DESIGN BASIS REPORT

FOR WEST SIDE STATION

BUILDING OF

UDHANA RAILWAY STATION AT UDHANA OF MUMBAI DIVISION IN WESTERN RAILWAYS

STRUCTURE DBR

CLIENT:
WESTERN RAILWAY

PMS:
SGS INDIA PVT.

EPC CONTRACTOR:
BRIDGE & ROOF INDIA LIMITED.

EPC CONSULTANT:

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CHAPTER 2 – (Structure)

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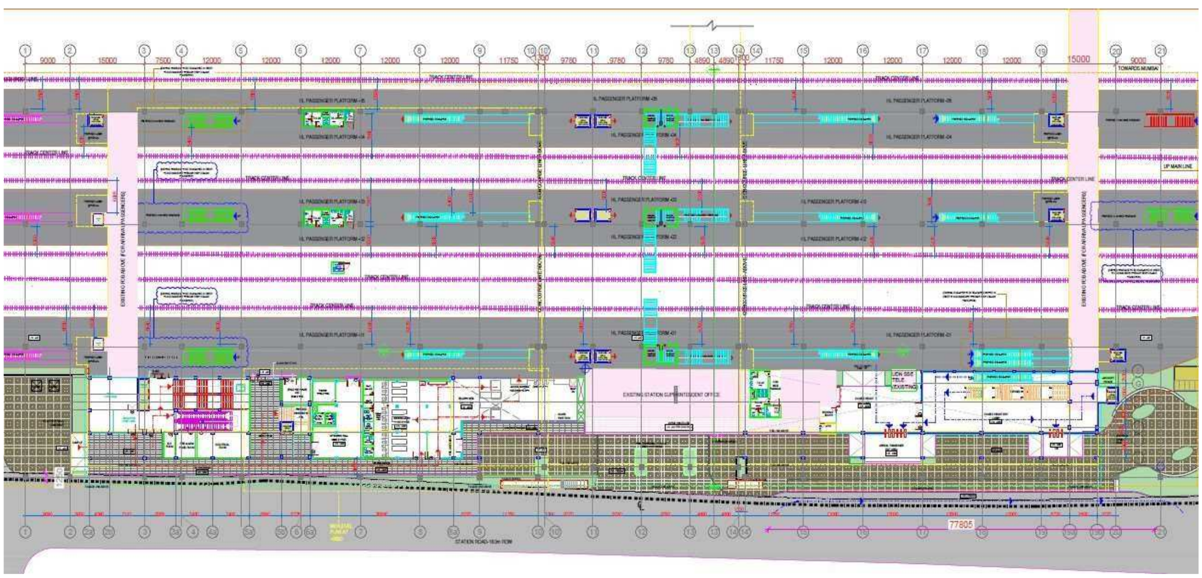
2.1 INTRODUCTION

The development of Udhana station shall play a key role in providing the first impression of the city to the visitors arriving by train from other parts of the country. The development of Udhana station shall include related amenities that will also serve non-travelers, generating revenue as well as creating a facility that will enhance the urban character unique to the city and locality. This chapter details the station facilities and amenities proposed for Udhana Railway Station. These facilities shall integrate a harmonious and elegant architectural statement with a comfortable and efficient passenger experience, ease of movement, security, safety, accessibility, and efficient connectivity to other transport lines. The station building design shall take into consideration to reflect the city's character, historical background, and community.



2.2 PURPOSE

The purpose of the present document is to detail the different assumptions of preliminary structure design i.e general, material specification/grade, loading data, load combinations, deflections/displacements and substructure considerations applicable to calculate steel/RCC structure of West side Area which includes Steel structures from Grid- 1 to 12 (upto roof) with expansion joint separating concourse area (Floor+roof) for grid-12 to 16 and 16 to 24 (upto roof). RCC structure are not connected to steel structures and designed separately.

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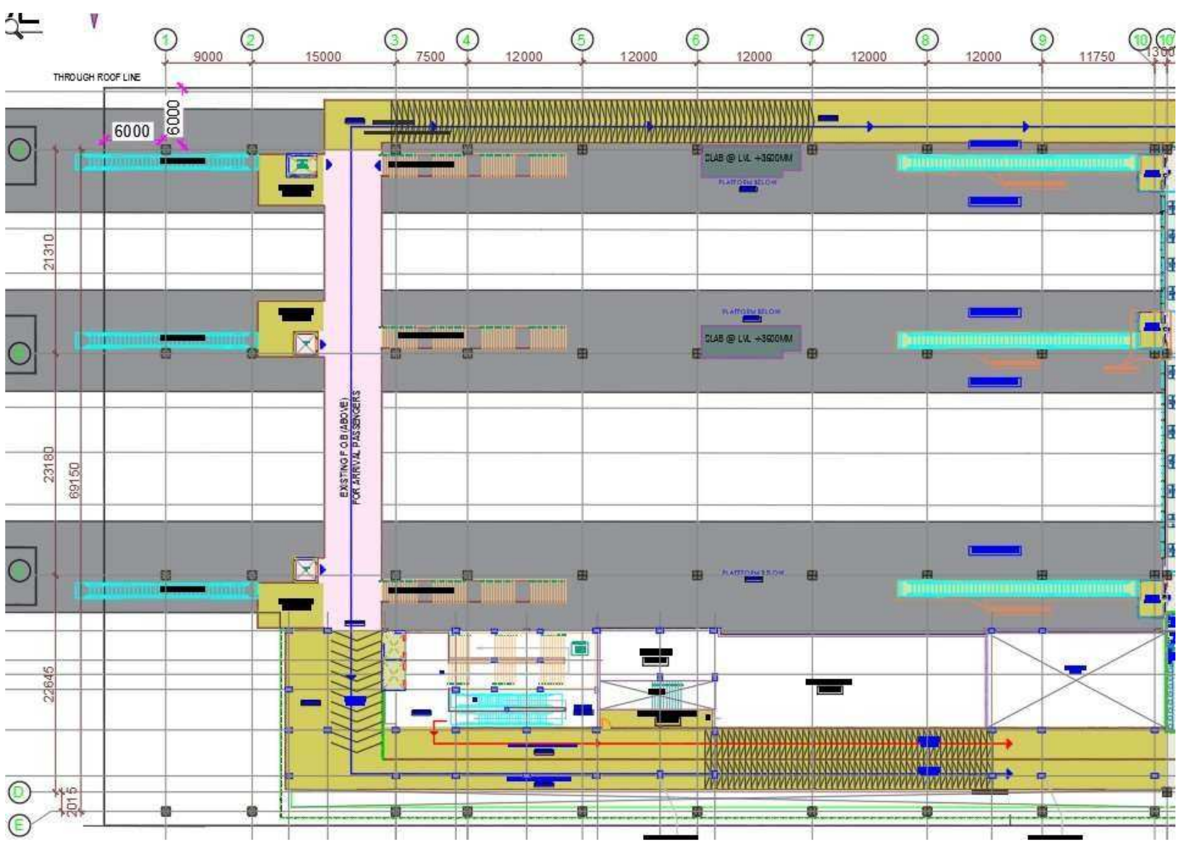
2.2.1 West Side, Air concourse, Platform, Through roof Area Structure

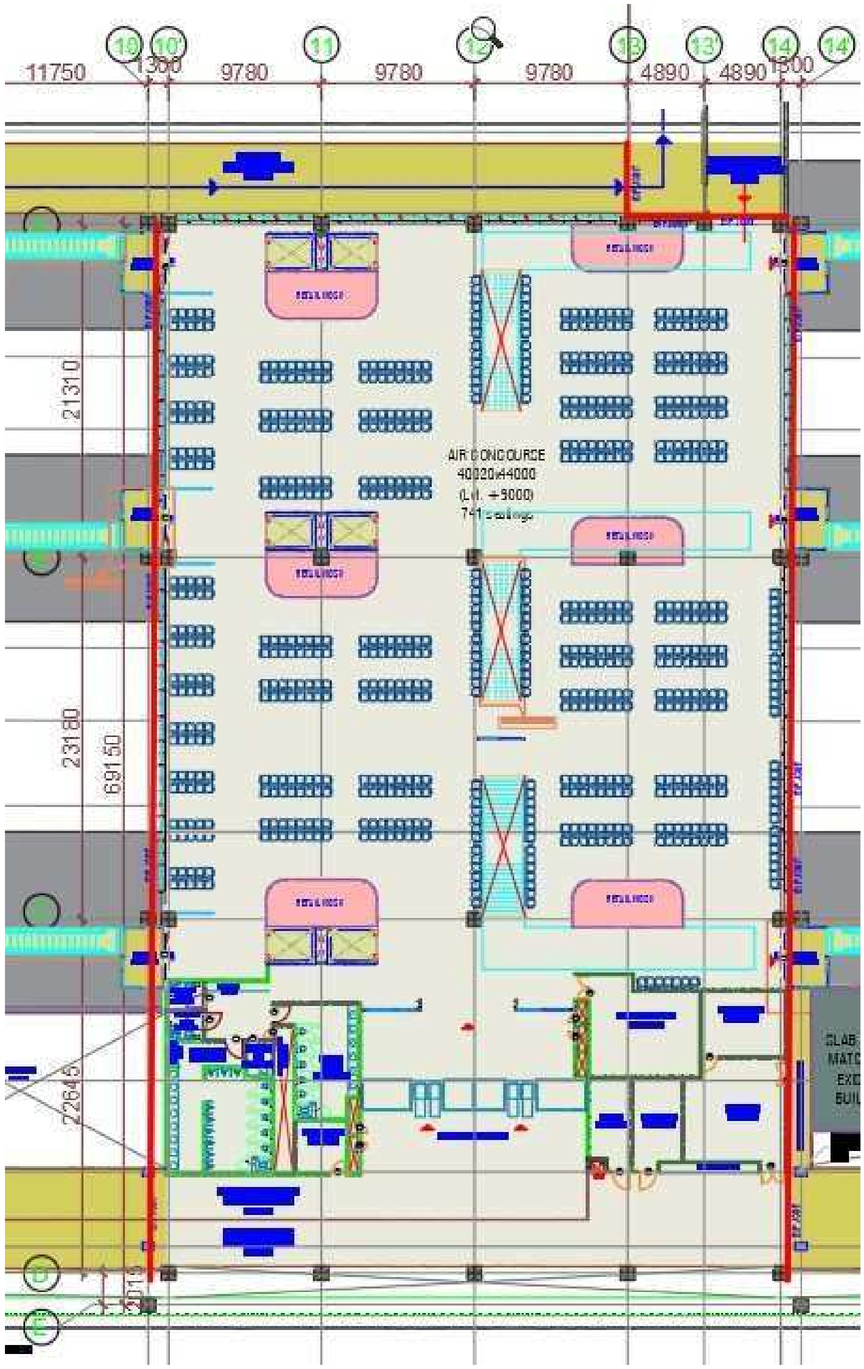
Structural parameters are such that 66.345 m width with 6m overhang on grid-A,1,21 side & 1.6 m overhang on grid-D side, 232.72 m length with expansion joint (109.05 m + 39.12 m + 84.55 m) and height as per the elevation for Steel structure. Concourse (Floor) of 66.75 mwidth, 38.82 m length & F.O.B of 10 m width, 40 m (approx.) of each span.

Steel column grid of 24 m x 12 m for through roof considered in design at grid-1 to 10 &14′ to 21.

At concourse basic grid of 9.78 m x 24 m considered in design at grid -10′ to 14. Concourse floor is designed with Slab + Steel deck sheet resting on I section steel beams. At grid -13 to 14 F.O.B girder is resting on concourse column with expansion joint through bracket and beam connection. Vertical & horizontal load of F.O.B transferred from connectedBracket & beam to concourse column.



THROUGH ROOF AT GRID - 1 to 10



CONCOURSE FLOOR + ROOF + FOB AT GRID – 10' to 14

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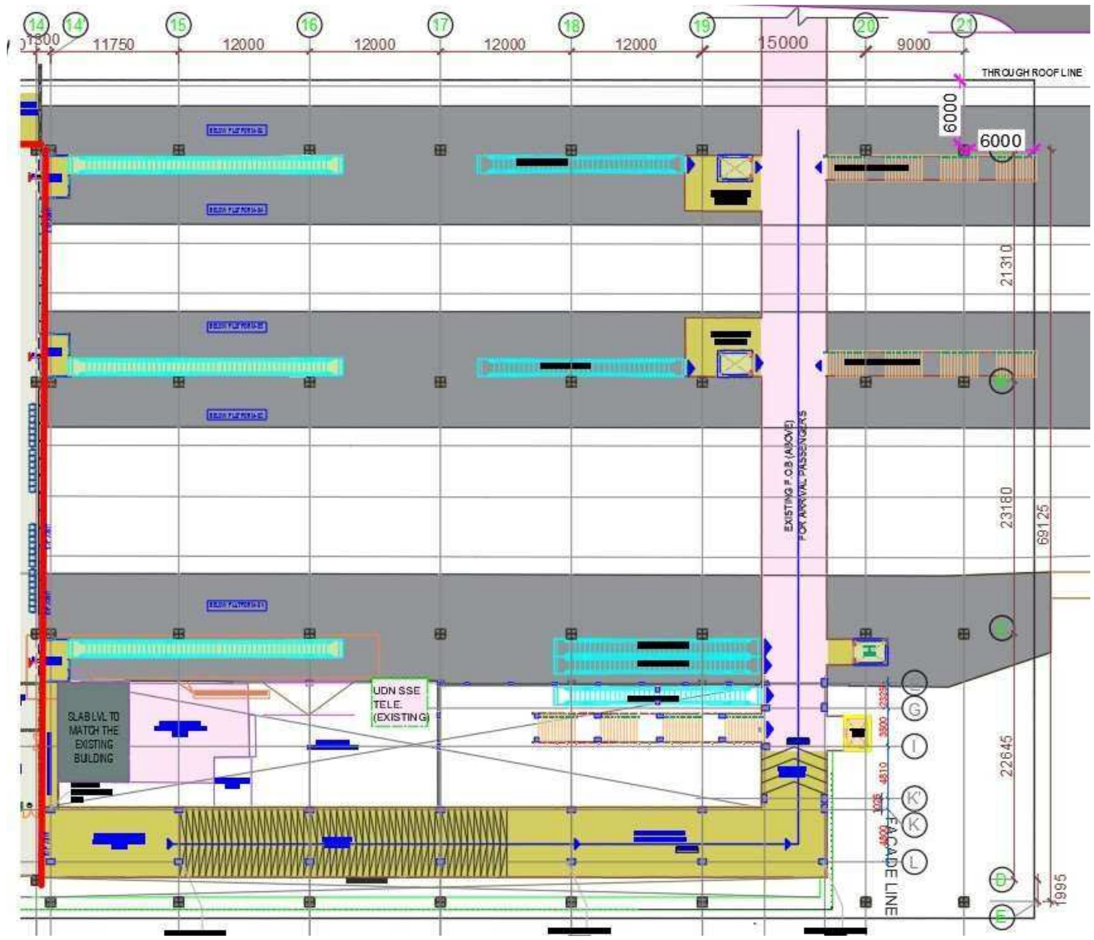

STEEL STRUCTURE UPTO ROOF AT GRID - 14 to 21

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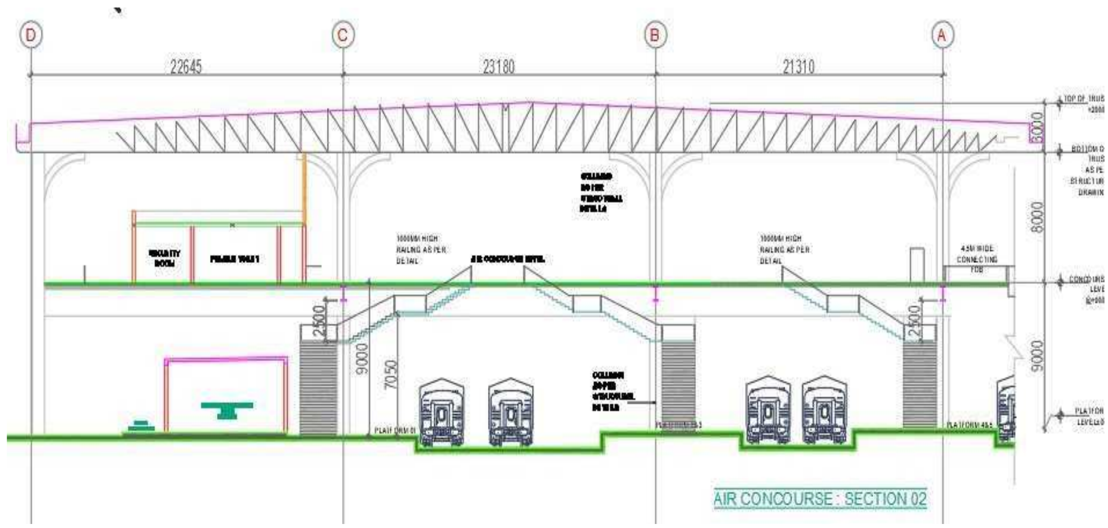

ELEVATION FOR GRID - 1 to 24

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2.3 REFERENCE DOCUMENTS (IMPORTANT INDIAN STANDARDS FOLLOWED FOR DESIGNS)

Sr. No.	Code	Title
1.	IS:269	Specs for Ordinary and Low Heat Portland Cement
2.	IS:8112	Specification for 43 Grade Ordinary Portland Cement
3.	IS:12269	Specification for 53 Grade Ordinary Portland Cement
4.	IS:455	Specifications for Portland Slag Cement
5.	IS:1489 (Part-1)	Specifications for Portland Pozzolana Cement (Fly ash based)
6.	IS:383	Specs for coarse and find aggregate from natural sourcesfor concrete
7.	IS:432 (Part-1)	Specs for Mild Steel & Medium tensile steel bars
8.	IS:456-2000	Code of Practice for plain and Reinforced Concrete
9.	IS:800-2007	Code of Practice for General Construction in Steel, Excluding chapter-12.
10.	IS:875-1987 (Parts1,2, 4 & 5) & IS:875-2015(Part-3)	Code of Practice for Design Loads (other than Earthquake)for Building and Structures
11.	IS:1080	Design and Construction of Shallow Foundations in soils (other than Raft, Ring & shell)
12.	IS:1786	Specs. For High Strength Deformed steel bars and wires for concrete reinforcement.
13.	IS:1904-1986	Design and construction of foundation in soils: General Requirements.
14.	IS:2950-1981	Design and construction of Raft Foundations
15.	IS:1905-1987	Code of Practice for Structural use of Un-reinforced Masonry
16.	IS:3370	Concrete Structures for the storage of liquids.
17.	IS:9103	Specifications for Admixtures for Concrete.
18.	IS:8009	Calculation of settlement of shallow foundations.
19.	IS:8007	Design of Concrete structures for retaining aqueous liquids.
20.	IS:8110	Structural use of concrete.
21.	IS:1893-2016	Criteria for Earth quake Resistant Design of Structures
22.	IS:4326	Code of Practice for Earth quake Resistant Design and Construction of Buildings.

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23.	IS:13920-2016	Ductile Detailing of reinforced Concrete Structuressubjected to Seismic Forces
24.	IS:2062	Steel for general structural purposes specifications
25.	IS:2950	Raft Foundation
In addition to above mentioned codes, we have followed below mentioned codes for concourse Structure above railway track.		
26.	IRS	Substructure and foundation codes- latest version
27.	IRS	Bridge rule- latest version
28.	IRS	Concrete bridge code – latest version
29.	IRS	Code of earthquake resistant design -2017
30.	IRS	Steel Bridge code

2.4 MATERIAL

2.4.1 Concrete structure

Grade of concrete used for structure are as follows: -

S.No.	Specification	Garde of Concrete (MPa)
1	Pedestal for Steel column	M30
2	Column & Shear wall	M30
3	Beam and slab	M30
4	Foundation	M30
5	Water tank & retaining wall	M30

High yield strength deformed bars with FY = 500 N/mm2 shall be used in this project. Only lapped splices shall be used.

Reinforcement bar, TMT - Thermo-mechanically treated (Fe 500 D) for Main Reinforcement bar & Stirrup Reinforcement Bar Conforming to IS: 1786 shall only to beused.

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2.4.2 Steel structure

Material used for Steel structure are as follows:-

S.No.	Specification	Garde of Steel (MPa)
1	Built-up Steel member	E350
2	Hot rolled I - section, Channel, ISA, UB, UC, HE	E350
3	Hollow section(CHS, RHS, SHS)	E355
4	Steel connection Plates	E350
5	Anchor/foundation Bolts	8.8/10.9 grade
6	Connection Bolts	8.8/10.9 grade
7	Deck-sheet	E350

2.5 COVER FOR FIRE PROTECTION.

S.No.	Specification	Cover
1	R.C.C. Column & Shear wall	40mm / 2hrs
2	R.C.C. Beam	30mm / 2hrs
3	R.C.C. Slab	25mm / 2hrs
4	R.C.C. Foundation	50mm
5	Fire treatment as per NBC for beam, column, deck sheetslab	If required

Protection against corrosion :

a) Internal structures : All steels have to be protected by painting.
b) External structures : All steels have to be corrosion protected by painting.

2.6 LOADS (CONSIDERED IN DESIGN)

2.6.1 Dead Load

UNIT/DEAD WEIGHT	
For RCC	25 kN/m^3
For Brick wall	20 kN/m^3
For Plastering	20 kN/m^3
For Floor finish	24 kN/m^3
For Structural steel	78.5 kN/m^3
For Soil	18 kN/m^3

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Self-weight of structure will be automatically considered by the software used.

TYPICAL LOADING DATA

S.No.	Specification	Loading
1	Roof sheeting load	0.13 kN/m^2
2	Services load	0.30 kN/m^2
3	Brickwork load (230 mm)	5 kN/mt/mt height
4	Railing load	0.5 kN/m height
5	Additional roof load for toilet & rooms above concourse level	5.0 kN/m^2
6	OHE load considered (air concourse main girder)	Int no- T1/IN/0047

Concourse Floor loading Data

S.No.	Specification on each floor	Loading
1	Floor finish load 75mm thk. = 0.075 x 24	1.8 kN/m^2
2	Deck R.C.C. slab load 150mm thk. = 0.150 x 25	3.75 kN/m^2
3	Services (MEP)	0.30 kN/m^2
4	Partition Load	1.50 kN/m^2
	TOTAL	7.35 kN/m^2

2.6.2 Live Load

The live load for floors and services area shall generally be in accordance with IS:875 (Part-2) unless specifically modified by users/clients otherwise. Following values of live load proposed are taken in design: -

TYPICAL LOADING DATA

S.No.	Specification	Loading
1	Roof Live load (Inaccessible)	0.75 kN/m^2
2	Roof Live load (Accessible)	1.5 kN/m^2
3	Concourse Floor	5.0 kN/m^2
3a	Concourse Floor(as per IRS)	4.8 kN/m^2
4	FOB Floor	5.0 kN/m^2
5	Ramp Live load	5.0 kN/m^2
6	Toilet's	2.0 kN/m^2
7	UPS/battery/electrical room	As per actual/min 3.0 kN/m^2

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2.6.3 Wind Loads

Whole building Wind loads will be calculated in accordance with IS 875: Part 3-2015.

Main structure + concourse, Wind load as per IS 875: Part 3-2015		
V_b , basic wind speed (ref. annex A, Page-51)	=	44 m/s
K_1 , probability factor/risk coefficient, (for 100 yearsmean probable design life), Table 1, page 7	=	1.07
K_2 , terrain roughness and height factor, as per table2, pg-8 based on terrain category & structure height	=	1.07 (Category-2, for 20 mheight)
K_3 , topography factor, as per Cl.6.3.3.1, page-8	=	1.0
K_4 , importance factor for cyclonic region, as perCl.6.3.4, page-8	=	1.3
Design wind speed $V_z = V_b*K_1*K_2*K_3*K_4$	=	44*1.07*1.07*1*1.3 = 65.488 m/s
$pz = 0.6*V_z^2$, wind pressure at height z,	=	0.6*65.488*65.488/1000 = 2.573 kN/m^2
K_a , area averaging factor, page 9/10	=	1
K_c , combination factor, page 9/16	=	1
K_d , wind directionality factor, page 9/10	=	1
p_d , The design wind pressure = $K_a*K_c*K_d*p_z$	=	1*1*1*2.573 = 2.573 kN/m^2
$p_d \geq 0.7*p_z$	=	0.7*2.573 = 1.801 kN/m^2

Hence design wind pressure will be **2.573** kN/m^2 as per design consideration.

PRESSURE COEFFICIENT FOR ROOF

From Table-9 Pressure Coefficients for Free Standing Double Sloped Roofs ,For angle = · 0.5

Cp = +0.3 , -0.9

Downward pressure = 0.3*2.573 = **0.772 kN/m^2** (·)

Upward pressure = 0.9*2.573 = **2.357 kN/m^2** (·)

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PRESSURE COEFFICIENT FOR WALL

From Table-26 Force Coefficients for Low Walls or

Hoardings,b = 215 m , h = 20 m , b/h = 10.75 , Cf = 1.2

Wind pressure for height (0-10), $p_d \times C_f$ = 2.248 x 1.2 = **2.7 kN/m²**

Wind pressure for height (10-20), $p_d \times C_f$ = 2.573 x 1.2 = **3.088 kN/m²**

2.6.4 Temperature load

As per the clause 19.5.1 (Shrinkage, Creep and Temperature Effects) of IS 456:2000,

In ordinary building, such as low-rise dwellings whose lateral dimension do not exceed 45m, the effects due to temperature fluctuations and shrinkage and creep can be ignored in design calculations.

For Steel Structures Temperature load are applied as per IS 875:1987 (Part-5), Fig.1 &Fig.2.

Highest Temperature =

+45 ^{0}CLowest

Temperature = +2.5 ^{0}C

Mean temperature = (45-2.5)/2 = 22 ^{0}C

Temperature load of 22 ^{0}C considered for RCC/Steel structure.

2.6.5 Earthquake load

As per codal provisions of IS1893: part-1 2016 following parameters are being considered for analysis.

Z, seismic zone factor given in Table -3 (Clause 6.4.2), pg.-10	=	0.16, Seismic Zone III
R, Response Reduction factor given in Table 9 (Clause 7.2.6), pg-20, Steel building with OMR	=	3
R, Response Reduction factor given in Table 9 (Clause 7.2.6), pg-20, RCC Ductile Frame	=	5
I, Importance Factor given in Table 8 (Clause 7.2.3), pg-19	=	1.5
Soil type 2		Medium (n corr =15-20)
Time period as per clause 7.6.2 (c) page-21 (For Steel structures)		$T = 0.085*h^{0.75}$
Time period as per clause 7.6.2 (c) page-21 (For RCC		$T = 0.075*h^{0.75}$

structure) without infill		
Time period as per clause 7.6.2 (c) page-21 (For RCC structure) with infill		Ta = 0.09*h/√d

As per codal provisions of IRS earthquake resistant design -2017following parameters are being considered for concourse structure above railway track analysis.

Z, seismic zone factor given in Table -1A (Clause 9.4.6),pg.-12	=	0.16, Seismic Zone III
R, Response Reduction factor given in Table 3 (Clause 9.4.5), pg-14, Steel integral frame without ductile detailing(OMRF)	=	3.3
I, Importance Factor given in Table 2 (Clause 9.4.4), pg-12 , for bridge crossing railway lines	=	1.2
Time period as per clause 7.6.2 (c) page-21 (For Steel structures)		$T = 0.085*h^{0.75}$
Time period as per clause 7.6.2 (c) page-21 (For RCC		$T = 0.075*h^{0.75}$

2.6.5.1 References for Earthquake Loads (IS 1893-1:2016)

For Important buildings following value has been used for structures as perclause mentioned below of IS1893: part-1 2016.

6.4.6 The design seismic acceleration spectral value A_v or vertical motions shall be taken as:

$$A_v = \begin{cases} \dfrac{\left(\frac{2}{3}\times\frac{Z}{2}\right)(2.5)}{\left(\frac{R}{I}\right)} & \text{For buildings governed by IS 1893 (Part 1)} \\ \dfrac{\left(\frac{2}{3}\times\frac{Z}{2}\right)(2.5)}{\left(\frac{R}{I}\right)} & \text{For liquid retaining tanks governed by IS 1893 (Part 2)} \\ \dfrac{\left(\frac{2}{3}\times\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)} & \text{For bridges governed by IS 1893 (Part 3)} \\ \dfrac{\left(\frac{2}{3}\times\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)} & \text{For industrial structures governed by IS 1893 (Part 4)} \end{cases}$$

Table 3 Seismic Zone Factor *Z*
(*Clause* 6.4.2)

Seismic Zone Factor (1)	II (2)	III (3)	IV (4)	V (5)
Z	0.10	0.16	0.24	0.36

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Table 8 Importance Factor (*I*)
(*Clause* 7.2.3)

Sl No. (1)	Structure (2)	*I* (3)
i)	Important service and community buildings or structures (for example, critical governance buildings, schools), signature buildings, monument buildings, lifeline and emergency buildings (for example, hospital buildings, telephone exchange buildings, television station buildings, radio station buildings, bus station buildings, metro rail buildings and metro rail station buildings), railway stations, airports, food storage buildings (such as warehouses), fuel station buildings, power station buildings, and fire station buildings), and large community hall buildings (for example, cinema halls, shopping malls, assembly halls and subway stations)	1.5
ii)	Residential or commercial buildings [other than those listed in Sl No. (i)] with occupancy more than 200 persons	1.2
iii)	All other buildings	1.0

Table 9 Response Reduction Factor *R* for Building Systems
(*Clause* 7.2.6)

Sl No. (1)	Lateral Load Resisting System (2)	R (3)
i)	**Moment Frame Systems**	
	a) RC buildings with ordinary moment resisting frame (OMRF) (*see* Note 1)	3.0
	b) RC buildings with special moment resisting frame (SMRF)	5.0
	c) Steel buildings with ordinary moment resisting frame (OMRF) (*see* Note 1)	3.0
	d) Steel buildings with special moment resisting frame (SMRF)	5.0
ii)	**Braced Frame Systems** (*see* Note 2)	
	a) Buildings with ordinary braced frame (OBF) having concentric braces	4.0
	b) Buildings with special braced frame (SBF) having concentric braces	4.5
	c) Buildings with special braced frame (SBF) having eccentric braces	5.0

Table 2 Importance Factor
(Clause 9.4.4)

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

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

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

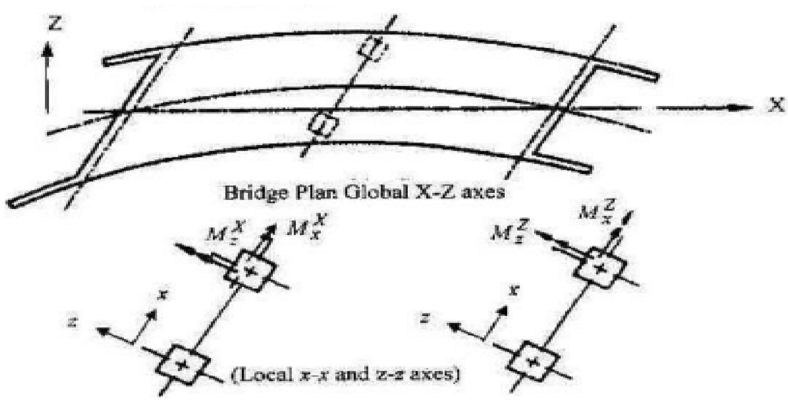
Sl. No. (1)	Structure, Component or Connection (2)	R (3)
i)	Superstructure	2
ii)	Substructure:	
	a) Reinforced concrete piers with ductile detailing cantilever type, wall type	3.0
	b) Reinforced concrete piers without ductile detailing*, cantilever type, wall type	2.5
	c) Masonry piers (un reinforced) cantilever type, wall type	1.5
	d) Reinforced concrete, framed construction in piers, with ductile detailing, columns of RCC bents, RCC single column piers	4.0
	e) Steel framed construction	2.5
	f) Steel cantilever piers	1.0
	g) Steel trussed arch	1.5
	h) Reinforced concrete arch	3.5
	k) Abutments of mass concrete and masonry	1.0
	m) R.C.C. abutment	2.5
	n) Integral frame with ductile detailing, and	4.0
	o) Integral frame without ductile detailing	3.3

2.6.5.2 Additional point for earthquake load

IRS code for earthquake resistant design 2017 is based on IS 1893 (part-3) as mentioned in first page of IRS code and also stated that unless noted otherwise , this standard shall be read necessarily in conjunction with IS1893 (part-1).

As per clause 7.2 of IRS, 30% of earthquake forces from other direction shall also be considered simultaneously with main horizontal direction. But as per IS 1893 (part-3) clause 8.3 (given in the below image). It can be concluded that when local axis of columns differs from global axis of structure only then additional 30% forces from other direction shall be considered.

this purpose, two separate analyses shall be performed for design seismic forces acting along two orthogonal horizontal directions. The design seismic force resultants (i.e. axial force, bending moments, shear forces, and torsion) at any cross-section of a bridge component resulting from the analyses in the two orthogonal horizontal directions shall be combined as shown below in the Figure



But in our case, local axis of columns match with global axis of structure, So here only one horizontal component of seismic is considered with vertical component of Earthquake. It is clearly mentioned in IS 1893 (part-1) clause no – 6.3.2.1, 6.3.4.4.and for long span vertical seismic component 0.3 factor is considered as per clause 6.3.3.

6.3.2 *Design Horizontal Earthquake Load*

6.3.2.1 When lateral load resisting elements are oriented along two mutually orthogonal horizontal directions, structure shall be designed for effects due to full design earthquake load in one horizontal direction at a time, and not in both directions simultaneously.

6.3.4.4 When components corresponding to only two ground motion components (say one horizontal and one vertical, or only two horizontal) are combined, the equations in **6.3.4.1** and **6.3.4.2** should be modified by deleting the term representing the response due to the component of motion not being considered.

So in view of this we have considered one horizontal direction with Vertical component of Earthquake as mentioned in IS code 1893.

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2.7 LOAD COMBINATIONS

All Design Load Combinations as per codal provisions conforming with IS: 800-2007. Detailed combinations & corresponding safety values has been provided in below tables. For all structures unless noted otherwise following combinations shall be used.

Load Combinations	Limit State of Collapse				Limit State of Serviceability			
	DL	LL	WL/EL	TL	DL	LL	WL/EL	TL
DL + LL	1.5	1.5	—	—	1	1	—	—
DL + LL + TL	1.5	1.5	—	1	1	1	—	1
DL + WL	1.5 or 0.9*	—	1.5	—	1	—	1	—
DL + WL +TL	1.5 or 0.9*	—	1.5	1	1	—	1	1
DL + EL	1.5 or 0.9*	—	1.5	—	1	—	1	—
DL + EL + TL	1.5 or 0.9*	—	1.5	1	1	—	1	1
DL + TL	1.5	—	—	1	1	—	—	1
DL+LL+WL	1.2	1.2	1.2	—	1	0.8	0.8	—
DL+LL+WL+TL	1.2	1.2	1.2	1	1	0.8	0.8	1
DL+LL+EL	1.2	1.2	1.2	—	1	0.8	0.8	—
DL+LL+EL+TL	1.2	1.2	1.2	1	1	0.8	0.8	1
DL+TL	1.4	—	—	1.4	1	—	—	1
DL+LL+TL	1.05	1.275	—	1.05	1	1	—	1

LEGENDS:

DL = DEAD LOAD

LL = LIVE LOAD

EL(X) =HORIZONTAL EARTHQUAKE LOAD IN X – DIRECTION

EL(Y) = VERTICAL EARTHQUAKE LOAD IN Y – DIRECTION, FACTOR OF 0.3 IS CONSIDERED DUETO LONG SPAN WITH MAIN DIRECTION EARTHQUAKE (X & Z DIRECTION)

EL(Z) = HORIZONTAL EARTHQUAKE LOAD IN Z – DIRECTION

WL- WIND LOAD

TL = TEMPERATURE LOAD

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All Design Load Combinations as per codal provisions conforming with IRS steel bridge code. For concourse steel structure above railway track following combinations shall be used.

Load Combinations	Working stress design			
	DL	LL	WL/EL	TL
DL + LL	1.0	1.0	—	—
DL + WL	1.0*	—	1.0*	—
DL + WL +TL	1.0*	—	1.0*	1.0*
DL + EL	1.0*	—	1.0*	—
DL + EL + TL	1.0*	—	1.0*	1.0*
DL+LL+WL	1.0*	1.0*	1.0*	—
DL+LL+WL+TL	1.0*	1.0*	1.0*	1.0*
DL+LL+EL	1.0*	1.0*	1.0*	—
DL+LL+EL+TL	1.0*	1.0*	1.0*	1.0*
DL+TL	1.0	—	—	1.0
DL+LL+TL	1.0	1.0	—	1.0

LEGENDS:

* = REDUCTION FACTOR DUE TO PERMISSIBLE STRESS INCREMENT AS PER IRS CODE IS APPLICABLE.

DL = DEAD LOAD

LL = LIVE LOAD

EL(X) = HORIZONTAL EARTHQUAKE LOAD IN X – DIRECTION

EL(Y) =VERTICAL EARTHQUAKE LOAD IN Y – DIRECTION, FACTOR OF 0.3 IS

CONSIDERED DUE TO LONG SPAN WITH MAIN DIRECTION EARTHQUAKE

(X& Z DIRECTION)

EL(Z) = HORIZONTAL EARTHQUAKE LOAD IN Z – DIRECTION

WL- WIND LOAD

TL = TEMPERATURE LOAD

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2.7.1 References for Load combinations

All Design Load Combinations as per codal provisions conforming with IS: 800-2007. Detailed combinations & corresponding safety values has been providedin below tables.

Table 4 Partial Safety Factors for Loads, γ_f, for Limit States
(*Clauses* 3.5.1 *and* 5.3.3)

Combination	Limit State of Strength					Limit State of Serviceability			
	DL	LL[1]		WL/EL	AL	DL	LL[1]		WL/EL
		Leading	Accompanying				Leading	Accompanying	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
DL+LL+CL	1.5	1.5	1.05	—	—	1.0	1.0	1.0	—
DL+LL+CL+	1.2	1.2	1.05	0.6	—	1.0	0.8	0.8	0.8
WL/EL	1.2	1.2	0.53	1.2					
DL+WL/EL	1.5 (0.9)[2]	—	—	1.5	—	1.0	—	—	1.0
DL+ER	1.2 (0.9)[2]	1.2	—	—	—	—	—	—	—
DL+LL+AL	1.0	0.35	0.35	—	1.0	—	—	—	—

[1] When action of different live loads is simultaneously considered, the leading live load shall be considered to be the one causing the higher load effects in the member/section.
[2] This value is to be considered when the dead load contributes to stability against overturning is critical or the dead load causes reduction in stress due to other loads.
Abbreviations:
DL = Dead load, LL = Imposed load (Live loads), WL = Wind load, CL = Crane load (Vertical/Horizontal), AL = Accidental load, ER = Erection load, EL = Earthquake load.
NOTE — The effects of actions (loads) in terms of stresses or stress resultants may be obtained from an appropriate method of analysis as in **4**.

All Design Load Combinations as per codal provisions conforming with IS: 456-2002. Detailed combinations & corresponding safety values has been provided in below tables.

Table 18 Values of Partial Safety Factor γ_f for Loads
(*Clauses* 18.2.3.1, 36.4.1 *and* B-4.3)

Load Combination	Limit State of Collapse			Limit States of Serviceability		
	DL	*IL*	*WL*	*DL*	*IL*	*WL*
(1)	(2)	(3)	(4)	(5)	(6)	(7)
DL + IL	1.5		1.0	1.0	1.0	-
DL + WL	1.5 or 0.9[1)]	-	1.5	1.0	-	1.0
DL + IL + WL	1.2			1.0	0.8	0.8

NOTES

1 While considering earthquake effects, substitute *EL* for *WL*.

2 For the limit states of serviceability, the values of γ_f given in this table are applicable for short term effects. While assessing the long term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

[1)] This value is to be considered when stability against overturning or stress reversal is critical.

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2.8 DEFLECTIONS

2.8.1 Steel structure Deflection limit as per (IS 800:2007)

Table 6 Deflection Limits

Type of Building (1)	Deflection (2)	Design Load (3)	Member (4)	Supporting (5)	Maximum Deflection (6)
Industrial Buildings	Vertical	Live load/ Wind load	Purlins and Girts	Elastic cladding	Span/150
				Brittle cladding	Span/180
		Live load	Simple span	Elastic cladding	Span/240
				Brittle cladding	Span/300
		Live load	Cantilever span	Elastic cladding	Span/120
				Brittle cladding	Span/150
		Live load/ Wind load	Rafter supporting	Profiled Metal Sheeting	Span/180
				Plastered Sheeting	Span/240
		Crane load (Manual operation)	Gantry	Crane	Span/500
		Crane load (Electric operation up to 50 t)	Gantry	Crane	Span/750
		Crane load (Electric operation over 50 t)	Gantry	Crane	Span/1 000
	Lateral	No cranes	Column	Elastic cladding	Height/150
				Masonry/Brittle cladding	Height/240
		Crane + wind	Gantry (lateral)	Crane (absolute)	Span/400
				Relative displacement between rails supporting crane	10 mm
		Crane+ wind	Column/frame	Gantry (Elastic cladding; pendent operated)	Height/200
				Gantry (Brittle cladding; cab operated)	Height/400
Other Buildings	Vertical	Live load	Floor and Roof	Elements not susceptible to cracking	Span/300
				Elements susceptible to cracking	Span/360
		Live load	Cantilever	Elements not susceptible to cracking	Span/150
				Elements susceptible to cracking	Span/180
	Lateral	Wind	Building	Elastic cladding	Height/300
				Brittle cladding	Height/500
		Wind	Inter storey drift	—	Storey height/300

Deflection	Sr. No.	Structural Member	Deflection Limits
Vertical Deflection	1	Through Roof rafter	Span / 180
	2	Roof Purlins	Span / 150
	3	Wall Girts	Span / 150
	4	Concourse/FOB Floor Beam	Span / 300
Lateral Deflection	1	Through Roof column/FOB column	Height / 150
	2	Concourse column	Height /240

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2.8.2 Concrete structure Deflection limit as per (IS 456:2002)

23.2 Control of Deflection

The deflection of a structure or part thereof shall not adversely affect the appearance or efficiency of the structure or finishes or partitions. The deflection shall generally be limited to the following:

a) The final deflection due to all loads including the effects of temperature, creep and shrinkage and measured from the as-cast level of the supports of floors, roofs and all other horizontal members, should not normally exceed span/250.

b) The deflection including the effects of temperature, creep and shrinkage occurring after erection of partitions and the application of finishes should not normally exceed span/350 or 20 mm whichever is less.

23.2.1 The vertical deflection limits may generally be assumed to be satisfied provided that the span to depth ratios are not greater than the values obtained as below:

a) Basic values of span to effective depth ratios for spans up to 10 m:

Cantilever	7
Simply supported	20
Continuous	26

b) For spans above 10 m, the values in (a) may be multiplied by 10/span in metres, except for cantilever in which case deflection calculations should be made.

c) Depending on the area and the stress of steel for tension reinforcement, the values in (a) or (b) shall be modified by multiplying with the modification factor obtained as per Fig. 4.

d) Depending on the area of compression reinforcement, the value of span to depth ratio be further modified by multiplying with the modification factor obtained as per Fig. 5.

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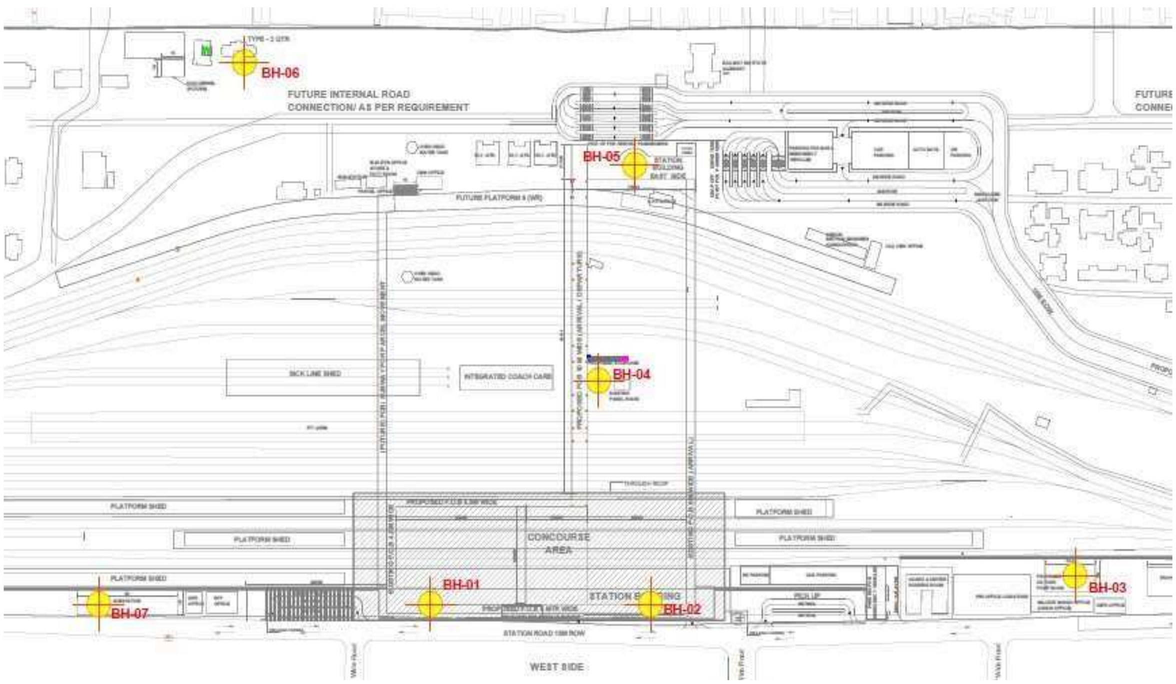
2.9 FOUNDATION

As per geo technical investigation report no. **2223073 (R3)** dated-30.08.22



BORE HOLE LOCATION PLAN

ALLOWABLE BEARING CAPACITY CALCULATIONS: -

1. For BH-01 & 03

Results of Bearing Capacity

Sl	Width	Depth	ABC @ 75 mm settlement	ABC @ 40 mm settlement	Settlement
No	(m)	(m)	T/m²	T/m²	(mm)
1	2.00	3.00	22.6	22.6	35.7
2	2.00	4.00	24.3	24.3	35.7
3	2.00	5.00	26.1	26.1	36.2
4	2.50	3.00	21.5	19.7	43.6
5	2.50	4.00	22.9	21.5	42.6
6	2.50	5.00	24.3	22.9	42.5
7	3.00	3.00	20.9	16.0	52.1
8	3.00	4.00	22.0	17.5	50.2
9	3.00	5.00	23.2	18.8	49.3
10	3.50	3.00	20.3	13.4	60.6
11	3.50	4.00	21.4	14.8	57.7

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Results of Bearing Capacity

Sl	Width	Depth	ABC @ 75 mm settlement	ABC @ 40 mm settlement	Settlement
No	(m)	(m)	T/m²	T/m²	(mm)
12	3.50	5.00	22.3	16.0	55.8
13	4.00	3.00	20.0	11.6	69.2
14	4.00	4.00	20.9	12.7	65.7
15	4.00	5.00	21.7	13.7	63.2
16	4.50	3.00	19.1	10.2	75.0
17	5.00	3.00	17.0	9.1	74.7

2. For BH-02 & 05

Results of Bearing Capacity

Sl	Width	Depth	ABC @ 75 mm settlement	ABC @ 40 mm settlement	Settlement
No	(m)	(m)	T/m²	T/m²	(mm)
1	2.00	3.00	22.3	22.3	35.3
2	2.00	4.00	24.0	24.0	35.2
3	2.00	5.00	25.7	25.7	35.6
4	2.50	3.00	21.3	19.8	43.1
5	2.50	4.00	22.6	21.5	42.1
6	2.50	5.00	24.0	22.9	42.0
7	3.00	3.00	20.6	16.0	51.4
8	3.00	4.00	21.7	17.5	49.5
9	3.00	5.00	22.9	18.8	48.6
10	3.50	3.00	20.1	13.4	60.0
11	3.50	4.00	21.1	14.8	57.1
12	3.50	5.00	22.1	16.0	55.3
13	4.00	3.00	19.7	11.6	68.2
14	4.00	4.00	20.6	12.7	64.8
15	4.00	5.00	21.4	13.7	62.4
16	4.50	3.00	19.1	10.2	75.0
17	5.00	3.00	17.0	9.1	74.7

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3. For BH-04

Results of Bearing Capacity

Sl	Width	Depth	ABC @ 75 mm settlement	ABC @ 40 mm settlement	Settlement
No	(m)	(m)	T/m²	T/m²	(mm)
1	2.00	3.00	21.7	21.7	38.8
2	2.00	4.00	23.4	23.4	38.8
3	2.00	5.00	25.1	25.1	39.3
4	2.50	3.00	20.7	17.5	47.3
5	2.50	4.00	22.1	19.1	46.2
6	2.50	5.00	23.4	20.3	46.2
7	3.00	3.00	20.1	14.2	56.6
8	3.00	4.00	21.2	15.5	54.6
9	3.00	5.00	22.3	16.7	53.5
10	3.50	3.00	19.6	11.9	66.0
11	3.50	4.00	20.5	13.1	62.6
12	3.50	5.00	21.5	14.2	60.7
13	4.00	3.00	19.2	10.2	75.0
14	4.00	4.00	20.1	11.3	71.3
15	4.00	5.00	20.9	12.2	68.7
16	4.50	3.00	16.9	9.0	74.9
17	5.00	3.00	15.1	8.1	74.9

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4. For BH-06

Results of Bearing Capacity

SI	Width	Depth	ABC @ 75 mm settlement	ABC @ 40 mm settlement	Settlement
No	(m)	(m)	T/m²	T/m²	(mm)
1	2.00	3.00	21.4	20.9	41.0
2	2.00	4.00	23.1	22.5	41.1
3	2.00	5.00	24.7	23.8	41.5
4	2.50	3.00	20.4	16.3	50.1
5	2.50	4.00	21.8	17.7	49.2
6	2.50	5.00	23.1	18.9	49.0
7	3.00	3.00	19.8	13.2	59.9
8	3.00	4.00	20.9	14.5	57.8
9	3.00	5.00	22.0	15.5	56.7
10	3.50	3.00	19.3	11.1	69.8
11	3.50	4.00	20.3	12.2	66.6
12	3.50	5.00	21.2	13.2	64.3
13	4.00	3.00	17.8	9.5	74.7
14	4.00	4.00	19.6	10.5	74.7
15	4.00	5.00	20.6	11.3	72.8
16	4.50	3.00	15.7	8.4	74.7
17	5.00	3.00	14.0	7.5	74.6

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5. For BH-07

Results of Bearing Capacity

Sl	Width	Depth	ABC @ 75 mm settlement	ABC @ 40 mm settlement	Settlement
No	(m)	(m)	T/m²	T/m²	(mm)
1	2.00	3.00	23.2	23.2	32.7
2	2.00	4.00	25.0	25.0	32.6
3	2.00	5.00	26.7	26.7	33.0
4	2.50	3.00	22.1	22.1	39.9
5	2.50	4.00	23.5	23.5	39.0
6	2.50	5.00	25.0	25.0	38.8
7	3.00	3.00	21.4	18.0	47.6
8	3.00	4.00	22.6	19.7	45.9
9	3.00	5.00	23.8	21.2	45.0
10	3.50	3.00	20.9	15.0	55.6
11	3.50	4.00	21.9	16.6	52.8
12	3.50	5.00	22.9	18.0	51.0
13	4.00	3.00	20.5	13.0	63.2
14	4.00	4.00	21.4	14.3	60.0
15	4.00	5.00	22.3	15.4	57.9
16	4.50	3.00	20.2	11.4	70.6
17	5.00	3.00	19.1	10.2	74.8

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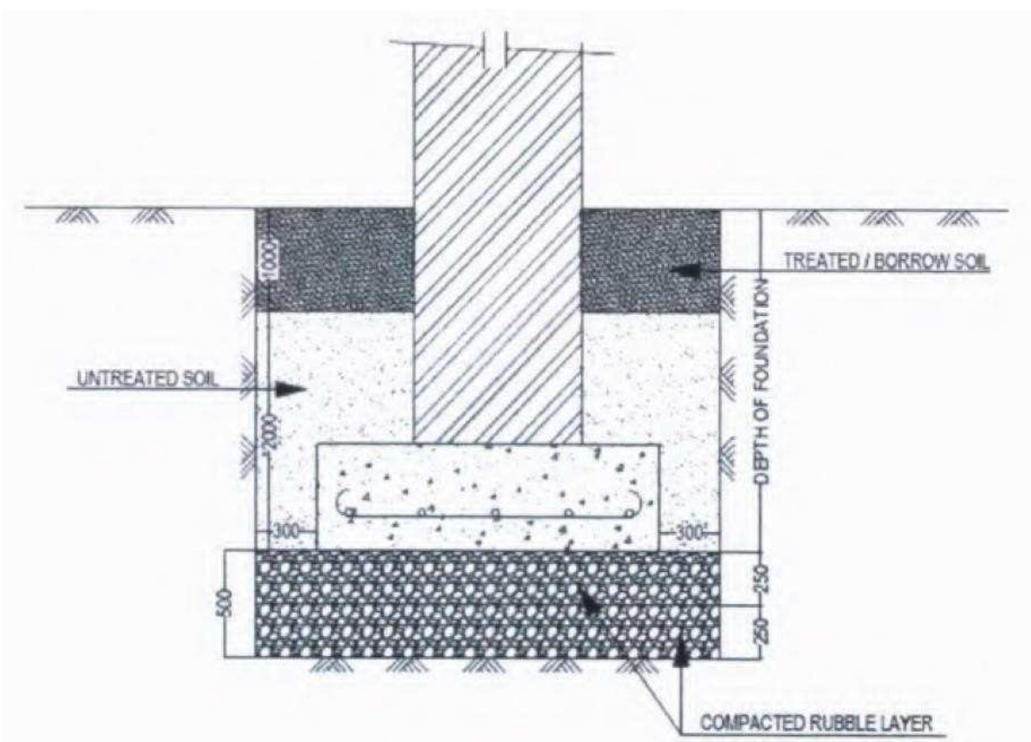
2.9.1 FOLLOWING ARE THE RECOMMENDATIONS: -

1. 1N corrected value is 15-20 (Soil will be medium soil) at 4 m depth.
2. Water table was encountered at average depth of 3 m - 5 m from NGL.
3. For West Side Building, SBC of 12.7 T/m2 at the depth of 4 m is considered for foundation design.
4. For F.O.B, SBC of 11.3 T/m2 at the depth of 4 m is considered for foundation design.
5. Foundation resting on soil having high swelling potential (upto 3 m depth), the pit shall be excavated up to foundation level + 500 mm below existing ground level and shall be backfilled upto 500 mm depth with rubble in two layers. Voids betweenthe rubble shall be filed with sand/store dust. Extent of the rubble layer shall be 300mm extra on all sides of the foundation.



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2.10 VIBRATION IMPACT ASSESSMENT

2.10.1 Description of Train Induced Emission

Nature of Ground Borne Vibration

The train-induced vibration and noise is a complex phenomenon which are dependent on several factors including track and train details, propagation paths in ground and structure, building typology, structural configuration, and material inherent damping at receiver's end. The vibration sources of the railway concern the whole train–track–foundation system. It has been found that train and track damping measures, such as dampers on the wheels and rails, elastic clips, the physical properties as well as distribution of soil and rock layers, etc., can have a significant impact on the vibration levels and frequency characteristics. A train moving along a track generates vibration in both the wheels and the track. The vibration of the wheel depends on the system above the wheel, i.e. the bogie and its springs and dampers, as well as the load of the vehicle whereas the vibration of the rail depends on the system below the rail, i.e. the track, the subsoil, and the soil. Since neither the wheel nor rail surfaces are perfectly smooth, the train wheel in effect runs across a series of "peaks" and "troughs" and thus is forced to move in a vertical direction. The track is not entirely stiff and so also moves vertically and in turn, this excites the rail pad and sleeper. In addition, the rail may be supported at discrete points (the sleepers) whereas it can vibrate freely between these fixation points. The sleepers in turn are held in place by the ballast bed. However, the subsoil under the ballast bed is often composed of different layers (inhomogeneous) and so the elasticity of the soil may vary along the track. Thus, the vehicle and track together with the track substructure and the soil interact with each other and vibrate in many different resonant frequencies. The reason for the individual elements (wheel, rail, sleeper, soil, etc.) depends on the overall connected system.

In Udhana Railway Station, the measures taken are following.

1) We had incorporated a soil cushion (sand) packed around the foundation. The soil around the foundation dampens the excess vibration levels.
2) To measure the vibration due to seismic forces in Concourse, a dynamic analysis is performed in Staad and as per analysis the frequency in 1st mode of vibration is 1.08 Cycle/Second as per below table:

CALCULATED FREQUENCIES AS PER STAAD ANLYSIS		
Mode	Frequency (Cycles/Sec)Hz	Period(Sec)
1	1.08	0.926

As per vibration assessment report received, the frequency of ground motion due to moving trains is greater than 100Hz.

Swiss (SN 640 312a) (Continuous Source	FREQUENCY (Hz)	0-10	10-30	30-60
	I. Buildings which are especially sensitive or worthy of protection.	not specified	3	3-5 mm/s

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of Vibration from Railway orHighway Traffic, Constructional equipment) (Structural damage criteria)	II. Buildings with foundations and basement floors of concrete construction, brick workwalls.		mm/s	
		not specified	5 mm/s	5-8 mm/s
	III. Building with foundation walls and floors inconcrete with walls in masonry (brick work, stonework)	Not specified	8 mm/s	8-12 mm/s
FTA (U.S.) (Continuous Source of Vibration)	Reinforced-concrete, steel, or timber (no plaster)	12.7 mm/s		
		Full frequency range		

As the calculated frequency of the structure is significantly away from frequency of ground motion due to moving trains as measured by the agency, there is no resonance condition anticipated. In long term, the designed structure is safe.

3 As per Journal namely "Soil Dynamics and Earthquake Engineering Volume 163, December 2022, 107553"

"The effect of the water table level on the railway-induced vibrations has been partially investigated in the literature. For instance, Bayındır and his colleagues studied the effect of the water table on the ground vibration through analyses of ground-borne vibrations of an at-grade high-speed railway system in Turkey [33]. From the experimental measurements, they found that in saturated soil, the vibrations are 35% less than dry soil. Similar results were presented and discussed by Schendels et al. [34] and Feng et al. [35]."

In our project, Water table is at foundation level, so it will also reduce vibration as per above literature.

2.10.2 Vibration Induced by Pedestrian

As per clause 20.1 of IRC: SP:56-2011, for span of less than 30m, structure are less susceptible to such vibration and therefore, checking on this account is not necessary. So, our structure is safe from the point of view of vibration induced by Pedestrian.

2.10.3 Vibration due to Acoustics

Construction material shall be used to take care of vibration due to acoustics.

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