DESIGN CONSIDERATION FOR AIRPORT BUILDING IN HIGH SEISMIC ZONE

Anamika Kadam
Deputy General Manager (Civil)
Larsen & Toubro

INTRODUCTION

The air travel market of India is one of the fastest growing in the world. With air passenger traffic expected to rise significantly and almost treble the numbers by in the next 20 years, the airlines are expanding their fleet size to meet the current and the projected traffic demand. This has resulted in the need to simultaneously expand and upgrade the airport infrastructure.

Fig. 1 Typical Airport Building

With such massive infrastructures being developed, it becomes equally important to ensure that these structures are also able to stand tall and unaffected within the most adverse natural hazards, that include earthquakes. In India, we are sitting in a seismically active zone, hence, adequate attention in this regard must be given while designing important service or community buildings like airports.

As per the current earthquake zonation in IS 1893-2016, the country's landmass is divided into 4 such zones namely Zone II to Zone V, depending on the severity of a possible earthquake basis historical data and extensive analysis by experts. The intent of this article is to provide a holistic view of the design approach that was adopted for the design of a lifeline structure like airport building located in high earthquake zone (e.g., Zone IV). Based on geotechnical investigation, the soil properties were evaluated and found to match the requirement of Type B (medium or stiff) soils in accordance with Table 1 of IS 1893.

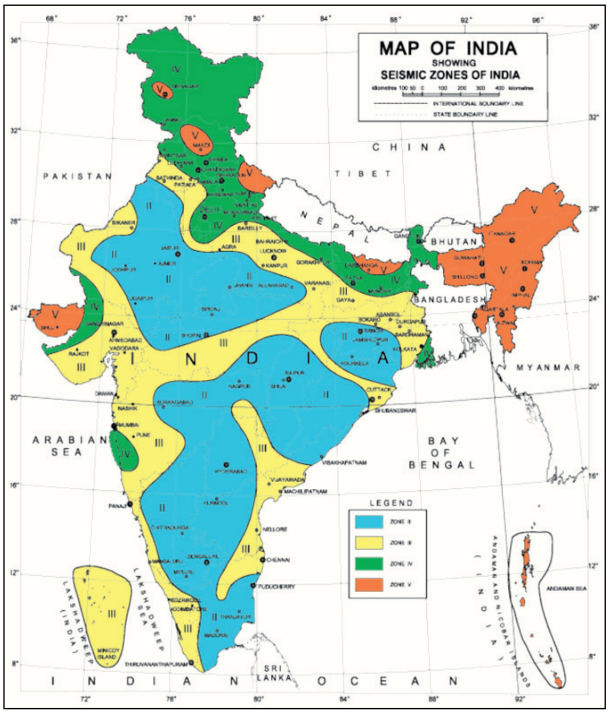


Fig. 2 Seismic Zones of India (Source IS 1893:2016)

DESIGN FOR STRUCTURAL MEMBERS –

To start with, the recommendations of the following codes and standards were strictly adhered to for design purposes–

- National Building Code of India 2016
- IS 875 for calculation design loads for buildings and structures
- IS 1893:2016 for determining the criteria of earthquake resistant design of structures
- IS 456:200 for design of reinforced concrete structures
- IS 13920:2016 for ductile design and detailing of reinforced concrete structures
- IS 800:2007 for general construction in steel.

The fundamental equation for calculation of horizontal acceleration coefficient was as per Cl. 6.4.2 of IS 1893 as follows –

$$A_h = \frac{(Z/2)\,(S_a/g)}{(R/I)}$$

The zone factor (Z) for zone IV was considered as 0.24 in line with Table 2 of IS 1893. For important buildings like airport the importance factor to be considered is 1.5 as per Table 3 if IS 1893. For ancillary buildings it was considered as 1.0. This has been resonated in Table 47 of NBC 2016 Volume 1. The response reduction factor for steel and RCC structures were taken as 5 in accordance with Table 1 of IS 1893.

Table 1. Response reduction factor (R) (Source IS 1893)

LATERAL LOAD RESISTING SYSTEM		R
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

Table 2. Seismic Zone Factor (Z) (Source IS 1893)

SEISMIC ZONE FACTOR	II	III	IV	V
Z	0.10	0.16	0.24	0.36

Table 3. Importance Factor for buildings (I) (Source IS 1893)

SI NO.	STRUCTURE	I
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 SI No. (i)] with occupancy more than 200 persons	1.2
iii)	All other buildings	1.0

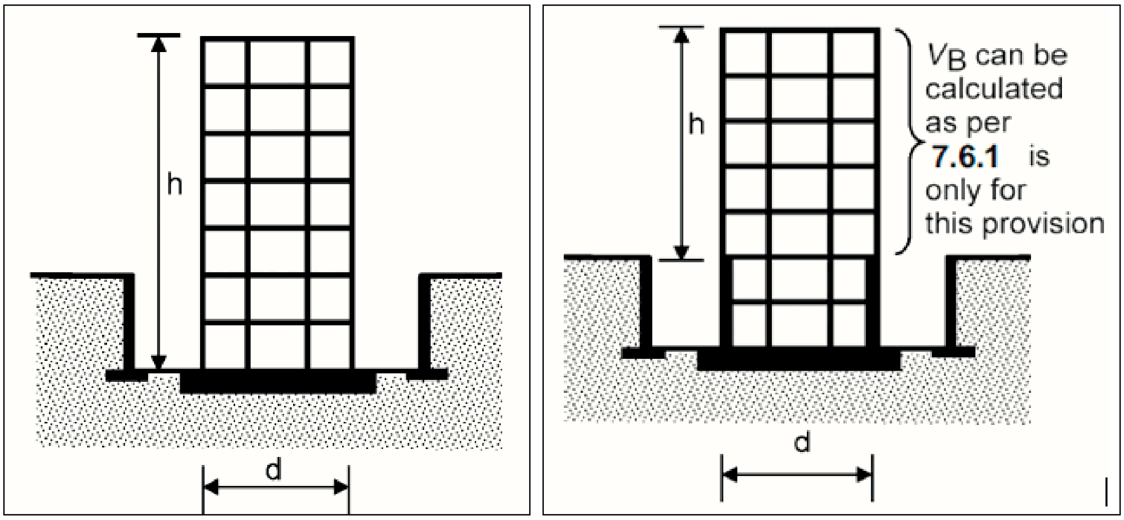
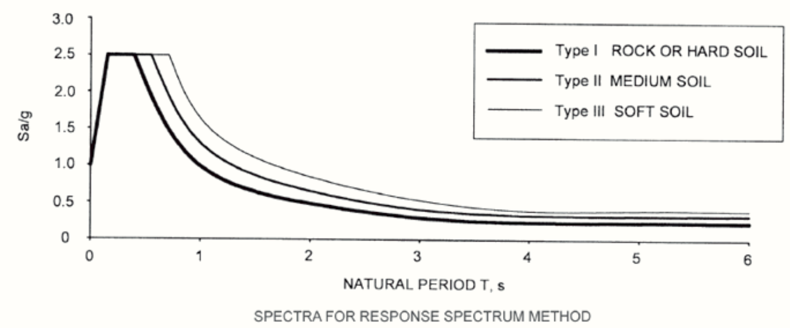


Fig. 3 Height of building for calculation of time period (Source IS 1893)



$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 1+15T & T<0.10\ \text{s} \\ 2.5 & 0.10\ \text{s}<T<0.40\ \text{s} \\ \frac{1}{T} & 0.40\ \text{s}<T<4.00\ \text{s} \\ 0.25 & T>4.00\ \text{s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 1+15T & T<0.10\ \text{s} \\ 2.5 & 0.10\ \text{s}<T<0.55\ \text{s} \\ \frac{1.36}{T} & 0.55\ \text{s}<T<4.00\ \text{s} \\ 0.34 & T>4.00\ \text{s} \end{cases} \\ \text{For soft soil sites} & \begin{cases} 1+15T & T<0.10\ \text{s} \\ 2.5 & 0.10\ \text{s}<T<0.67\ \text{s} \\ \frac{1.67}{T} & 0.67\ \text{s}<T<4.00\ \text{s} \\ 0.42 & T>4.00\ \text{s} \end{cases} \end{cases}$$

Fig. 4 Design spectral coefficient for different soil types, corresponding to natural period of structure for response spectra method for 5% damping (Source IS 1893)

For calculations of fundamental time period of the structure were calculated as follows according to Cl. 7.6.2 of IS 1893 –

1). For RCC structure without masonry infill,
$T = 0.075\, h^{0.75}$

2). For RCC structure with masonry infill,
$T = 0.09h/\sqrt{d}$

3). For steel structure,
$T = 0.085\, h^{0.75}$

Where h = height of the building (in meters) and d = base dimension of the building at the plinth level along the considered direction of earthquake.

The vertical acceleration coefficient was calculated as per Cl. 6.4.6 of IS 1893 as follows –

$$A_h = \frac{(2/3 * Z/2)\,(2.5)}{(R/I)}$$

Table 4. Values of horizontal acceleration corresponding to time period for medium/stiff soil

PERIOD(S)	SA/G	ACC.	PERIOD(S)	SA/G	ACC.
0	1.00	0.353	2.00	0.68	0.240
0.10	2.50	0.883	2.10	0.65	0.229
0.20	2.50	0.883	2.20	0.62	0.218
0.30	2.50	0.883	2.30	0.59	0.209
0.40	2.50	0.883	2.40	0.57	0.200
0.50	2.50	0.883	2.50	0.54	0.192
0.55	2.50	0.883	2.60	0.52	0.185
0.67	2.03	0.717	2.70	0.50	0.178
0.70	1.94	0.686	2.80	0.49	0.172
0.80	1.70	0.600	2.90	0.47	0.166
0.90	1.51	0.534	3.00	0.45	0.160
1.00	1.36	0.480	3.10	0.44	0.155
1.10	1.24	0.437	3.20	0.43	0.150
1.20	1.13	0.400	3.30	0.41	0.146
1.30	1.05	0.369	3.40	0.40	0.141
1.40	0.97	0.343	3.50	0.39	0.137
1.50	0.91	0.320	3.60	0.38	0.133
1.60	0.85	0.300	3.70	0.37	0.130
1.70	0.80	0.283	3.80	0.36	0.126
1.80	0.76	0.267	3.90	0.35	0.123
1.90	0.72	0.253	4.00	0.34	0.120

Table 5. Values of vertical acceleration corresponding to time period for medium/ stiff soil		
PERIOD(S)	SALG	ACC.
0	2.50	0.589
0.50	2.50	0.589
1.00	2.50	0.589
1.50	2.50	0.589
2.00	2.50	0.589
2.50	2.50	0.589
3.00	2.50	0.589
3.50	2.50	0.589
4.00	2.50	0.589
4.50	2.50	0.589
5.00	2.50	0.589
5.50	2.50	0.589
6.00	2.50	0.589

The structure was designed taking into account three directional earthquake shaking in accordance with Cl. 6.3.4.1 of IS 1893, using the assumption that when the maximum response from one component occurs, the response of the other two components are 30 percent each of their maximum.

(i) $\pm EL_X \pm 0.3EL_Y \pm 0.3EL_Z$

(ii) $\pm EL_Y \pm 0.3EL_X \pm 0.3EL_Z$

(iii) $\pm EL_Z \pm 0.3EL_X \pm 0.3EL_Y$

The final load combinations were considered as follows –

(1) $1.2[DL + IL \pm (EL_X \pm 0.3EL_Y \pm 0.3EL_Z)]$

$1.2[DL + IL \pm (EL_Y \pm 0.3EL_X \pm 0.3EL_Z)]$

(2) $1.5[DL \pm (EL_X \pm 0.3EL_Y \pm 0.3EL_Z)]$

$1.5[DL \pm (EL_Y \pm 0.3EL_X \pm 0.3EL_Z)]$

(3) $0.9DL \pm 1.5(EL_X \pm 0.3EL_Y \pm 0.3EL_Z)$

$0.9DL \pm 1.5(EL_Y \pm 0.3EL_X \pm 0.3EL_Z)$

Dynamic 3D analysis was done for all the structures using finite element method. Response spectrum approach was used for the purpose.

The RCC detailing was done as per IS 456 and IS 13920. Ductile detailing was adopted for all RCC beams, columns, and walls.

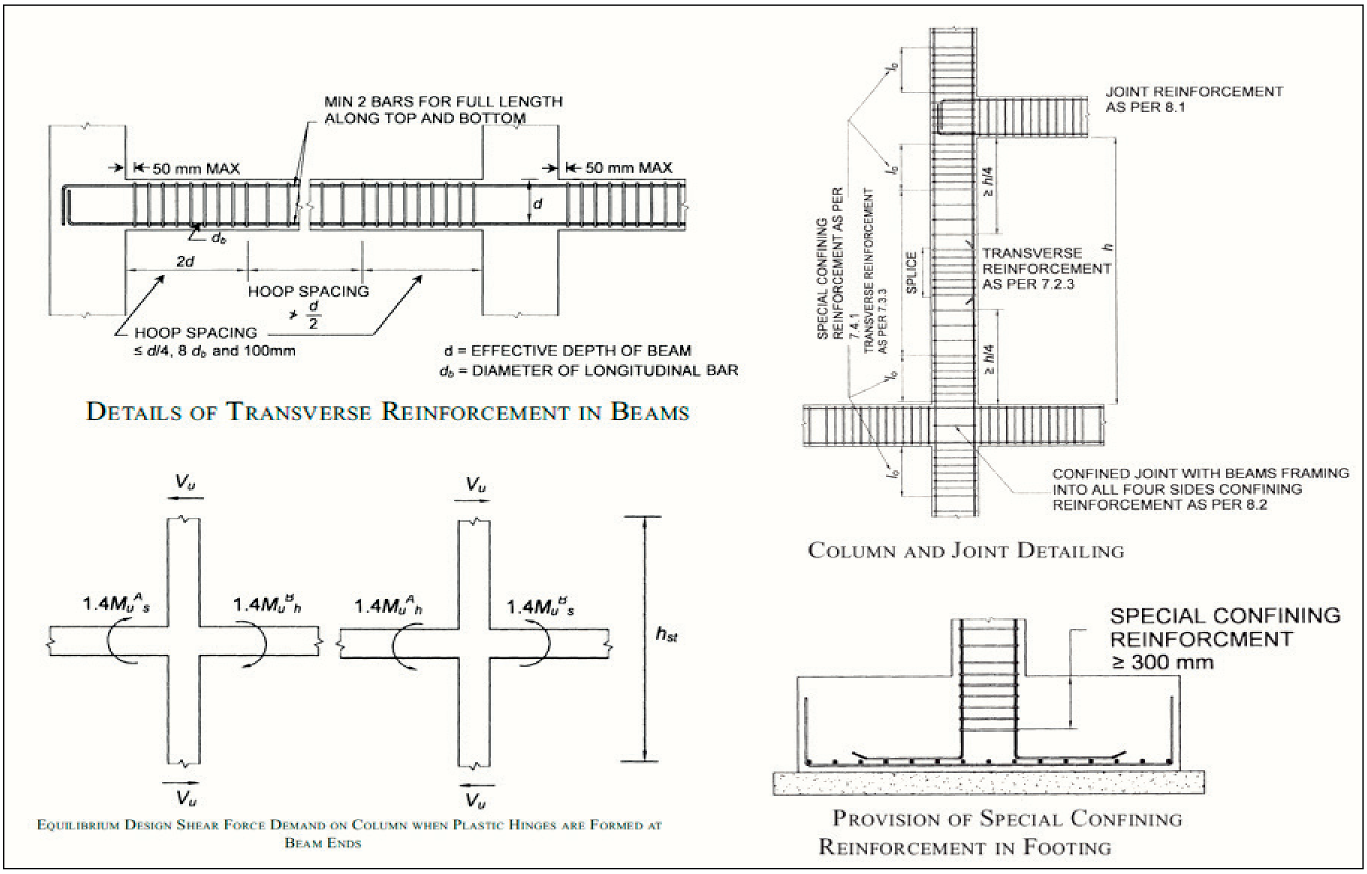


Fig. 5 Reinforcement detail in structural members (Source IS 13920)

For steel roof truss, normal connection details were adopted as per IS 800. For steel floor (e.g., mezzanines), ductile detailing as per section 12 of IS 800 was taken as the basis. Additional load combinations in accordance with section 12 of IS 800 were considered as follows –

(1) $1.2DL + 0.5LL + 2.5EL$

(2) $0.9DL + 2.5EL$

The sections selected for the beams and columns were checked to satisfy the following requirement as per IS 800 –

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.2$$

Where $\sum M_{pc}$ is the sum of the moment capacity of the column above and below the beam centerline and $\sum M_{pb}$ is the sum of the moment capacities of the beams at the intersection of beam and column intersection.

The individual thickness of the column webs and doubler plates –

$$t \geq {(d_p + b_p)}/{90}$$

Where t is the thickness of the column web or doubler plate, d_p is the panel zone depth between continuity plate and b_p is the panel zone width between the column flanges.

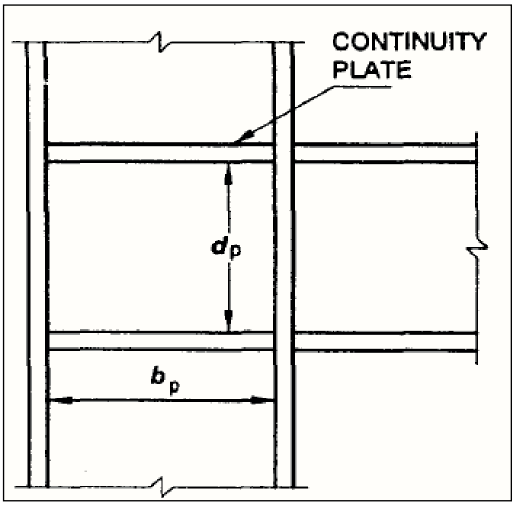


Fig. 6 Continuity plate

All beam to column connections were designed to withstand a moment of at least 1.2 times the plastic moment (M_p) of the connected beam. The connections were designed to withstand a shear resulting from the load combination of $1.2DL+0.5LL$ in addition to the shear resulting from the application of $1.2M_p$ in the same direction, at each end of the beam, resulting in double curvature.

All bolts used in frames designed to resist earthquake loads were fully tensioned high strength friction grip (HSFG) bolts or turned and fitted bolts. The welds used were complete penetration butt welds. The bolted joints were designed to ensure they did not share load in combination with welds on the same faying surface.

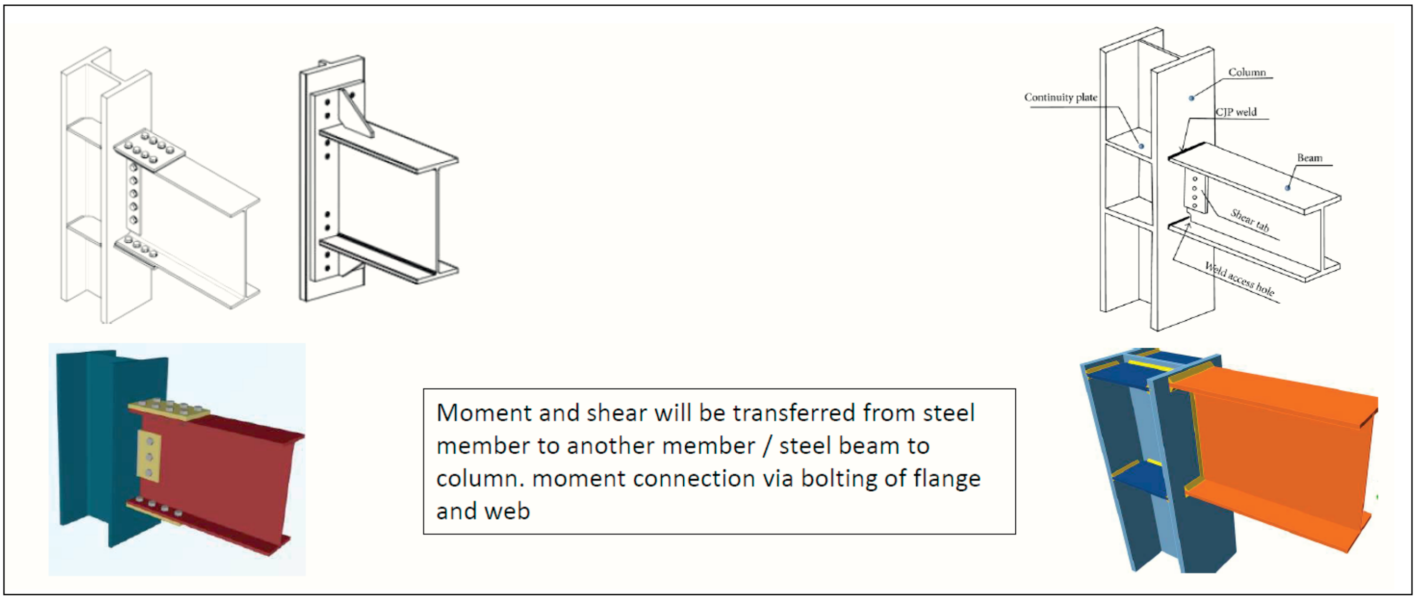


Fig. 7 Connection details

DESIGN FOR NON-STRUCTURAL ELEMENTS

Non-structural elements are extensively provided in airports to ensure buidling functionality. These are of paramount importance with respect to seamless operation and their failure in the event of an earthquake have severe repercussions which include loss of life, major damage to assets, completely jeopardising the safe evacuation and also can render the building non-functional. The National Building Code clearly mentions that the non-structural elements critical to operability of essential facilities such as hospitals, airports, emergency response centers, data centers, buildings vital to national defence, etc. continue to operate following strong earthquake shaking.They should be adequately attached to the supporting structure so that earthquake shaking does not cause them to topple or fall. Reference to clause 12.6.4 of NBC Volume 2 may be made in this regard.

Fig. 8 Typical arrangement of utility system in an airport building

In order to ensure the safety of the non-structural elements, the MEP supports were designed with a conservative approach considering zone V for the purpose. The support spacings and their arrangement were designed accordingly to render effective stability during an earthquake. It is important to not just secure the arrangements in place, but also avoid excessive sway since they may have a pounding effect on the adjacent supports and result in progressive failure.

CONCLUSION

The intent of this article is to present a guideline on codal reference which was adopted for design of an airport project in high seismic zone. The latest revisions of all relevant standards were adopted for the purpose of designing. However, the development in terms of evolution of standards is dynamic and the design work may be improved in future projects by adoption of more relevant standards like IS 18168 for Earthquake Reisstant Design and Detailing of Steel Buildings, IS 16700 for seismic design of non-structural elements, etc. In areas where we are still making progress with respect to development of standards, the encouragement is to adopt international standards to ensure comprehensive design is performed for each and every element of the structure. Any engineering judgement arising out of inadequate knowhow and thumb rule adoption, specifically in such critical structures, can really translate into major failure in adverse conditions.