height for Safdarjung station considered in this study is presented in the table below.

Table 5 - Basic wind speed for Safdarjung (Fig. 1, IS 875-3)

City/ Town	Basic Wind Speed V_b (m/s)
Safdarjung	50

The basic wind speed considered in the design for a Safdarjung station:

- Basic wind speed V_b: 50m/s

The design wind speed is expressed as:

$V_z = V_b k_1 k_2 k_3 k_4$ (Cl. 6.3, IS 875-3)

For determination of below factors, a mean probable design life of 100 years is assumed.

Table 6 - Factor k1 for basic wind speed

Wind speed (m/s)	**k_1 (Table 1 IS 875-3, Cl. 6.3.1)**
50	1.08

The stations are in urban areas therefore Terrain Category 3 is considered in accordance with Cl. 6.3.2.1, IS 875-3.

Factor k_2 is a terrain height factor which generally increases with increase in the height of the structure. The table below summarises a maximum height above ground level for different type of structures considered. The wind pressure will be considered constant throughout the entire height of the structure.

Table 7: Terrain height factor (k_2)

Structure	**Height z (m)**	**k_2 (IS 875-3 – Table 2, Cl. 6.3.2.2)**
Air Concourse	8.5	0.91
Through Roof	25	1.03

The topographic factor k_3 is assumed equal to 1 (assuming station is not located on any hills or escarpment – Cl. 6.3.3, IS 875-3)

Importance factor k_4=1.0 (Assuming the structures are categorized under all other structures and Safdarjung is not prone to cyclones) (Cl. 6.3.4, IS 875-3)

The design wind pressure is defined as:

$$p_z = 0.6V_z^2 \quad [\mathrm{N/m^2}] \quad (\text{Cl. 7.2, IS 875-3})$$

The following table summarises the design wind pressure used depending on the structure's span, width and without pedestrian traffic: (If k_3 without traffic too high can be reduced)

Table 8 - Design wind pressure for various structures

Structure	**V_b**	**k_1**	**k_2**	**k**	**k_4**	**V_z**	**p_z**	**K_d**	**K_a**	**K**	**p_d**
Air Concourse	50	1.08	0.91	1	1	49.14	1.45	1	1	1	1.45
Through Roof	50	1.08	1.03	1	1	55.62	1.86	1	1	1	1.86

Here,

- k_1 – Probability factor (risk coefficient)
- k_2 – Terrain roughness and height factor
- k_3 – Topography factor
- k_4 – Importance factor for the cyclonic region
- V_z – Design wind speed at height z, (m/s)
- p_z – Wind pressure at height z, (kN/m^2)
- K_d – Wind directionality factor
- K_a – Area averaging factor
- K_c – Combination factor
- p_d – Design Wind Pressure (kN/m^2)

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During construction, the hourly mean wind pressure shall be taken as 70% of the values stated in Table 8.

5.4.2 Wind pressure coefficients determination

5.4.3 Air concourse

Calculation of wind pressure (edges/parapets) on air concourse

The air concourse structure was treated as a flat canopy with a parapet. Hence, in accordance with cl. 7.3.2.2, the following pressure coefficient have been used:

Table 9 - Summary of wind loads on air concourse and foot over bridge

Wind load case	**C_{pe}**	**Solidity ratio Φ**	**Wind pressure/Wind UDL**
Wind uplift	-1.0	1	-1.45 kPa
Wind downward	+0.2	all	+0.29 kPa
Horizontal wind*	n.a.	n.a.	3.915 kN/m

The horizontal wind force on the concourse was derived multiplying the area exposed to the wind by the wind force. The horizontal wind load is applied on both freestanding edges of the air concourse.

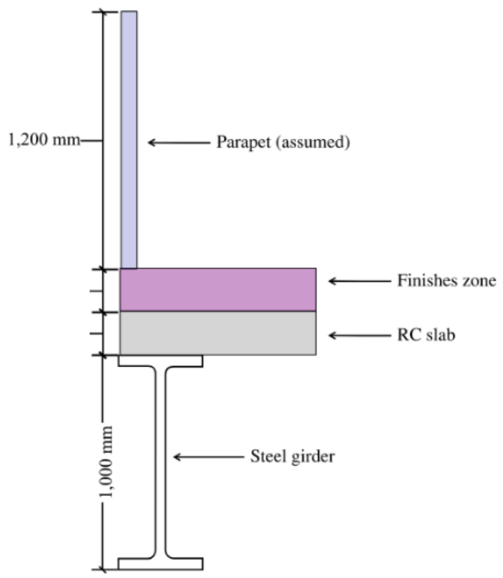
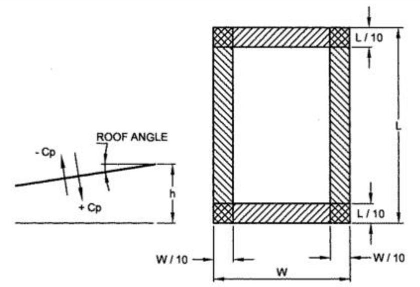


Figure 7 - Assumed cross section at edge of air concourse for wind loads

Table 8 Pressure Coefficients for Monoslope Free Roofs

(*Clause* 7.3.3.3)



ROOF ANGLE (Degree) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS			
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS		
0	All values of Φ	+ 0.2	+ 0.5	+ 1.8	+ 1.1
5		+ 0.4	+ 0.8	+ 2.1	+ 1.3
10		+ 0.5	+ 1.2	+ 2.4	+ 1.6
15		+ 0.7	+ 1.4	+ 2.7	+ 1.8
20		+ 0.8	+ 1.7	+ 2.9	+ 2.1
25		+ 1.0	+ 2.0	+ 3.1	+ 2.3
30		+ 1.2	+ 2.2	+ 3.2	+ 2.4
0	Φ = 0	- 0.5	- 0.6	- 1.3	- 1.4
	Φ = 1	- 1.0	- 1.2	- 1.8	- 1.9
5	Φ = 0	- 0.7	- 1.1	- 1.7	- 1.8
	Φ = 1	- 1.1	- 1.6	- 2.2	- 2.3
10	Φ = 0	- 0.9	- 1.5	- 2.0	- 2.1
	Φ = 1	- 1.3	- 2.1	- 2.6	- 2.7
15	Φ = 0	- 1.1	- 1.8	- 2.4	- 2.5
	Φ = 1	- 1.4	- 2.3	- 2.9	- 3.0
20	Φ = 0	- 1.3	- 2.2	- 2.8	- 2.9
	Φ = 1	- 1.5	- 2.6	- 3.1	- 3.2
25	Φ = 0	- 1.6	- 2.6	- 3.2	- 3.2
	Φ = 1	- 1.7	- 2.8	- 3.5	- 3.5
30	Φ = 0	- 1.8	- 3.0	- 3.8	- 3.6
	Φ = 1	- 1.8	- 3.0	- 3.8	- 3.6

NOTES

1 For monopitch canopies the centre of pressure should be taken to act at 0.3 w from the windward edge.

2 W and L are overall width and length including overhangs.

Figure 8 Wind pressure coefficients mono sloped free roof (Table-8, Clause 7.3.3.3, IS 875-3)

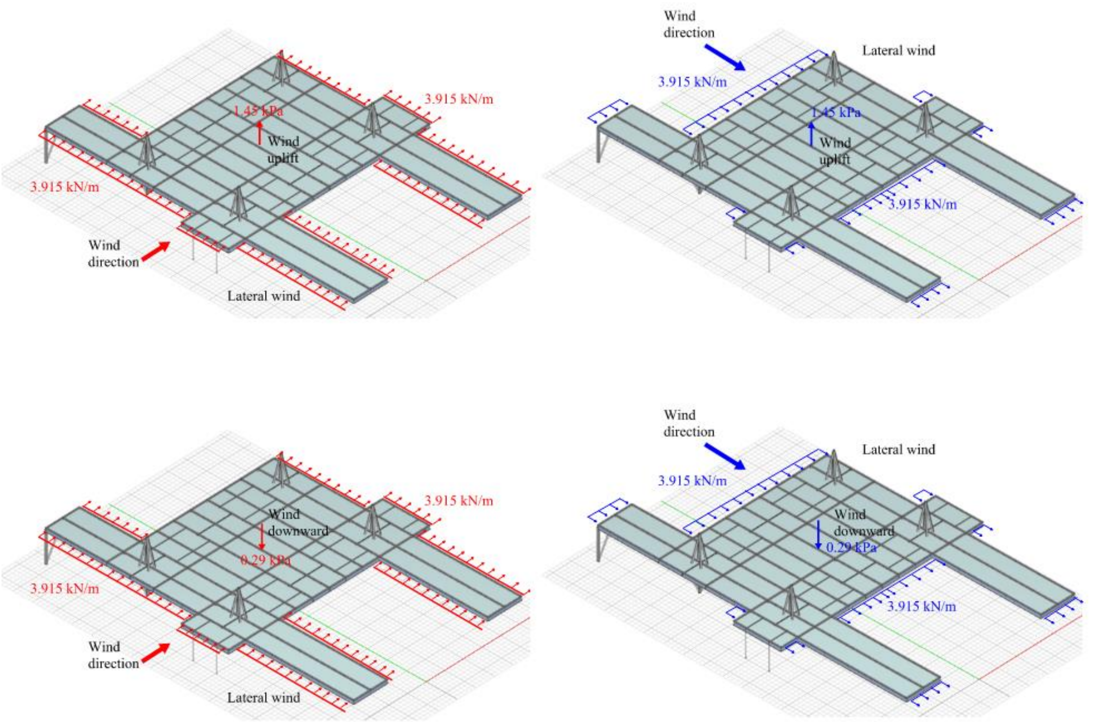


Figure 9 - Wind loads on air concourse.

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5.4.4 Through Roof

X-Direction

To determine wind pressure coefficients in X-direction (along the tracks), the through roof structure can be assumed as free standing multispan double sloped roof with positive roof angle ~10˚.

The pressure on free standing canopies is also dependent on solidity ratio (area of obstruction divided by gross area) beneath the structure. The positive pressure coefficients (towards the surface) are independent of the solidity ratio and remains same for all values of ø.

However, the negative pressure coefficients (away from the surface – uplift) are increasing in nature with increase in the value of ø.

The concourse deck is located below the middle roof vault while the two side vaults are sheltering the platforms only. Given the limited presence of obstructions underneath the roof for this direction, the roof is treated as a canopy with ø = 0.

Table- 9 and clause 7.3.3.3 of IS 875-3 gives the values for overall pressure coefficients for canopy roofs with ¼<h/w<1 and 1<L/w<3 acting normal to the canopy.

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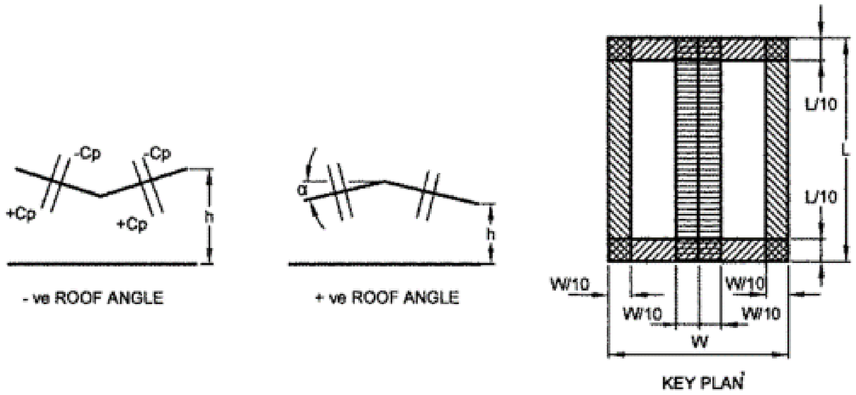
Table 9 Pressure Coefficients for Free Standing Double Sloped Roofs
(*Clause* 7.3.3.3)



ROOF ANGLE (Degrees) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS				
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS			
- 20	All values of Φ	+ 0.7	+ 0.8	+ 1.6	+ 0.6	+ 1.7
- 15		+ 0.5	+ 0.6	+ 1.5	+ 0.7	+ 1.4
- 10		+ 0.4	+ 0.6	+ 1.4	+ 0.8	+ 1.1
- 5		+ 0.3	+ 0.5	+ 1.5	+ 0.8	+ 0.8
+ 5		+ 0.3	+ 0.6	+ 1.8	+ 1.3	+ 0.4
+ 10		+ 0.4	+ 0.7	+ 1.8	+ 1.4	+ 0.4
+ 15		+ 0.4	+ 0.9	+ 1.9	+ 1.4	+ 0.4
+ 20		+ 0.6	+ 1.1	+ 1.9	+ 1.5	+ 0.4
+ 25		+ 0.7	+ 1.2	+ 1.9	+ 1.6	+ 0.5
+ 30		+ 0.9	+ 1.3	+ 1.9	+ 1.6	+ 0.7
- 20	Φ = 0	- 0.7	- 0.9	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.9	- 1.2	- 1.7	- 1.9	- 1.2
- 15	Φ = 0	- 0.6	- 0.8	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.8	- 1.1	- 1.7	- 1.9	- 1.2
- 10	Φ = 0	- 0.6	- 0.8	- 1.3	- 1.5	- 0.6
	Φ = 1	- 0.8	- 1.1	- 1.7	- 1.9	- 1.3
- 5	Φ = 0	- 0.5	- 0.7	- 1.3	- 1.6	- 0.6
	Φ = 1	- 0.8	- 1.5	- 1.7	- 1.9	- 1.4
+ 5	Φ = 0	- 0.6	- 0.6	- 1.4	- 1.4	- 1.1
	Φ = 1	- 0.9	- 1.3	- 1.8	- 1.8	- 2.1
+ 10	Φ = 0	- 0.7	- 0.7	- 1.5	- 1.4	- 1.4
	Φ = 1	- 1.1	- 1.4	- 2.0	- 1.8	- 2.4
+ 15	Φ = 0	- 0.8	- 0.9	- 1.7	- 1.4	- 1.8
	Φ = 1	- 1.2	- 1.5	- 2.2	- 1.9	- 2.8
+ 20	Φ = 0	- 0.9	- 1.2	- 1.8	- 1.4	- 2.0
	Φ = 1	- 1.3	- 1.7	- 2.3	- 1.9	- 3.0
+ 25	Φ = 0	- 1.0	- 1.4	- 1.9	- 1.4	- 2.0
	Φ = 1	- 1.4	- 1.9	- 2.4	- 2.1	- 3.0
+ 30	Φ = 0	- 1.0	- 1.4	- 1.9	- 1.4	- 2.0
	Φ = 1	- 1.4	- 2.1	- 2.6	- 2.2	- 3.0

NOTES

1 Each slope of a duopitch canopy should be able to withstand forces using both the maximum and the minimum coefficients, and the whole canopy should be able to support forces using one slope at the maximum coefficient with the other slope at the minimum coefficient. For duoptich canopies the centre of pressure should be taken to act at the centre of each slope.

2 *W* and *L* are overall width and length including overhangs

Figure 10 Wind pressure coefficients double sloped roof (Table-9, Clause 7.3.3.3, IS 875-3)

Wind pressure calculation in upward and downward direction on through roof is shown in Table 10 below.

Table 10 - Wind pressure (upward/ downward) on through roof for wind in X-direction

Through Roof α **= 10 deg**	p_z **(kPa)**	**Solidity ratio**	**Pressure coefficient**	$P = C_f p_d$ **(kPa)**
Downward pressure	1.86	0	0.4	0.74
Uplift	1.86	0	-0.70	-1.30

Y-Direction

Due to the large radius, it is assumed the roof in Y-direction is not curved, but flat. Therefore, the roof can be simplified as a mono-pitch canopy with roof angle of 0˚. To determine wind pressure coefficients in Y-direction, the wind pressure coefficients for mono-slope free roof can be derived from Cl. 7.3.3.3 and Table-8 of IS 875-3.

In this case, the solidity ratio is assumed as 1 accounting for the presence of the newly constructed office building which obstruct the wind in Y-direction (See Figure 11 below).

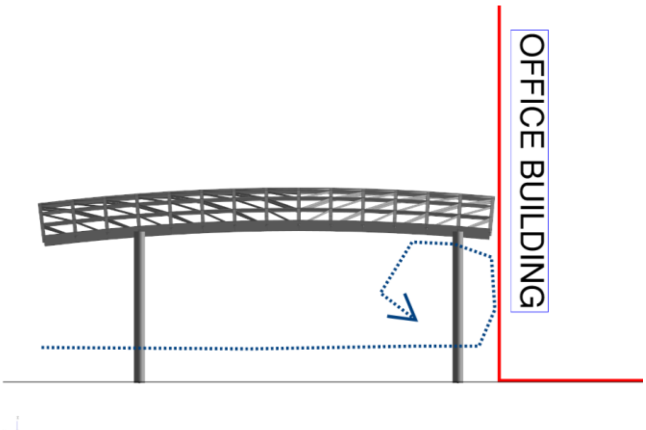
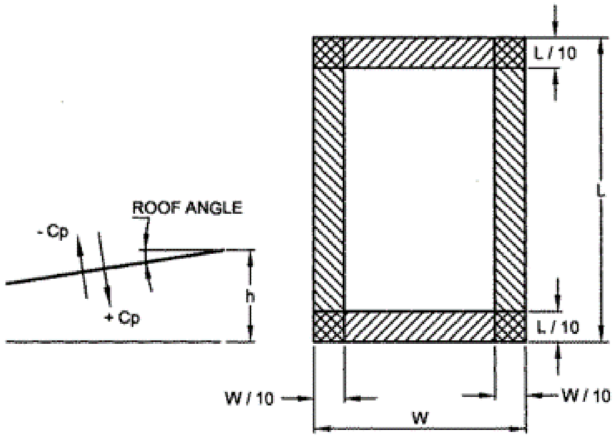


Figure 11 ø=1 for wind in Y-direction

Table-8 and clause 7.3.3.3 of IS 875-3 gives the values for overall pressure coefficients for mono-slope free roofs (Figure 12).

Table 8 Pressure Coefficients for Monoslope Free Roofs

(*Clause* 7.3.3.3)



ROOF ANGLE (Degree) α	SOLIDITY RATIO Φ	MAXIMUM (LARGEST + ve) AND MINIMUM (LARGEST - ve) PRESSURE COEFFICIENTS			
		OVERALL COEFFICIENTS	LOCAL COEFFICIENTS		
0	All values of Φ	+ 0.2	+ 0.5	+ 1.8	+ 1.1
5		+ 0.4	+ 0.8	+ 2.1	+ 1.3
10		+ 0.5	+ 1.2	+ 2.4	+ 1.6
15		+ 0.7	+ 1.4	+ 2.7	+ 1.8
20		+ 0.8	+ 1.7	+ 2.9	+ 2.1
25		+ 1.0	+ 2.0	+ 3.1	+ 2.3
30		+ 1.2	+ 2.2	+ 3.2	+ 2.4
0	Φ = 0	- 0.5	- 0.6	- 1.3	- 1.4
	Φ = 1	- 1.0	- 1.2	- 1.8	- 1.9

Figure 12 Monoslope roof wind pressure coefficient for wind in Y-direction

Table 11 - Wind pressure (upward/ downward) on through roof for wind in Y-direction

Through Roof α = 0 deg	p_z (kPa)	Solidity ratio	Pressure coefficient	$P = C_f p_d$ (kPa)
Downward pressure	1.86	0	0.2	0.37
Uplift	1.86	1	-1.00	-1.86

5.4.5 Platform canopies

Platform canopies will be treated as free standing double sloped canopy roofs see: Table-9 and clause 7.3.3.3 of IS 875-3.

5.5 Thermal actions

Due to the length of the through roof and air concourse a similar approach to thermal loading shall be taken as for bridges.

According to clause 3.4.2 of IS800:2007, “the temperature range varies for different localities and under different diurnal and seasonal conditions. The absolute maximum and minimum temperatures, which may be expected in different localities of the country, may be obtained from the Indian Metrological Department, and used in assessing the maximum variations of temperature for which provision for expansion and contraction has to be made in the structure.”

5.5.1 Uniform temperature component

The highest maximum and lowest Minimum temperatures are stated for stations considered in this study in the table below. For locations other than the stations listed in Annexure F, IRC-6, the values corresponding to nearest station is used. Maximum contraction and expansion range of the uniform bridge temperature component is taken as shown in Table 12

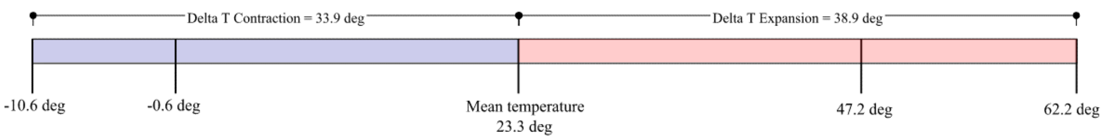
When the difference between the maximum and minimum air shade temperatures exceeds 20°C, the bridge temperature when the structure is effectively restrained is taken as the mean of maximum and minimum air shade temperature ±10°C in accordance with Table 15 of IRC 6-107.

Air shade temperatures considered for the design have been determined based on the Annexure F of IRC 6 at the locations under consideration.

Table 12 (i) Highest maximum and lowest minimum temperature [1]

Station	Shade air Temperature (˚C)		Mean T [°C]	ΔTN,con [°C]	ΔTN,exp [°C]	Note
	Max.	Min.				
New Delhi (Safdarjung)	47.2	-0.6	(47.2+0.6)/2 = 23.3	(-0.6-10) – 23.3 =-33.9	(47.2+15) – 23.3 = 38.9	Steel structure in non-snowbound area

The coefficient of thermal expansion for steel shall be taken as $12x10^{-6}$ (IRC 215.4).



[1] (source: Climatological Normals 1981-2010, IMD, Pune) (Annexure F, IRC: 6 2017) and (ii) Maximum and minimum contraction and expansion range of the uniform temperature component

5.6 Seismic actions

The seismic force on structures considered in this report is calculated based on IRS Seismic Bridge Code. IRS' design procedure and methodology is the same as given in the IS:1893.

5.6.1 Seismic Zone

India is divided in four seismic zones namely zone II, zone III, zone IV and zone V. Safdarjung Station Delhi comes under seismic zones zone IV as per IRS Seismic Bridge Code

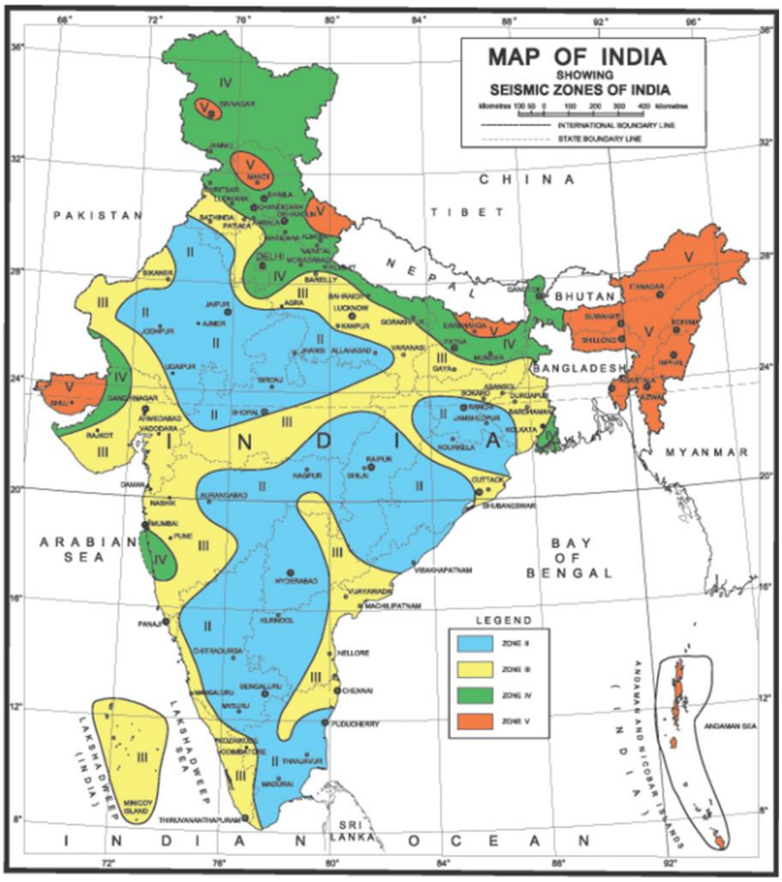


Figure 13 Seismic zones of India, IRS Seismic Bridge Code

The seismic zone factors for various stations under consideration are reported in the table below:

Table 13: Seismic zone factor Z

City/ Town	Zone	Seismic Zone Factor Z
Delhi (Safdarjung)	IV	0.24

5.6.2 Time period calculation

The approximate fundamental translational natural period T_a of oscillation of structure will be calculated in accordance with clause 7.6.2 of IS 1893

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Table 14: Structural time periods

Structure	Type	Height of structure h (m)	Time period calculation formula	Time period (s)
Air concourse	RC-Steel Composite MRF	9	$0.080\ h^{0.75}$	0.42

For the roof structure, the modal analysis and the response spectrum analysis are solely used to calculate the natural period and the seismic actions.

5.6.3 Importance factor (I)

The importance factor for various structures under consideration in this report is derived based on Table 2 (Clause 9.4.4) as per IRS Seismic Bridge Code.

Table 15: Importance Factor

Structure	Importance Factor	Remark
Air concourse	1.5	Important service and community buildings or structures
Through roof	1.5	All other buildings
Platform canopies	1.5	All other buildings

5.6.4 Response reduction factor (R)

Depending upon lateral load resisting system of the structure, response reduction factor is given as per table below in compliance with Table 3, Cl 9.4.5 of IRS Seismic Code for Earthquake Resistant design of Railway Bridges for the Air concourse.

Table 16: Response reduction factors

Structure	Response reduction factor	Lateral load resisting system
Air concourse	2	Superstructure
Through roof	1.0	Cantilever columns, designed in the elastic range.
Platform canopies	1.0	Cantilever columns, designed in the elastic range.

5.6.5 Soil Type

Types of soils are classified as Type I, Type II and Type III according to IRS Seismic Bridge Code. Type I, II and III soils refer to rock or hard soils, medium or stiff soils and soft soils respectively.

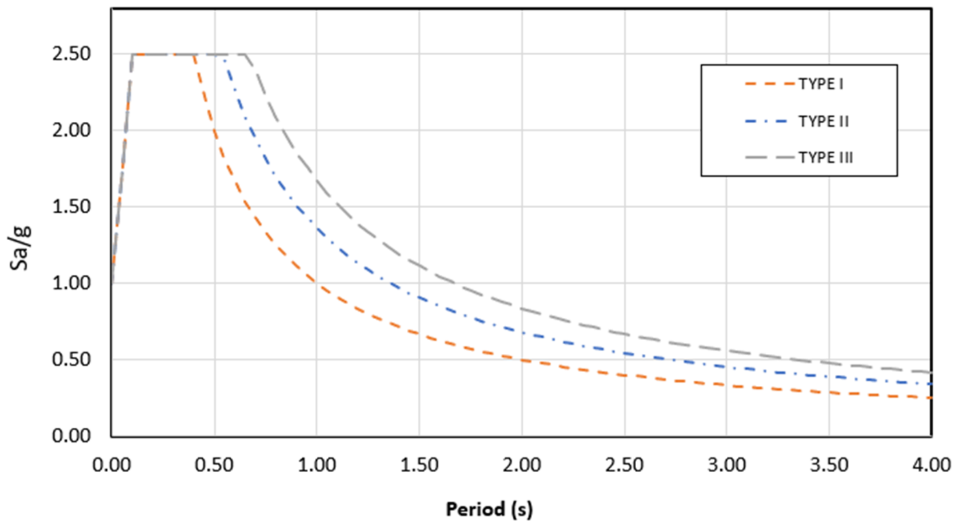

Figure 14: Spectral acceleration coefficient corresponding to 5% damping for response spectrum method

It is necessary to determine the type of soil on which the structure will be placed in order to determine the correct spectrum to be used for estimating S_a/g.

Based on the type of foundation and soil, the net bearing pressure in soils can be increased as per Table 1 and Table 2 of IS1893:2016.

We have not received a soil investigation report but our understanding is that a ground Type II is appropriate.

Our assumptions are based onto the Annex C of IS 1893-Part 1: 2016 which provides an indication of the range of geologies of India.

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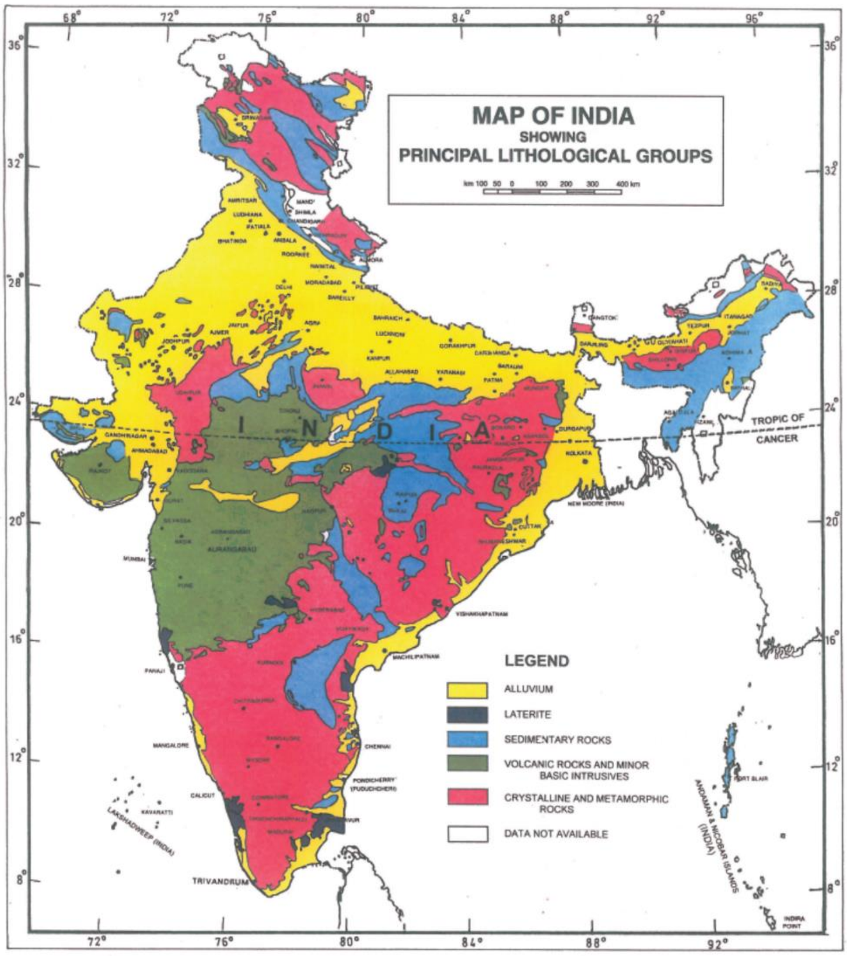

Figure 15 Map of India showing principal lithological groups

5.6.6 Seismic weight

IS 1893 specifies to consider full dead plus percentage of imposed load for estimating design seismic force. In compliance with clause 7.3.2, imposed load on roof except equipment and permanently fixed facilities need not be considered. Hence, for through roofs and cover on platforms no imposed load is assumed to contribute to the seismic weight of the structure.

However, for air concourse, where imposed load is above 3 kN/m^2, 50% of total imposed load shall be considered in calculation of seismic weight (table-10, clause 7.3.1).

5.6.7 Seismic analysis methods

In compliance with clause 7.6, equivalent static method can be employed only for regular buildings with height less than 15m in Seismic Zone II.

As per clause 7.7.1 of IS1893-2016, except regular buildings lower than 15m in Seismic Zone II, the linear dynamic analysis shall be performed to obtain the design lateral force. Hence, the response spectrum method of dynamic analysis shall be employed for all such type of structures, including foot over bridges.

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In compliance with clause 7.7.3 of IS 1893:2016, the design base shear estimated using dynamic analysis methods shall not be less than the design base shear calculated using a fundamental period depicted in Table 14 of this report.

5.6.8 Damping ratio

According to clause 7.2.4 of IS1893:2016, irrespective of the material of construction, the value of damping shall be considered as 5% of critical damping for estimating horizontal seismic coefficient A_h.

5.6.9 Vertical earthquake effect

In compliance with clause 6.3.3.1 of IS1893:2016, when the structure is

(i) Located in seismic zone IV or V or

(ii) Having vertical or plan irregularities or

(iii) Rested on soft soil or

(iv) A bridge or

(v) Having long spans or

(vi) Having large horizontal overhangs of structural members or sub-systems

The effects due to vertical earthquake shaking shall be considered.

Therefore, due to the length of span vertical effects shall be considered for the structures under assessment.

The design seismic acceleration spectral value A_v shall be calculated as

$$A_v = \frac{\left(\frac{2}{3}\right)\left(\frac{Z}{2}\right)(2.5)}{\left(\frac{R}{I}\right)} \qquad \text{(cl. 6.4.6, IS1893:2016)}$$

With reference to Table 5 (v), buildings undergo complex earthquake behaviours and hence damage, when they do not have lateral force resisting systems oriented along two plan directions that are orthogonal to each other. Hence, in compliance with clause 6.3.2.2 and 6.3.4.1 of IS1893:2016, such building systems shall be designed for earthquake load combinations listed below.

Seismic vibration in the longitudinal, transverse and vertical directions shall be combined following the rule:

$r1 \pm 0.3\ r2 \pm 0.3r3$ (IS1893:2016 cl. 6.3.4.1, IRC: SP: 114-2018, cl.4.2.2)

where r1 is the leading seismic direction and r2, r3 are the remaining seismic directions.

5.7 Aerodynamic actions from passing trains

Our structures are not sensitive to aerodynamics effects from passing trains.

5.8 Fatigue

Air concourse is considered as Foot Over Bridge in the IR policy documents.

As per Note mentioned below the clause 3.6.5 of IRS Steel Bridge Code: *"No Allowance for fatigue need be made in the design of Foot Over Bridge"*.

Therefore, the fatigue consideration is not required.

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.

3.6.2 All details shall be designed to avoid as far as possible stress concentrations likely to result in excessive reductions of the fatigue strength of members or connections. Care shall be taken to avoid a sudden reduction of the section of a member or a part of a member, especially where bending occurs.

3.6.3 Stresses due to dead load, live load and impact, stresses resulting from curvature and eccentricity of track and secondary stresses as defined in clause 3.3.2 (a) only shall be considered for effects due to fatigue. All other items mentioned in clause 3.1 and secondary stresses as defined in clause 3.3.2(b) shall be ignored when considering fatigue.

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.

Note:-
No allowance for fatigue need be made in the design of Foot over bridges.

3.6.6. **Connection riveted or bolted-** The number of rivets and bolts shall be calculated without any allowance for fatigue but rivets or bolts subjected to reversal of stress during passage of live load shall be designed for the arithmetical sum of the maximum load plus 50% of the reversed load. In the case of wind bracings, the connection shall be designed to resist the greater load only.

3.6.7. The welds shall be designed according to the permissible stresses given in IRS Welded Bridge Code.

6

Figure 16 Extract from the Steel Bridge Code

5.9 Durability

For corrosion protection of structural steel members, protective coatings to be applied as per IRS B1-2001 specification after fabrication. In-service protection and maintenance of the structure through its design life to be as per relevant provisions of Indian Railways Bridge Manual and outlined in the O&M manual, provided by the fabricator.

5.10 Fire Resistance

The requirements shall apply to steel building elements designed to exhibit a required fire resistance-level (FRL) as per relevant specifications.

For protected steel members and connections, the thickness of protection material shall be greater than or equal to that required to give a period of structural dependency (PSA) greater than the required FRL.

For unprotected steel members and connections, the exposed surface area to mass ratio shall be less than or equal to that required to give a PSA equal to the required FRL.

The fire resistance time for steel structural elements up to 6.7m above the track level shall be considered as per NBC-2016:

Columns/Struts/Bracing	2.0h
Beam/Girders/Truss	1.5h
Metal Deck	1.5h

5.11 Fabrication and erection

Fabrication of the structure to be done in accordance with the IRS B1-2001 and IRS Welded Bridge Code.

5.12 Vibration

Structure shall also be analysed for effect of vibration due to movement of passengers by conducting dynamic analysis satisfying the codal provision.

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6 Deflection limits

6.1 Steel elements

Floors (Elements not susceptible to Cracking)	: L/325
Roofs (Elements not susceptible to Cracking)	: L/300
Floors & Roofs (Elements susceptible to Cracking)	: L/360
Cantilevers (Elastic cladding)	: L/150
Steel columns: Wind/EQ lateral drift	Height/300

7 Load combinations

Load combinations for design of structure should be as per Table 18 of IS 456:2000, Table 4 of IS 800:2007 and IRS Seismic Code.

Structure shall also be checked for load combination for construction stage as given clause 7.2 (c) of IRS seismic code.

Limit State of Collapse

1.5 DL + 1.5 IL ± 1.05 TL

1.2 DL + 1.2 IL + 1.2 WL ± 0.53 TL

1.5 DL + 1.5 WL

0.9 DL + 1.5 WL

1.2 DL + 1.2 IL + 1.2 EL# ± 0.53 TL

1.5 DL + 1.5 EL#

0.9 DL + 1.5 EL#

0.9 DL + 1.5 TL

Limit State of Serviceability

1.0 DL + 1.0 IL ± 1.0 TL

1.0 DL + 1.0 IL

1.0 DL + 1.0 WL

1.0 DL + 1.0 WL

1.0 DL + 1.0 EL#

1.0 DL + 1.0 EL#

1.0 DL + 0.8 IL + 0.8 WL ± 0.8 TL

1.0 DL + 0.8 IL + 0.8 WL ± 0.8 TL

1.0 DL + 0.8 IL + 0.8 EL# ± 0.8 TL

1.0 DL + 0.8 IL + 0.8 EL# ± 0.8 TL

Where:

- DL = Dead Load;
- IL = Imposed Load;
- WL = Wind Load;
- EL = Earthquake Load;
  - $EL_X = \pm EL_X \pm 0.3EL_Z$;
  - $EL_Y = \pm EL_Y \pm 0.3EL_Z$ and;
  - $EL_Z = \pm EL_Z \pm 0.3EL_X \pm 0.3EL_Y$-
- TL = Temperature Load.