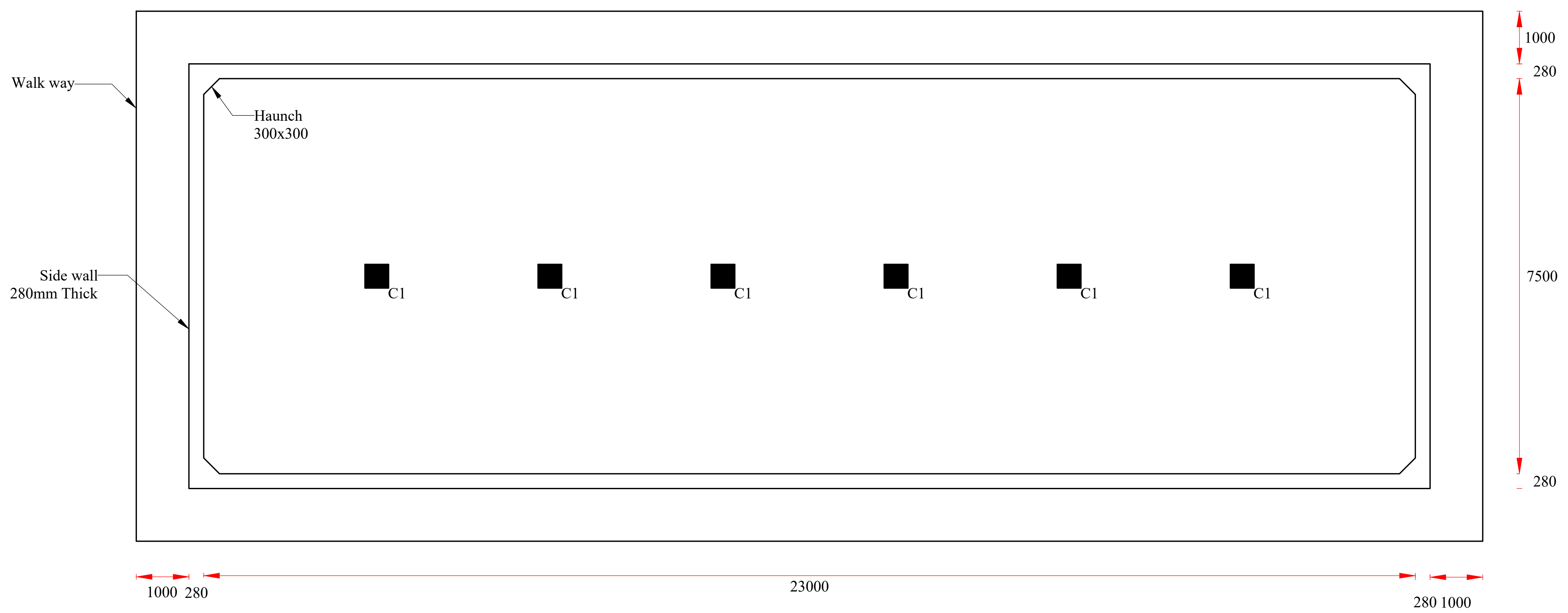
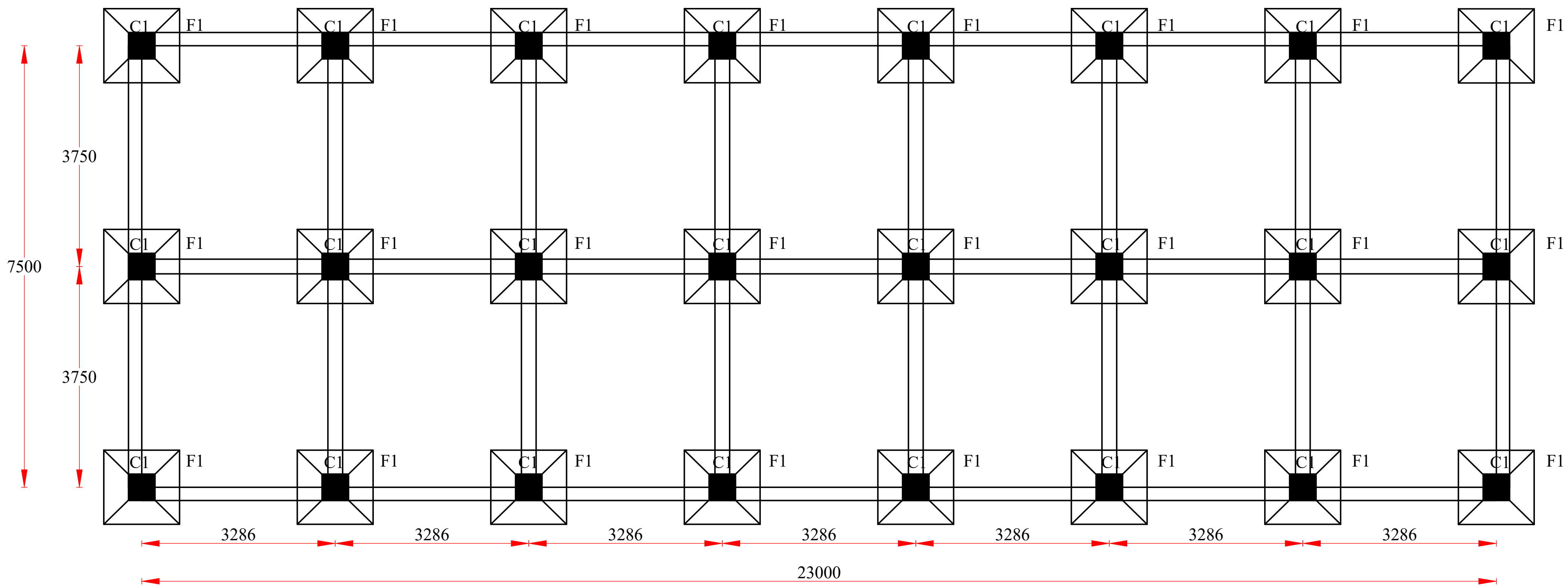


Plan of OHSR
(Scale - 1:100)



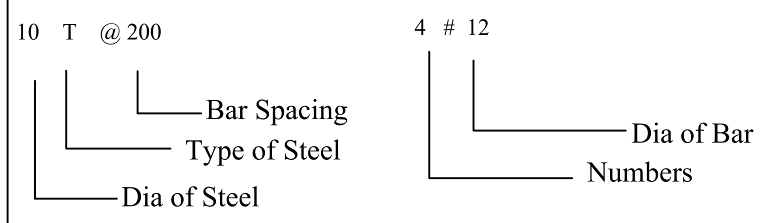
Foundation and Column Arrangement
(Scale - 1:100)

NOTES

1. All dimensions are in millimeters and levels are in meters unless otherwise specified.
2. Do not scale the drawing but follow figured dimensions only.
3. PCC mix to be 1:3:6 with 40 mm broken stones.
4. Concrete Mix M30 for water retaining portions (Roof Slab, Roof Beam, Side Wall, Floor Slab and Floor Beams) and M25 for Other Portions.
5. Allowable bearing capacity of soil at a depth 2.0 m below existing ground level is 250 kN/m².
6. Use Reinforcement of TMT with fy > 500 N/mm^2
7. All internal faces in contact with water to be plastered with c.m. 1:3, 12mm tck with flushing coat.
8. Maximum Width of Brick Masonry is 150 mm
9. Clear cover to the reinforcement should be provided as follows

Water face of slab,beam and side wall	-45mm
Column	-40mm
All other faces	-30mm.

LEGEND



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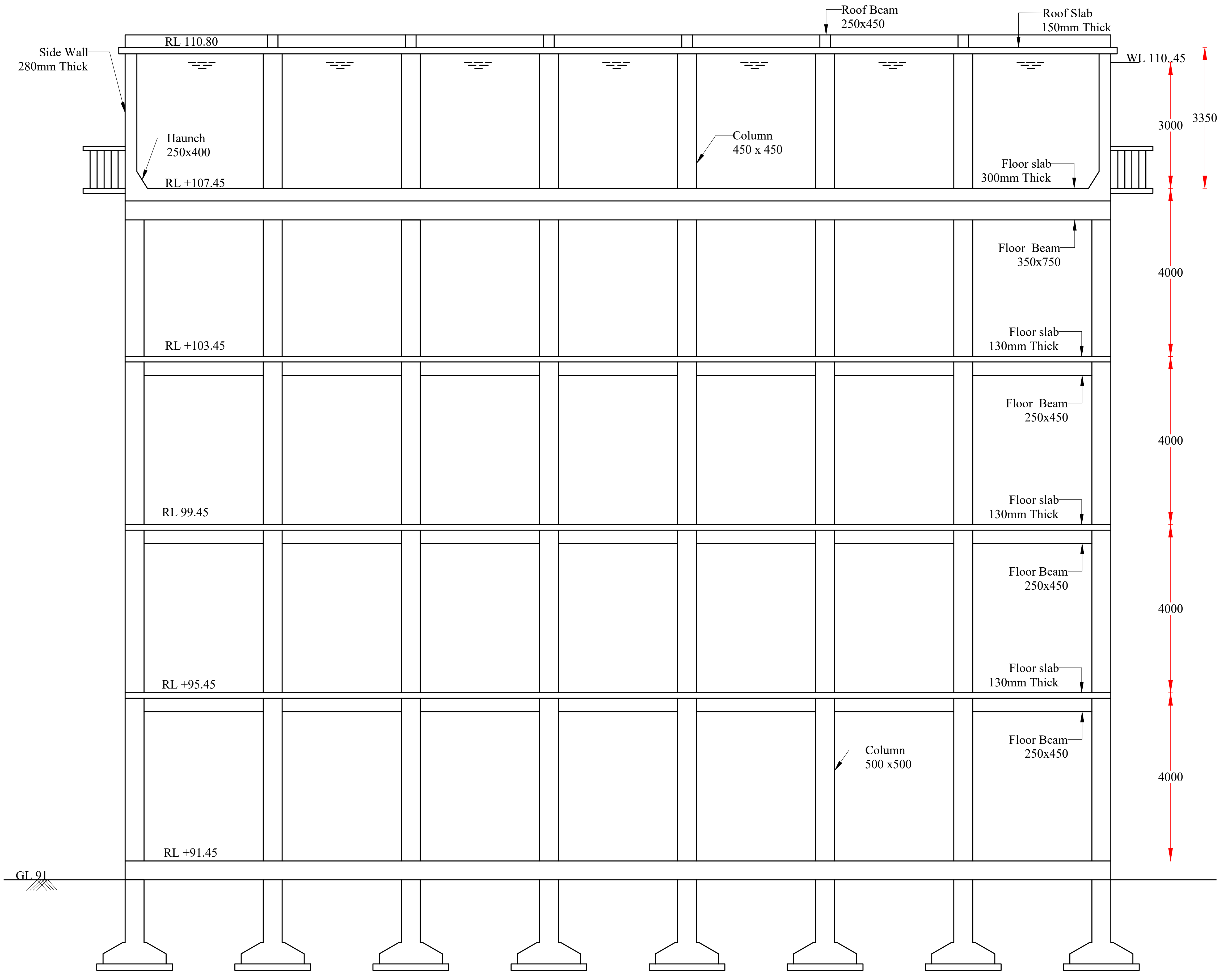
Public Health Division, Kottarakkara

Name of Work :-
JJM -2020-21 : BAWSS to Ezhukone Grama Panchayath Balance Work - Construction of 5 Lakh Litre Capacity OHSR at Thoppilpara in Ezhukone Panchayath for improving Water Supply in Ezhukone Panchayath

Component :- Foundation and Column Arrangements of Columns and Plan of Water Tank

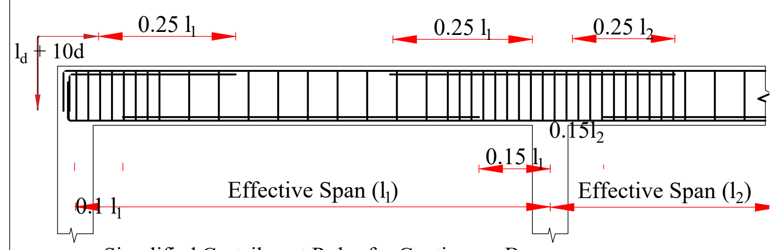
Sheet No : - 1 of 5



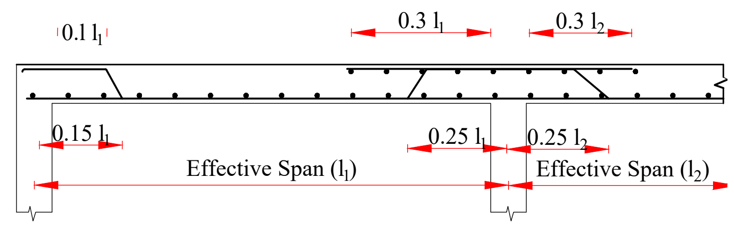
Sectional Elevation
(Scale - 1:100)

Development Length(L_d)

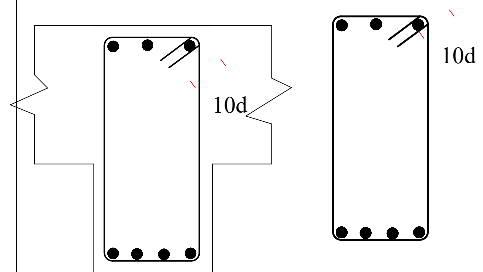
Sl No	Dia (mm)	M25	M30
		Length (mm)	Length (mm)
1	8	390	365
2	10	490	455
3	12	585	545
4	16	777	725
5	20	975	910
6	25	1214	1140



Simplified Curtailment Rules for Continuous Beams



Simplified Curtailment Rules for Continuous Slabs



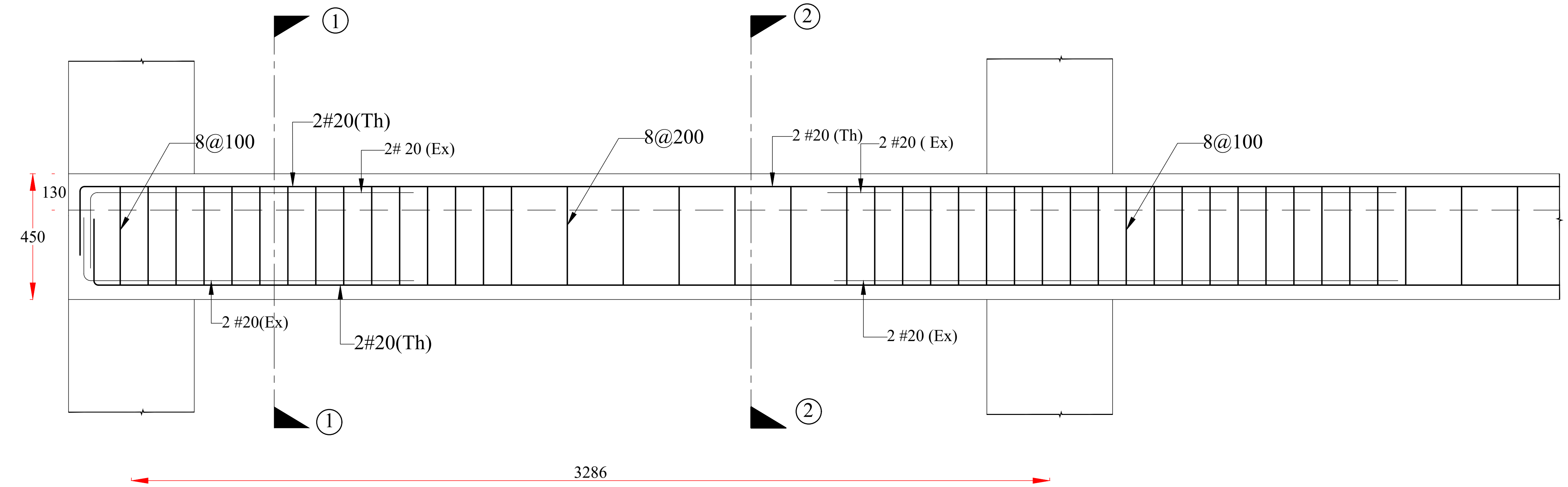
Typical Section for Beam and stirrups

Kerala Water Authority
Public Health Division, Kottarakkara

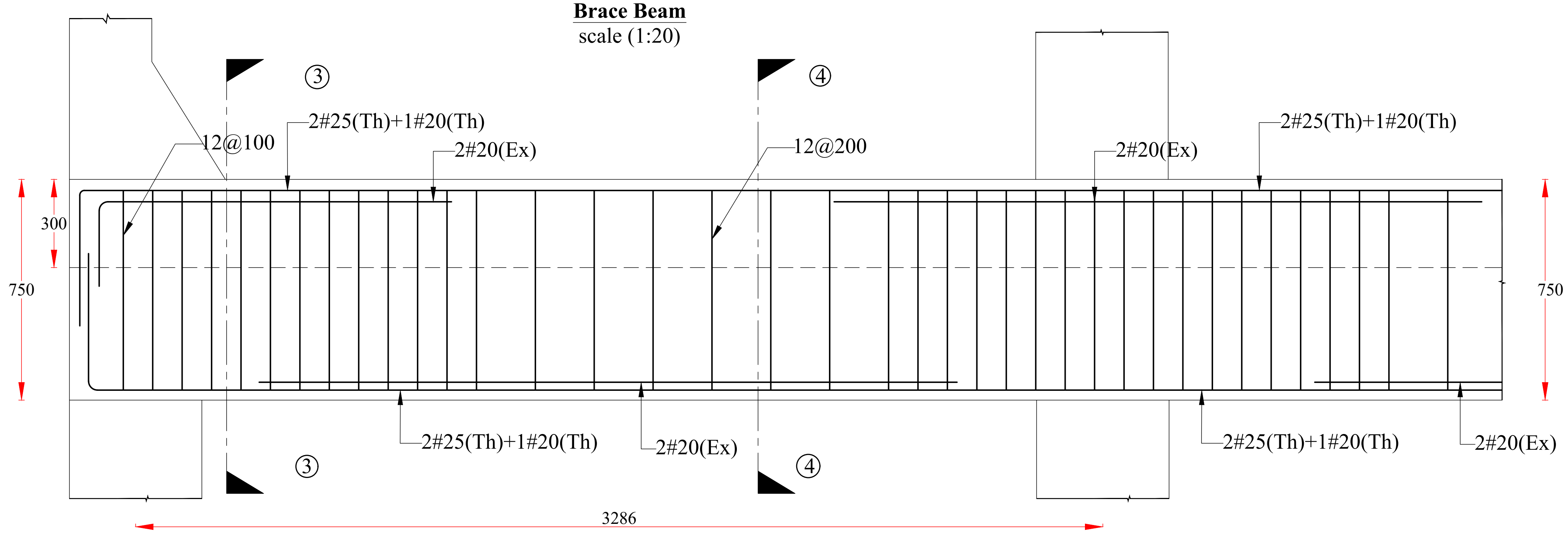
Name of Work :-
JJM -2020-21 : BAWSS to Ezhukone Grama Panchayath Balance Work - Construction of 5.0 LL Capacity OHSR at Thoppilpara in Ezhukone Panchayath for improving Water Supply in Ezhukone Panchayath

Component :- Sectional Elevation

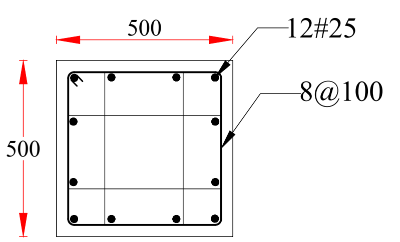
Sheet No : - 2 of 5



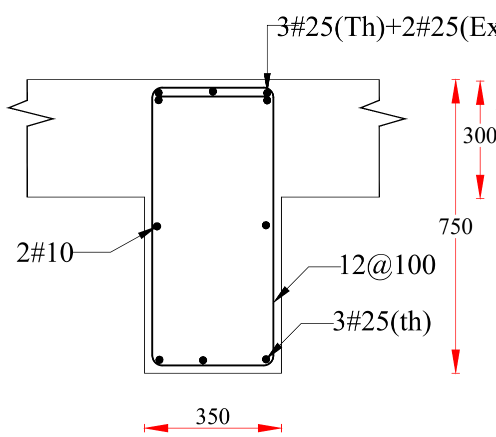
Brace Beam
scale (1:20)



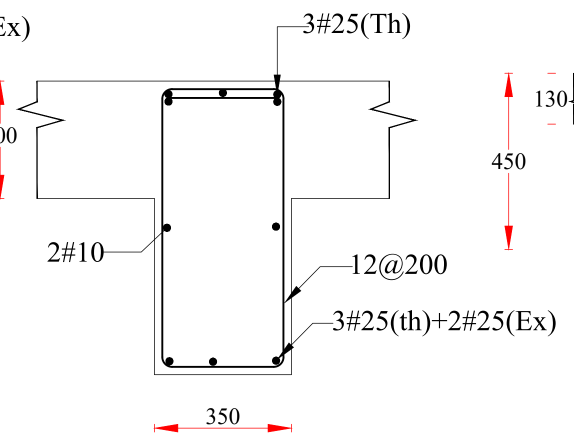
Floor Beam (B2)
scale (1:20)



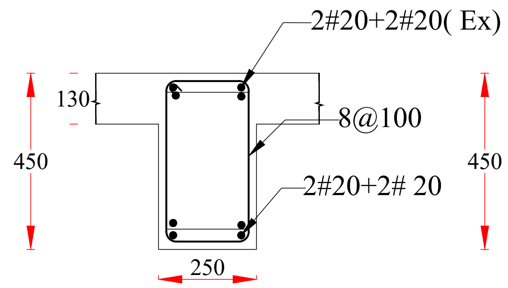
Column (C1)
Near Beam Face
scale (1:20)



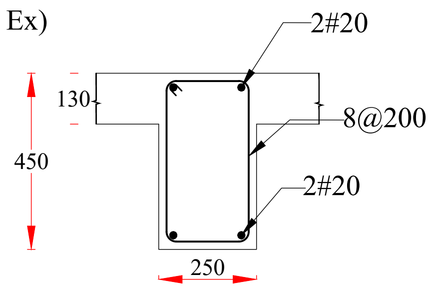
Floor Beam of OHSR(B2)
Support
Section 3-3
scale (1:20)



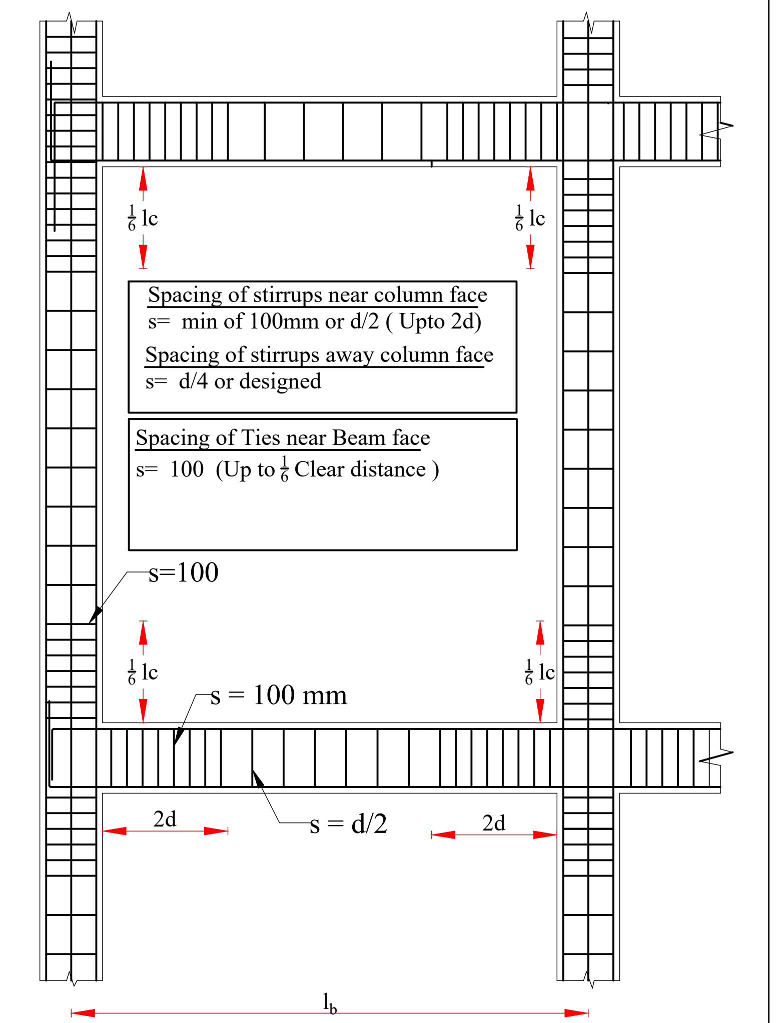
Floor Beam of OHSR(B2)
Midspan
Section 4-4
scale (1:20)



Brace Beam
Support
Section 1-1
scale (1:20)



Brace Beam (B1)
Mid span
Section 2-2
scale (1:20)



Typical Column Beam Jn

d= D - clear cover - $\frac{1}{2}$ bar dia
Where
D = Total Depth
d = Effective Depth

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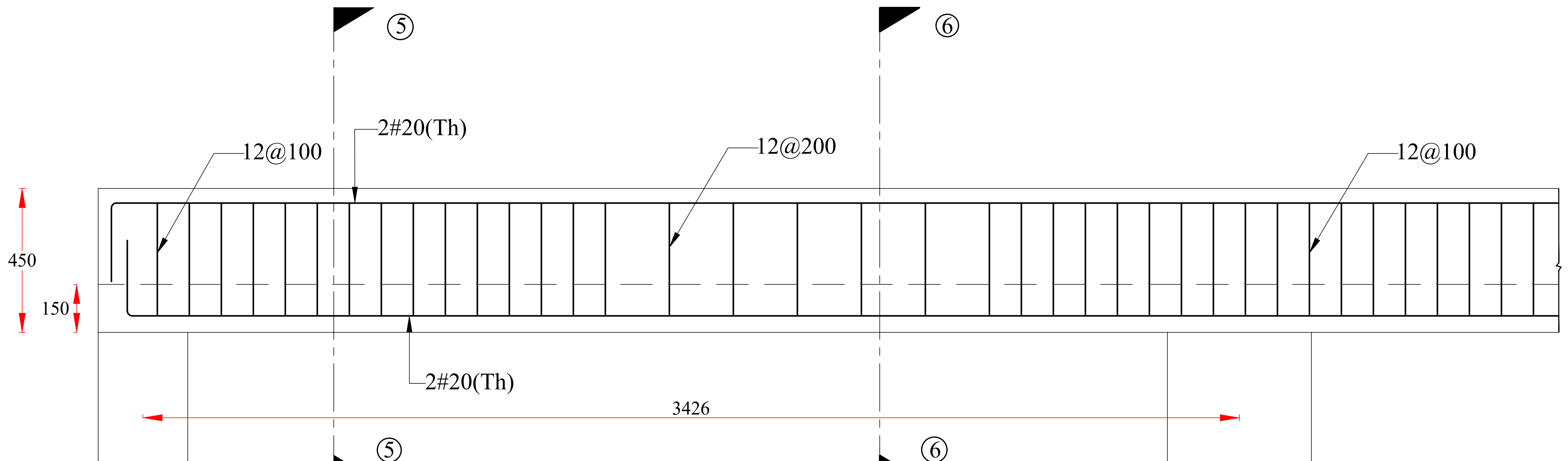
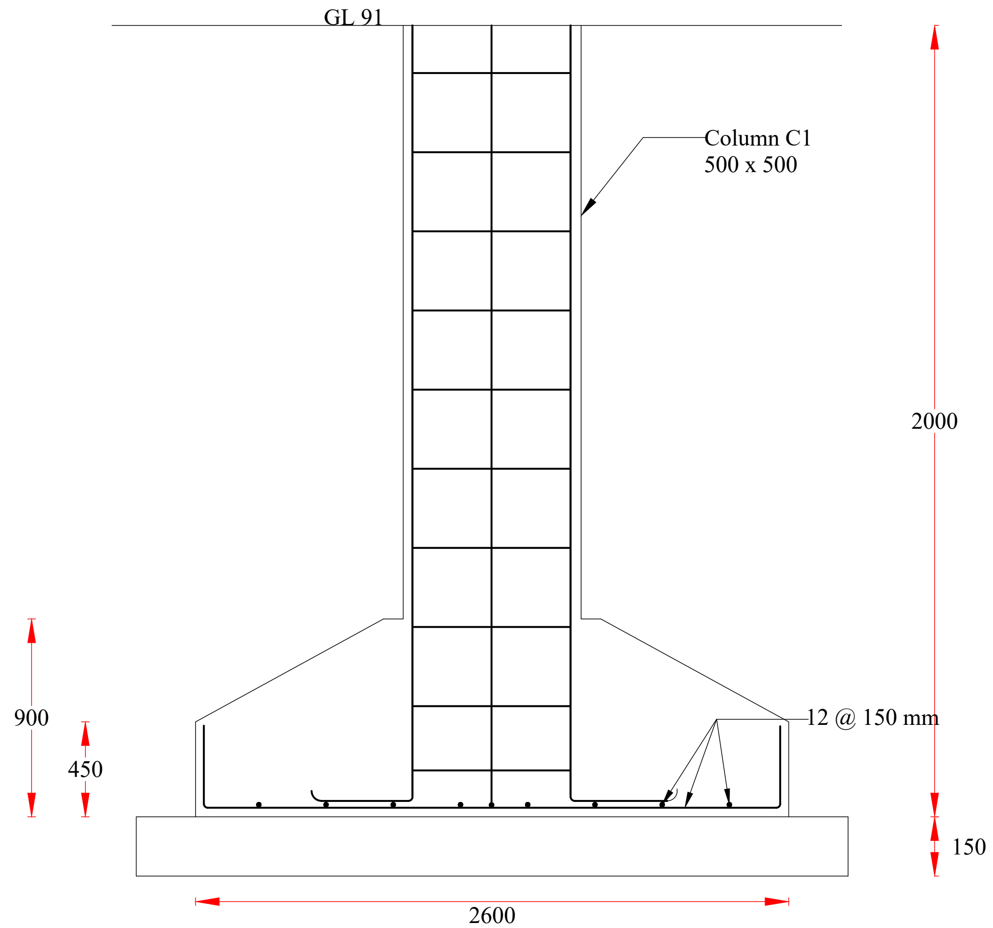
Name of Work :-
JJM -2020-21 : BAWSS to Ezhukone Grama Panchayath Balance Work - Construction of 5.00 LL Capacity OHSR at Thoppilpara in Ezhukone Panchayath for improving Water Supply in Ezhukone Panchayath

Component :- Reinforcement Details of Floor Beam, Roof Beam and Columns

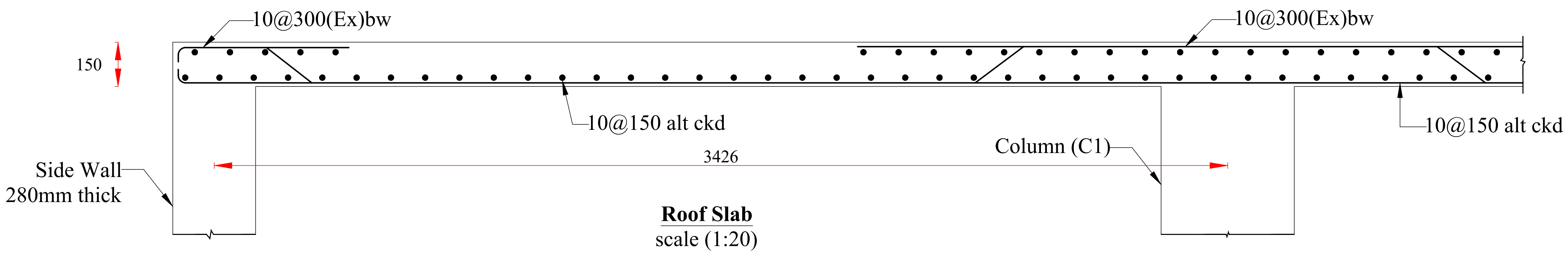
Sheet No : -3 of 5



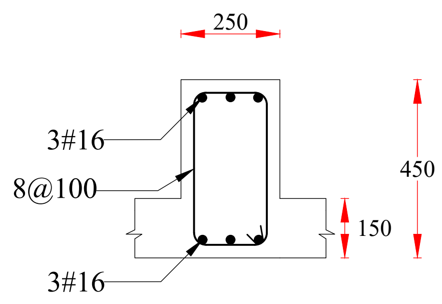
Roof Beam (B3)
scale (1:20)



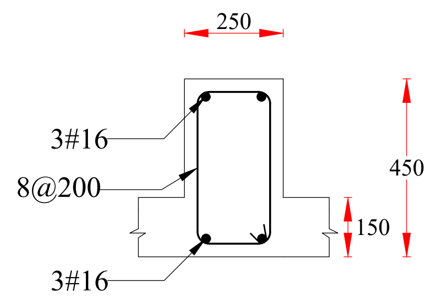
Section of footing X-X
Scale 1:20



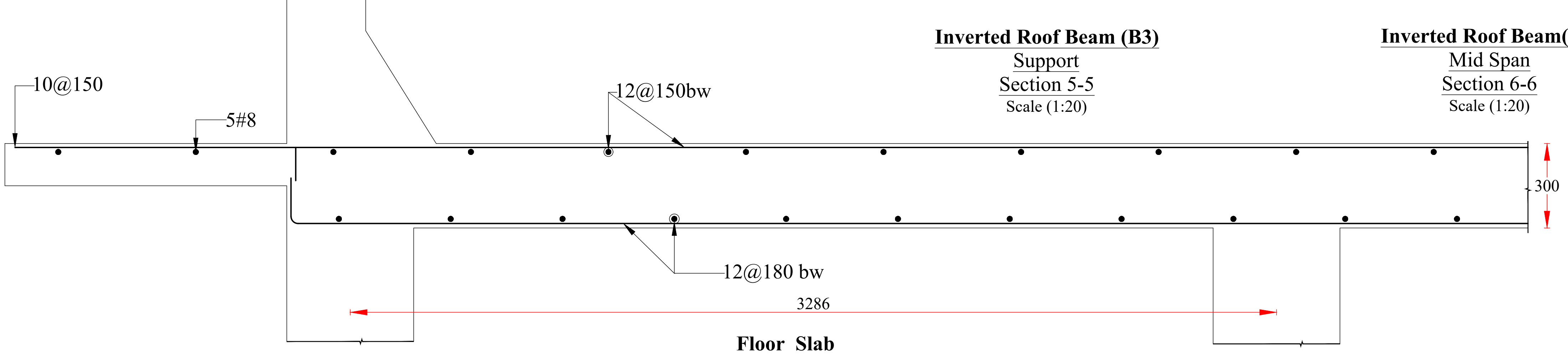
Roof Slab
scale (1:20)



Inverted Roof Beam (B3)
Support
Section 5-5
Scale (1:20)



Inverted Roof Beam(B3)
Mid Span
Section 6-6
Scale (1:20)



Floor Slab
scale (1:20)

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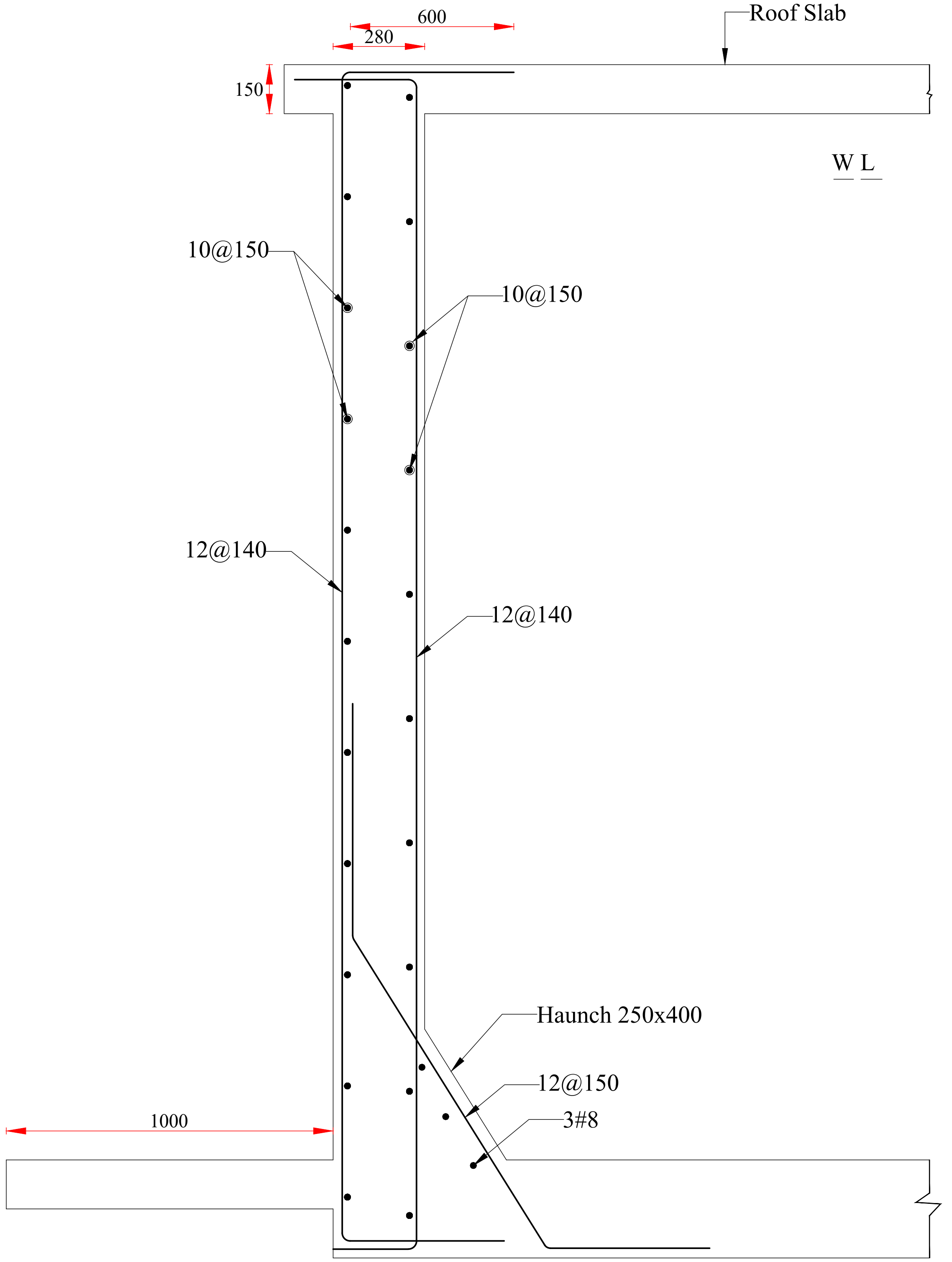
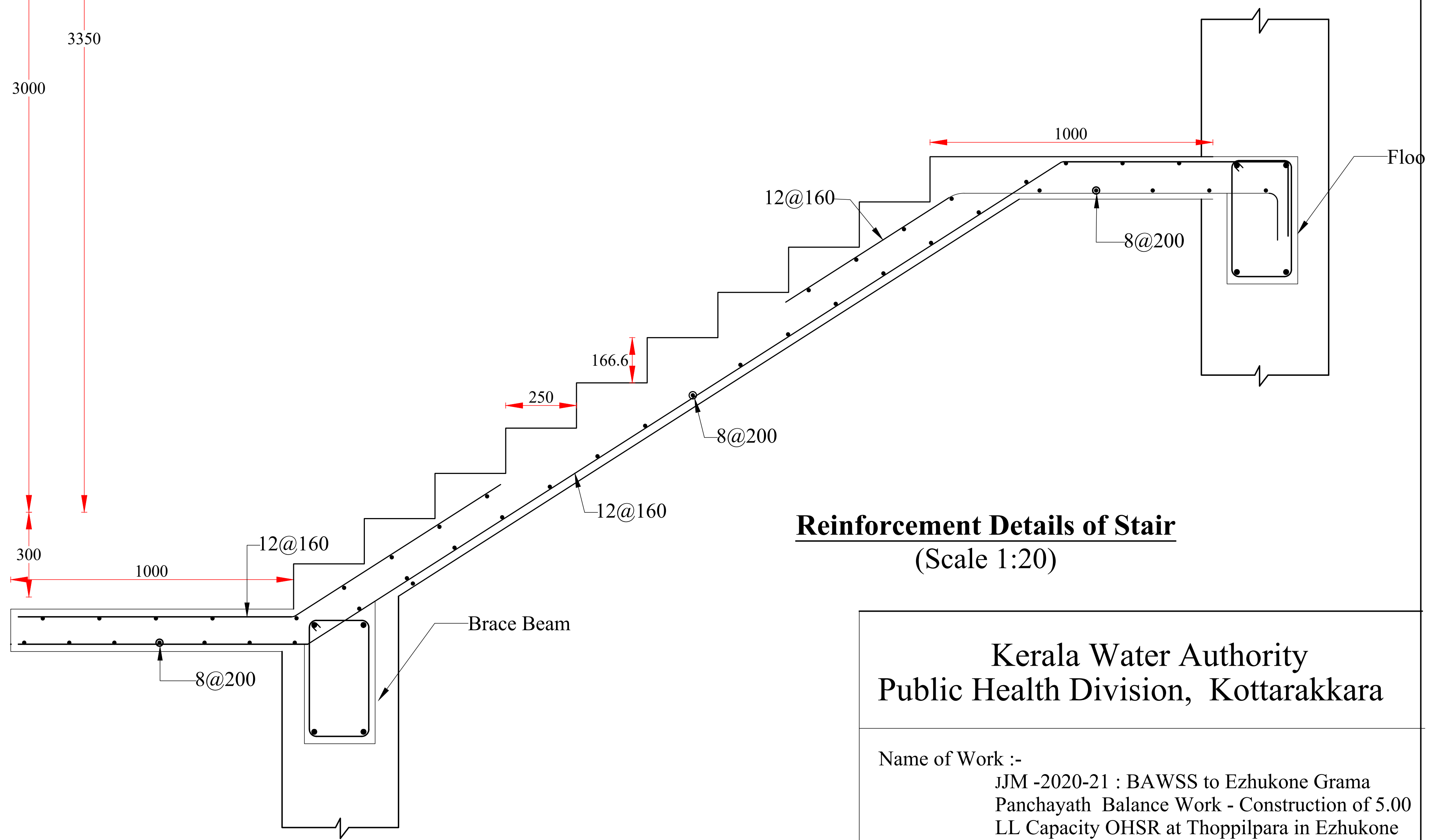
JJM -2020-21 : BAWSS to Ezhukone Grama Panchayath Balance Work - Construction of 5.0 LL Capacity OHSR at Thoppilpara in Ezhukone Panchayath for improving Water Supply in Ezhukone Panchayath

Component :- Reinforcement Details of Foundation, Floor Slab, Brace Slab and Roof Beam

Sheet No : - 4 of 5



Side Wall OHSR
scale (1:20)



Reinforcement Details of Stair
(Scale 1:20)

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Name of Work :-
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Component :- Reinforcement Details of Stair Case and SIde Wall

Sheet No : - 5 of 5

भारतीय मानक

Indian Standard

IS 1893 (Part 2) : 2014

संरचनाओं के भूकम्परोधी डिजाइन के मानदंड

भाग 2 द्रव धारित टैंक

(पाँचवाँ पुनरीक्षण)

Criteria for Earthquake Resistant Design of Structures

Part 2 Liquid Retaining Tanks

(Fifth Revision)

ICS 91.120.25

भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS
मानक भवन, 9 बहादुरशाह ज़फर मार्ग, नई दिल्ली-110002
MANAK BHAVAN, 9 BAHADUR SHAH ZAFAR MARG
NEW DELHI-110002
www.bis.org.in www.standardsbis.in

August 2014 **Price Group 8**

Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard (Part 2) (Fifth Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

In the fifth revision IS 1893 has been split into five parts. The other parts in the series are:

Part 1	General provisions and buildings
Part 3	Bridges and retaining walls
Part 4	Industrial structures including stack like structures
Part 5	Dams and embankments

Part 1 contains provisions that are general in nature and applicable to all types of structures. It also contains provisions that are specific to buildings only. Unless stated otherwise, the provisions in Part 2 to Part 5 shall be read in conjunction with the general provisions in Part 1.

This standard (Part 2) contains provisions for liquid retaining tanks. Unless otherwise stated, this standard shall be read necessarily in conjunction with IS 1893 (Part 1) : 2002.

As compared to provisions of IS 1893 : 1984, in this standard following important provisions and changes have been incorporated:

a) Analysis of ground supported tanks is included.
b) For elevated tanks, the single degree of freedom idealization of tank is done away with; instead a two-degree of freedom idealization is used for analysis.
c) Bracing beam flexibility is explicitly included in the calculation of lateral stiffness of tank staging.
d) The effect of convective hydrodynamic pressure is included in the analysis.
e) The distribution of impulsive and convective hydrodynamic pressure is represented graphically for convenience in analysis; a simplified hydrodynamic pressure distribution is also suggested for stress analysis of the tank wall.
f) Effect of vertical ground acceleration on hydrodynamic pressure is considered.
g) Quality control measures considered necessary in deign and construction of reinforced concrete tanks for achieving safe performance under normal as well as seismic conditions are also included.

The units used with the items covered by the symbols shall be consistent throughout this standard, unless specifically noted otherwise.

In the formulation of this standard due weightage has been given to international coordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field of this country.

In the formulation of this standard considerable help has been taken by the Indian Institute of Technology Kanpur, Institute of Technology Roorkee, Visvesvaraya National Institute of Technology, Nagpur and several other organizations including Guidelines prepared by IIT, Kanpur for GSDMA.

Reference has been made to the following documents in the formulation of this standard:

a) ACI 350.3, 2001, 'Seismic design of liquid containing concrete structures', American Concrete Institute, Farmington Hill, MI, USA.
b) Eurocode 8, 1998, 'Design provisions for earthquake resistance of structures, Part 1 General rules and Part 4 – Silos, tanks and pipelines', European Committee for Standardization, Brussels.

(Continued on third cover)

Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 2 LIQUID RETAINING TANKS

(Fifth Revision)

1 SCOPE

This standard (Part 2) covers ground supported liquid retaining tanks and elevated tanks supported on staging. Guidance is also provided on seismic design of buried tanks.

2 REFERENCES

The following standards contain provisions which, through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
456 : 2000	Code of Practice for plain and Reinforced Concrete (*fourth revision*)
1893 (Part 1) : 2002	Criteria for earthquake resistant design of structures: Part 1 General provisions and buildings (*fifth revision*)
3370	Code of Practice for concrete structures for the storage of liquids
(Part 1) : 2009	General requirements (*first revision*)
(Part 2) : 2009	Reinforced concrete structures (*first revision*)
(Part 3) : 1967	Prestressed concrete structures
(Part 4) : 1967	Design tables
4326 : 2013	Code of Practice for earthquake resistant design and construction of buildings (*third revision*)
11682 : 1985	Criteria for design of RCC staging for overhead water tanks
13920 : 1993	Ductile detailing of reinforced concrete structures subjected to seismic sorces — Code of Practice

3 SYMBOLS

The symbols and notations given below apply to the provisions of this standard:

A_h = Design horizontal seismic coefficient

$(A_h)_c$ = Design horizontal seismic coefficient for convective mode

$(A_h)_i$ = Design horizontal seismic coefficient for impulsive mode

A_v = Design vertical seismic coefficient

B = Inside width of rectangular tank perpendicular to the direction of seismic force

C_c = Coefficient of time period for convective mode

C_i = Coefficient of time period for impulsive mode

d = Deflection of wall of rectangular tank, on the vertical centre line at a height h, when loaded by a uniformly distributed pressure q, in the direction of seismic force

d_{max} = Maximum sloshing wave height

D = Inner diameter of circular tank

E = Modulus of elasticity of tank wall

EL_x = Response quantity due to earthquake load applied in x - direction

EL_y = Response quantity due to earthquake load applied in y - direction

g = Acceleration due to gravity

h = Maximum depth of liquid

$\bar{h}$ = Height of combined centre of gravity of half impulsive mass of liquid ($m_i/2$) and mass of one wall $\left(\bar{m}_w\right)$

h_c = Height of convective mass above bottom of tank wall (without considering base pressure)

h_i = Height of impulsive mass above bottom of tank wall (without considering base pressure)

h_s = Structural height of staging, measured from top of foundation to the bottom of container wall

h_t = Height of centre of gravity of roof mass above bottom of tank wall

h_w = Height of centre of gravity of wall mass above bottom of tank wall

h_c^* = Height of convective mass above bottom of tank wall (considering base pressure)

h_i^* = Height of impulsive mass above bottom of tank wall (considering base pressure)

h_{cg} = Height of centre of gravity of the empty container of elevated tank, measured from the top of footing

I = Importance factor given in Table 1

K_c = Spring stiffness of convective mode

K_s = Lateral stiffness of elevated tank staging

l = Length of a strip at the base of circular tank, along the direction of seismic force

L = Inside length of rectangular tank parallel to the direction of seismic force

m = Total mass of liquid in tank

m_b = Mass of base slab or plate

m_c = Convective mass of liquid

m_i = Impulsive mass of liquid

m_s = Mass of empty container (includes mass of members like roof, wall, tank floor, floor beams, etc) of elevated tank and one-third mass of staging (mass of tower excluding container and foundation. Mass of columns, braces and any other mass attached to staging shall be included in mass of staging. Mass of pedestal above foundation can be assumed to be attached to foundation)

m_t = Mass of roof slab

m_w = Mass of tank wall

$\bar{m}_w$ = Mass of one wall of rectangular tank perpendicular to the direction of loading

M = Total bending moment at the bottom of tank wall

M^* = Total overturning moment at base

M_c = Bending moment in convective mode at the bottom of tank wall

M^*_c = *Overturning* moment in convective mode at the base

M_i = Bending moment in impulsive mode at the bottom of tank wall

M^*_i = Overturning moment in impulsive mode at the base

p = Maximum hydrodynamic pressure on wall

p_{cb} = Convective hydrodynamic pressure on tank base

p_{cw} = Convective hydrodynamic pressure on tank wall

p_{ib} = Impulsive hydrodynamic pressure on tank base

p_{iw} = Impulsive hydrodynamic pressure on tank wall

p_v = Hydrodynamic pressure on tank wall due to vertical ground acceleration

p_{ww} = Pressure on wall due to its inertia

q = Uniformly distributed pressure on one wall of rectangular tank in the direction of ground motion

Q_{cb} = Coefficient of convective pressure on tank base

Q_{cw} = Coefficient of convective pressure on tank wall

Q_{ib} = Coefficient of impulsive pressure on tank base

Q_{iw} = Coefficient of impulsive pressure on tank wall

R = Response reduction factor given in Table 2

(S_a/g) = Average response acceleration coefficient as per IS 1893 (Part 1) and **4.5**

t = Thickness of tank wall

t_b = Thickness of base slab

T_c = Time period of convective mode (in s)

T_i = Time period of impulsive mode (in s)

V = Total base shear

V_c = Base shear in convective mode

V_i = Base shear in impulsive mode

x = Horizontal distance in the direction of seismic force, of a point on base slab from the reference axis at the centre of tank

y = Vertical distance of a point on tank wall from the bottom of tank wall

Z = Seismic zone factor as per Table 2 of IS 1893 (Part 1)

ρ = Mass density of liquid

ρ_w = Mass density of tank wall

ϕ = Circumferential angle

4 PROVISIONS FOR SEISMIC ANALYSIS

4.1 General

Hydrodynamic forces exerted by liquid on tank wall shall be considered in the analysis in addition to hydrostatic forces. These hydrodynamic forces are evaluated with the help of spring mass model of tanks.

For tank full as well as empty conditions, tank shall be analysed for all the load combinations as per IS 1893 (Part 1). For load combination with seismic load, the amount of liquid considered in the tank shall be normal liquid level under service condition only.

4.2 Spring Mass Model for Seismic Analysis

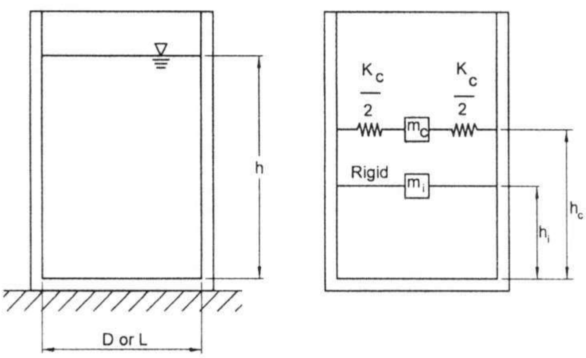
When a tank containing liquid vibrates, the liquid exerts impulsive and convective hydrodynamic pressure on the tank wall and the tank base in addition to the hydrostatic pressure. In order to include the effect of hydrodynamic pressure in the analysis, tank can be idealized by an equivalent spring mass model, which includes the effect of tank wall-liquid interaction. The parameters of this model depend on geometry of the tank.

4.2.1 *Ground Supported Tank*

4.2.1.1 Ground supported tanks can be idealized as spring-mass model shown in Fig. 1. The impulsive mass of liquid, m_i is rigidly attached to tank wall at height h_i (or h_i^*). Similarly, convective mass, m_c is attached to the tank wall at height h_c (or h_c^*) by a spring of stiffness K_c.

4.2.1.2 *Circular and rectangular tank*

For circular tanks, parameters m_i, m_c, h_i, h_i^*, h_c, h_c^* and K_c shall be obtained from Fig. 2 and for rectangular tanks these parameters shall be obtained from Fig. 3. h_i and h_c account for hydrodynamic pressure on the tank wall only and the tank base. Hence, the value of h_i and h_c shall be used to calculate moment due to hydrodynamic pressure at the bottom of the tank wall. The value of h_i^* and h_c^* shall be used to calculate overturning moment at the base of tank.



(a) Tank (b) Spring Mass Model

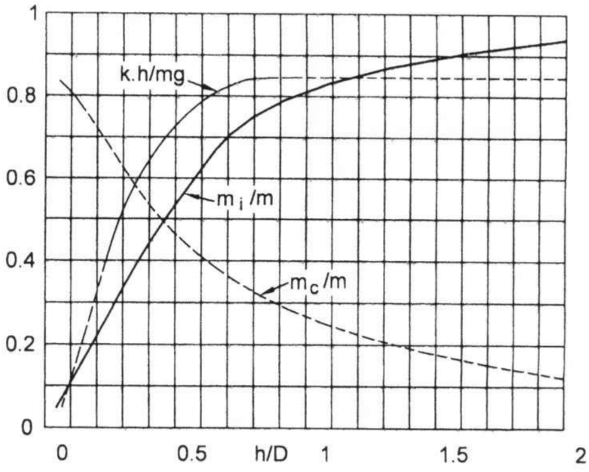
FIG. 1 SPRING MASS MODELS FOR GROUND SUPPORTED CIRCULAR AND RECTANGULAR TANK

4.2.2 *Elevated Tank*

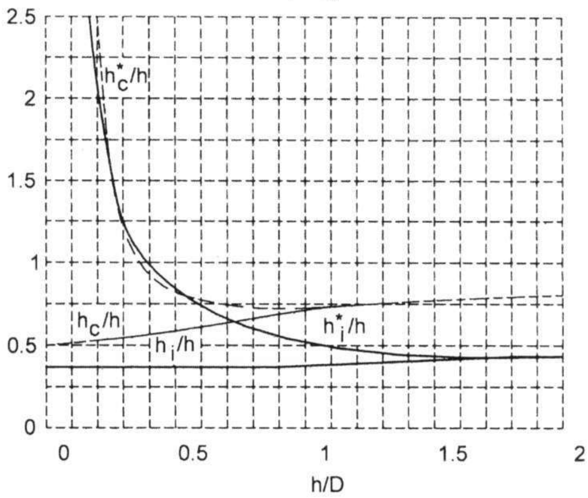
4.2.2.1 Elevated tanks (*see* Fig. 4a) can be idealized by a two-mass model as shown in Fig. 4c.

4.2.2.2 For elevated tanks with circular container, parameters m_i, m_c, h_i, h_i^*, h_c h_c^* and K_c shall be obtained from Fig. 2. For elevated tanks with rectangular container, these parameters shall be obtained from Fig. 3.

4.2.2.3 In Fig. 4c, m_s is the structural mass and shall comprise of mass of tank container and one-third mass of staging.



(a) Impulsive and Convective Mass and Convective Spring Stiffness

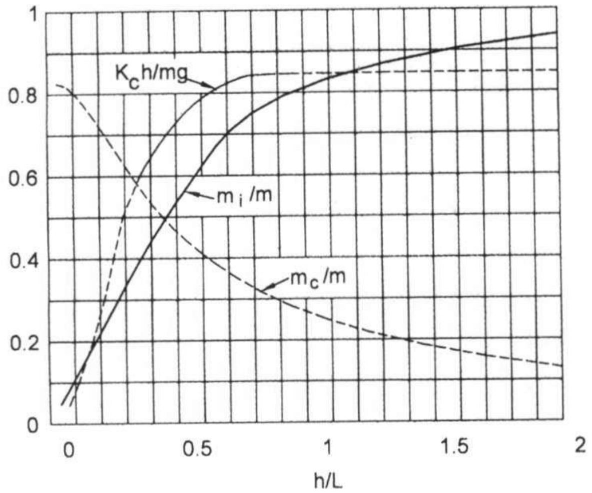


(b) Height of Impulsive and Convective Masses

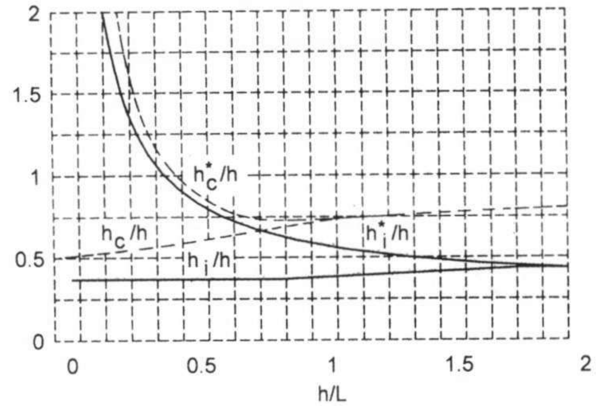
FIG. 2 PARAMETERS OF THE SPRING MASS MODEL FOR CIRCULAR TANK

4.2.2.4 For elevated tanks, the two degree of freedom system of Fig. 4c can be treated as two uncoupled single degree of freedom systems (*see* Fig. 4d), one representing the impulsive plus structural mass behaving as an inverted pendulum with lateral stiffness equal to that of the staging, K_s and the other representing the convective mass with a spring of stiffness, K_c.

4.2.3 For tank shapes other than circular (like intze, truncated conical shape), the value of *h*/*D* shall correspond to that of an equivalent circular tank of same volume and diameter equal to diameter of tank at top level of liquid; and m_i, , m_c , h_i, h_i^* , h_c h_c^* and K_c of equivalent circular tank shall be used. The equivalent cylindrical tank should be assumed to be located such that top level of the liquid in equivalent tank is same as in actual tank.



(a) Impulsive and Convective Mass and Convective Spring Stiffness



(b) Height of Impulsive and Convective Masses

FIG. 3 PARAMETERS OF THE SPRING MASS MODEL FOR RECTANGULAR TANK

4.3 Time Period

4.3.1 *Impulsive Mode*

4.3.1.1 *Ground supported circular tank*

For a ground supported circular tank, wherein wall is rigidly connected with the base slab (*see* Fig. 6a, 6b and 6c), time period of impulsive mode of vibration T_i, in second, is given by:

$$T_i = C_i \frac{h\sqrt{\rho}}{\sqrt{t/D}\sqrt{E}}$$

where

C_i = coefficient of time period for impulsive mode. Value of C_i can be obtained from Fig. 5;

h = maximum depth of liquid;

D = inner diameter of circular tank;

t = thickness of tank wall;

E = modulus of elasticity of tank wall; and

ρ = mass density of liquid.

NOTE — In some circular tanks, wall may have flexible connection with the base slab (Different types of wall to base slab connections are described in Fig. 6). For tanks with flexible connections with base slab, time period evaluation may properly account for the flexibility of wall to base connection.

4.3.1.2 *Ground supported rectangular tank*

For a ground supported rectangular tank, wherein wall is rigidly connected with the base slab, time period of impulsive mode of vibration, T_i in s, is given by:

$$T_i = 2\pi\sqrt{d/g}$$

where

d = deflection of the tank wall on the vertical center-line at a height of h, when loaded by uniformly distributed pressure of intensity q.

$$q = \frac{\left(\frac{m_i}{2} + \bar{m}_w\right)g}{Bh}$$

$$\bar{h} = \frac{\frac{m_i}{2}h_i + \bar{m}_w\frac{h}{2}}{\frac{m_i}{2} + \bar{m}_w}$$

$\bar{m}_w$ = mass of one tank wall perpendicular to the direction of seismic force; and

B = inside width of tank.

4.3.1.3 *Elevated tank*

Time period of impulsive mode, T_i, in s, is given by:

$$T_i = 2\pi\sqrt{\frac{m_i + m_s}{K_s}}$$

where

m_s = mass of empty container and one-third mass of staging; and

K_s = lateral stiffness of staging.

Lateral stiffness of the staging is the horizontal force required to be applied at the centre of gravity of the tank to cause a corresponding unit horizontal displacement.

NOTE — The flexibility of bracing beam shall be considered in calculating the lateral stiffness, K_s of elevated moment resisting frame type tank staging.

4.3.2 *Convective Mode*

Time period of convective mode can be calculated using **4.3.2.1** and **4.3.2.2**. However, shorter time period shall be used for design purposes.

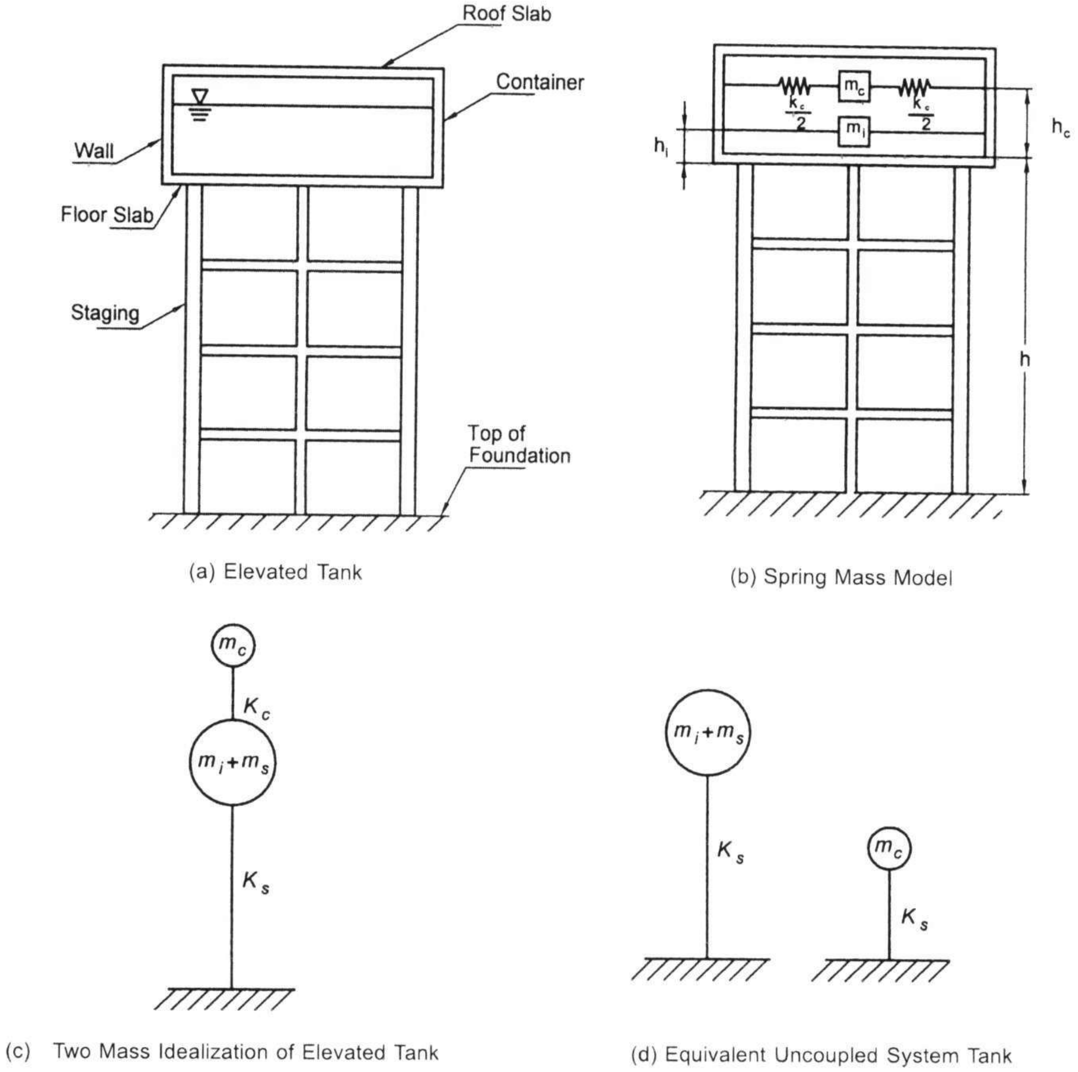


FIG. 4 TWO MASS IDEALIZATION FOR ELEVATED TANK

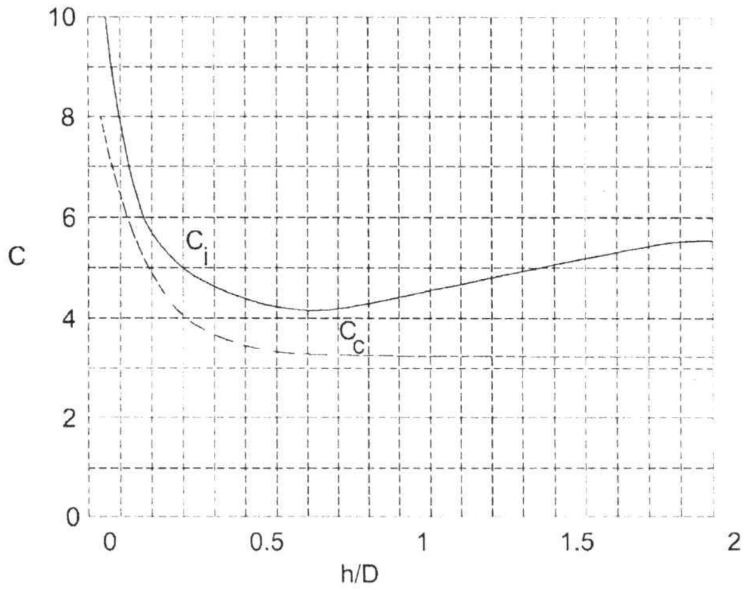


FIG. 5 COEFFICIENT OF IMPULSIVE AND CONVECTIVE MODE TIME PERIOD FOR CIRCULAR TANK

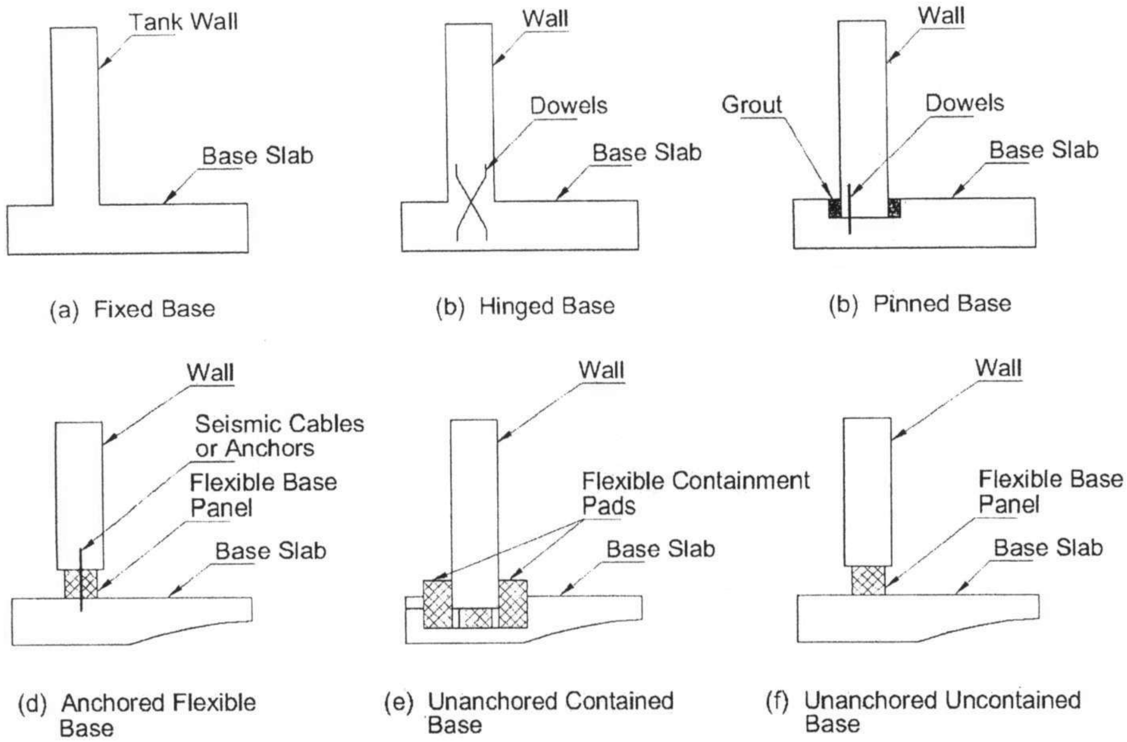


FIG. 6 TYPE OF CONNECTIONS BETWEEN TANK WALL AND BASE SLAB

4.3.2.1 Time period of convective mode, in second, is given by:

$$T_1 = 2\pi\sqrt{\frac{m_c}{K_c}}$$

The values of m_c and K_c can be obtained from Figs. 2a and 3a respectively, for circular and rectangular tanks.

4.3.2.2 Since the expressions for m_c and K_c are known, the expression for T_c can be alternatively expressed as:

a) *Circular tank* — Time period of convective mode, T_c, in s, is given by:

$$T_c = C_c\sqrt{\frac{D}{\rho}}$$

where

C_c = coefficient of time period for convective mode. Value of C_c can be obtained from Fig. 5; and

D = inner diameter of tank.

b) *Rectangular tank* — Time period of convective mode of vibration, T_c, in second, is given by:

$$T_c = C_c\sqrt{\frac{L}{\rho}}$$

where

C_c = coefficient of time period for convective mode. Value of C_c can be obtained from Fig. 7; and

L = inside length of tank parallel to the direction of seismic force.

4.3.3 For tanks resting on soft soil, effect of flexibility of soil may be considered while evaluating the time period. Generally, soil flexibility does not affect the convective mode time period. However, soil flexibility may affect impulsive mode time period.

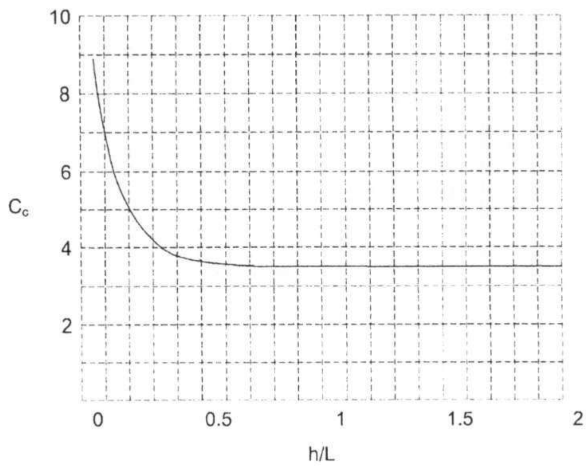


FIG.7 COEFFICIENT OF CONVECTIVE MODE TIME PERIOD (C_c) FOR RECTANGULAR TANK

4.4 Damping

Damping in the convective mode for all types of liquids and for all types of tanks shall be taken as 0.5 percent of the critical.

Damping in the impulsive mode shall be taken as 2 percent of the critical for steel tanks and 5 percent of the critical for concrete or masonry tanks.

4.5 Design Horizontal Seismic Coefficient

Design horizontal seismic coefficient, A_h shall be obtained by the following expression, subject to **4.5.1** and **4.5.2**:

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

where

Z = zone factor given in Table 2 of IS 1893 (Part 1);

I = importance factor given in Table 1;

R = response reduction factor given in Table 2 and Table 3; and

S_a/g = average response acceleration coefficient as given by Fig. 2 and multiplying factors for obtaining values for other damping as given in Table 3 of IS 1893(Part 1) and subject to **4.5.1** and **4.5.2**.

Table 1 Importance Factor, *I*

(*Clause* 4.5)

Sl No.	Type of Liquid Storage Tank	*I*
(1)	(2)	(3)
i)	Tanks used for storing drinking water, non-volatile material, low inflammable, etc, and intended for emergency services such as fire fighting services. Tanks of post earthquake importance	1.5
ii)	All other tanks with no risk to life and with negligible consequences to environment, society and economy	1.0

NOTE — Higher values of importance factor, I given in IS 1893 (Part 4) may be used where appropriate.

4.5.1 Design horizontal seismic coefficient, A_h shall be calculated separately for impulsive $(A_h)_i$, and convective $(A_h)_c$ modes.

4.5.2 Value of multiplying factor shall be taken as 1.0 for 5 percent, 1.4 for 2 percent and 1.75 for 0.5 percent damping.

4.6 Base Shear

4.6.1 *Ground Supported Tank*

Base shear in impulsive mode, at the bottom of tank wall is given by:

$$V_i = (A_h)_i\,(m_i + m_w + m_t)\,g$$

Table 2 Suggested Values of 'R' for Elevated Tanks

(*Clause* 4.5)

Sl No.	Type of Elevated Tank	'R'
(1)	(2)	(3)
i)	Tank supported on masonry shaft (*Not permitted in zones IV and V*):	
	a) Masonry shaft reinforced with horizontal bands	2
	b) Masonry shaft reinforced with horizontal bands and vertical bars	3
ii)	Tank supported on RC shaft:	
	a) RC shaft with reinforcement in one curtain (in both directions) at center of shaft thickness	2.5
	b) RC shaft with reinforcement in two curtains (in both directions)	3.5
	c) RC shaft with reinforcement in two curtains (in both directions) and with stiffened openings and bracings	4
iii)	Tank supported on RC frame[1)]:	
	a) Ordinary moment resisting frame (OMRF) type staging	2.5
	b) Special moment resisting frame (SMRF) conforming ductility requirements of IS 13920	4
iv)	Tank supported on steel frame [1)]:	
	a) Special moment resistant frame (SMRF) without diagonal bracing	3.5
	b) Special moment resistant frame (SMRF) with diagonal bracing	4

[1)] These R values are meant for liquid retaining tanks on frame type staging which are inverted pendulum type structures. These R values shall not be misunderstood for those given in other parts of IS 1893 for building and industrial frames.

NOTE — P – Δ effect should be considered in the design of the staging.

Table 3 Suggested Values of 'R' for Ground Supported Tanks

(*Clause* 4.5)

Sl No.	Type of Ground Supported Tank	R
(1)	(2)	(3)
i)	Masonry tank:	
	a) Masonry wall reinforced with horizontal bands (*Not permitted in zones IV and V*)	2.0
	b) Masonry wall reinforced with horizontal bands and vertical bars at corners and jambs of openings	3.0
ii)	RC / prestressed tank:	
	a) Fixed or hinged/pinned base tank (*see* Figs. 6a, 6b, 6c)	2.5
	b) Anchored flexible base tank (*see* Fig. 6d)	3.0
	c) Unanchored contained or uncontained tank (*see* Figs. 6e, 6f)	2.5
iii)	Steel tank:	
	a) Unanchored base	2.5
	b) Anchored base	3.0
iv)	Underground RC and steel tank (*see* Note)	4.0

NOTE — For partially buried tanks, values of R can be interpolated between ground supported and underground tanks based on depth of embedment.

and base shear in convective mode is given by:

$$V_c = (A_h)_c\, m_c\, g$$

where

$(A_h)_i$ = design horizontal seismic coefficient for impulsive mode;

$(A_h)_c$ = design horizontal seismic coefficient or convective mode;

m_i = impulsive mass of water;

m_w = mass of tank wall;

m_t = mass of roof slab; and

g = acceleration due to gravity.

4.6.2 *Elevated Tank*

Base shear in impulsive mode, just above the base of staging (that is, at the top of footing of staging) is given by:

$$V_i = (A_h)_i\, (m_i + m_s)\, g$$

and base shear in convective mode is given by:

$$V_c = (A_h)_c\, m_c\, g$$

where

m_s = mass of container and one-third mass of staging.

4.6.3 Total base shear V can be obtained by combining the base shear in impulsive and convective mode through square root of sum of squares (SRSS) rule and is given as follows:

$$V = \sqrt{V_i^2 + V_c^2}$$

4.7 Base Moment

4.7.1 *Ground Supported Tank*

4.7.1.1 Bending moment in impulsive mode, at the bottom of wall is given by:

$$M_i = (A_h)_i\, (m_i\, h_i + m_w\, h_w + m_t\, h_t)g$$

and bending moment in convective mode is given by:

$$M_c = (A_h)_c\, m_c\, h_c\, g$$

where

h_w = height of centre of gravity of wall mass; and

h_t = height of centre of gravity of roof mass.

4.7.1.2 Overturning moment in impulsive mode to be used for checking the tank stability at the bottom of base slab/plate is given by:

$$M_i^* = (A_h)_i\, [m_i(h_i^* + t_b) + m_w(h_w + t_b) + m_t(h_t + t_b) + m_b t_b/2]\ g$$

and overturning moment in convective mode is given by:

$$M_c^* = (A_h)_c\, m_c\, (h_c^* + t_b)\, g$$

where

M_b = mass of base slab/plate; and

t_b = thickness of base slab/plate.

4.7.2 *Elevated Tank*

Overturning moment in impulsive mode, at the base of the staging is given by:

$$M_i^* = (A_h)_i\, \left[m_i(h_i^* + h_s) + m_s\, h_{cg}\right] g$$

and overturning moment in convective mode is given by:

$$M_c^* = (A_h)_c\, m_c\, (h_{cg} + h_s)\, g$$

where

h_s = structural height of staging, measured from top of footing of staging to the bottom of tank wall; and

h_{cg} = height of centre of gravity of the empty container of elevated tank, measured from the top of footing.

4.7.3 Total moment shall be obtained by combining the moment in impulsive and convective modes under **4.7.1** and **4.7.2** through square of sum of squares (SRSS) and is given as follows:

$$M = \sqrt{(M_i^2 + M_c^2)} \quad \text{and}$$

$$M^* = \sqrt{(M_i^{*2} + M_c^{*2})}$$

4.7.4 For elevated tanks, the design shall be worked out for tank empty and tank full conditions.

4.8 Direction of Seismic Force

4.8.1 Ground supported rectangular tanks shall be analyzed for horizontal earthquake force acting non-concurrently along each of the horizontal axis of the tank for evaluating forces on tank walls.

4.8.2 For elevated tanks, staging components should be designed for the critical direction of seismic force. Different components of staging may have different critical directions.

4.8.3 As an alternative to **4.8.2**, staging components can be designed for either of the following load combination rules:

a) 100 percent + 30 percent rule:

$\pm EL_x \pm 0.3\, EL_y$ and $\pm 0.3\, EL_x \pm EL_y$

b) SRSS Rule:

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

where

EL_x = response quantity due to earthquake load applied in x-direction; and

EL_y = response quantity due to earthquake load applied in y-direction.

4.9 Hydrodynamic Pressure

During lateral base excitation, tank wall is subjected to lateral hydrodynamic pressure and tank base is subjected to hydrodynamic pressure in vertical direction.

4.9.1 *Impulsive Hydrodynamic Pressure*

The impulsive hydrodynamic pressure exerted by the liquid on the tank wall and base is given by:

a) For Circular Tank (*see* Fig. 8a):

Lateral hydrodynamic impulsive pressure on the wall, p_{iw}, is given by:

$$p_{iw} = Q_{iw}(y)\,(A_h)_i\,\rho\,g\,h\cos\phi$$

$$Q_{iw}(y) = 0.866\left[1-(y/h)^2\right]\tanh\left(0.866\frac{D}{h}\right)$$

where

ρ = mass density of liquid;

ϕ = circumferential angle; and

y = vertical distance of a point on tank wall from the bottom of tank wall.

Coefficient of impulsive hydrodynamic pressure on wall, $Q_{iw}(y)$ can also be obtained from Fig. 9a.

Impulsive hydrodynamic pressure in vertical direction, on base slab (y = 0) on a strip of length l', is given by:

$$P_{ib} = 0.866(A_h)_i\rho g h\frac{\sinh\left(1.732\dfrac{x}{h}\right)}{\cosh\left(0.866\dfrac{l'}{h}\right)}$$

x = horizontal distance of a point on base of tank in the direction of seismic force, from the centre of tank.

b) For Rectangular Tank (*see* Fig. 8b):

Lateral hydrodynamic impulsive pressure on wall p_{iw}, is given by:

$$p_{iw} = Q_{iw}(y)\,(A_h)_i\,\rho\,g\,h$$

where

$Q_{iw}(y)$ is same as that for a circular tank and can be read from Fig. 9a, with h/L being used in place of h/D.

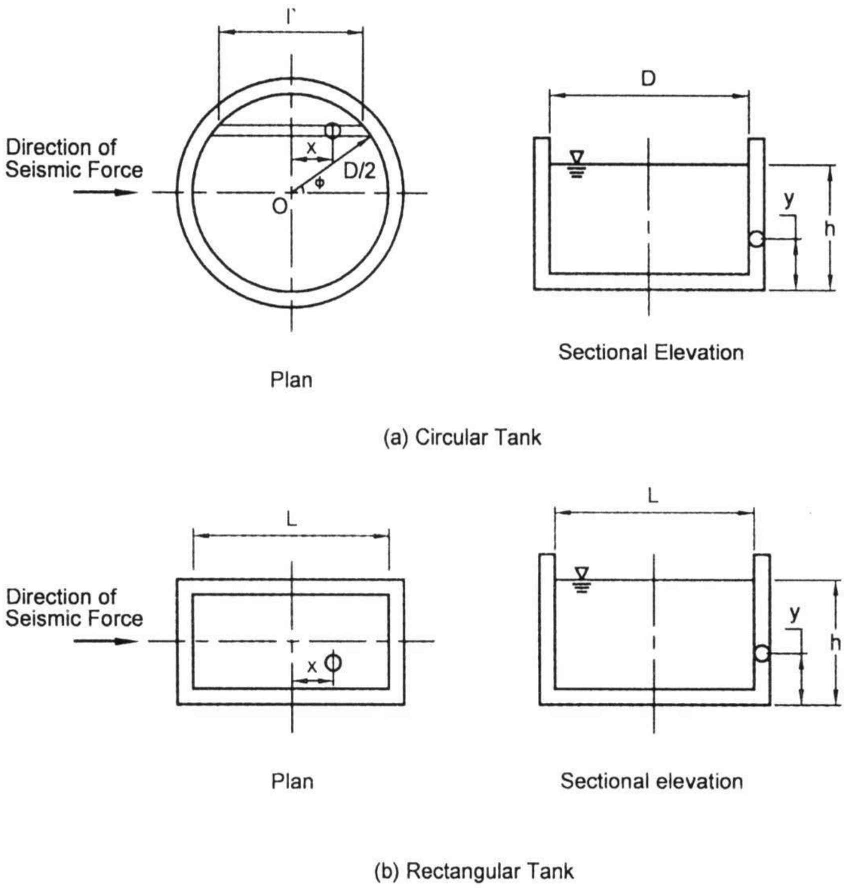


FIG. 8 GEOMETRY OF (a) CIRCULAR TANK AND (b) RECTANGULAR TANK

Impulsive hydrodynamic pressure in vertical direction, on the base slab ($y = 0$), is given by:

$$P_{ib} = Q_{ib}(x)\,(A_h)_i\,\rho\,g\,h$$

$$Q_{ib}(x) = \frac{\sinh\left(1.732\frac{x}{h}\right)}{\cosh\left(0.866\frac{L}{h}\right)}$$

The value of coefficient of impulsive hydrodynamic pressure on base $Q_{ib}(x)$, can also be read from Fig. 9b.

4.9.2 *Convective Hydrodynamic Pressure*

The convective pressure exerted by the oscillating liquid on the tank wall and base shall be calculated as follows:

a) Circular Tank (*see* Fig. 8a)

Lateral convective pressure on the wall p_{cw}, is given by:

$$P_{cw} = Q_{cw}(y)\,(A_h)_c\,\rho\,g\,h\,D\left(1-\frac{1}{3}\cos^2\phi\right)\cos\phi$$

$$Q_{cw}(y) = 0.562\,5\frac{\cosh\left(3.674\frac{y}{D}\right)}{\cosh\left(3.674\frac{h}{D}\right)}$$

The value of $Q_{cw}(y)$ can also be read from Fig. 10a.

Convective pressure in vertical direction, on the base slab ($y = 0$) is given by:

$$P_{cb} = Q_{cb}(x)\,(A_h)_c\,\rho\,g\,D$$

where

$$Q_{cb}(x) = 1.125\left(\frac{x}{D}-\frac{4}{3}\left(\frac{x}{D}\right)^3\right)\mathrm{sech}\left(0.367\,4\frac{h}{D}\right)$$

The value of $Q_{cb}(x)$ may also be read from Fig. 10b.

b) Rectangular Tank (*see* Fig. 8b):

The hydrodynamic pressure on the wall p_{cw}, is given by:

$$p_{cw} = Q_{cw}(y)\,(A_h)_c\,\rho\,g\,L$$

$$Q_{cw}(y) = 0.4165\,[\cosh(3.162\,y/L)/\cosh(3.162\,h/L)]$$

The value of $Q_{cw}(y)$ can also be obtained from Fig. 11a.

The pressure on the base slab ($y = 0$) is given by:

$$P_{cb} = Q_{cb}(x)\,(A_h)_c\,\rho\,g\,L$$

$$Q_{cb}(x) = 1.125\left(\frac{x}{L}-\frac{4}{3}\left(\frac{x}{L}\right)^3\right)\mathrm{sech}\left(3.162\frac{h}{L}\right)$$

where

The value of $Q_{cb}(x)$ can also be obtained from Fig. 11b.

4.9.3 In circular tanks, hydrodynamic pressure due to horizontal excitation varies around the circumference of the tank. However, for convenience in stress analysis of the tank wall, the hydrodynamic pressure on the tank wall may be approximated by an outward pressure distribution of intensity equal to that of the maximum hydrodynamic pressure (*see* Fig. 12a).

4.9.4 Hydrodynamic pressure due to horizontal excitation has curvilinear variation along wall height. However, in the absence of more exact analysis, an equivalent linear pressure distribution may be assumed so as to give the same base shear and bending moment at the bottom of tank wall (*see* Figs. 12b and 12c). The following expressions shall be used to linearise the pressure distribution:

For circular tanks : $q_i = \dfrac{(A_h)_i m_i}{\pi D/2} g$ and $q_c = \dfrac{(A_h)_c m_c}{\pi D/2} g$

For rectangular tanks :

$$q_i = \frac{(A_h)_i m_i}{2B} g \text{ and } q_c = \frac{(A_h)_c m_c}{2B} g$$

$$a_i = \frac{q_i}{h^2}(4h-6h_i),\ b_i = \frac{q_i}{h^2}(6h_i-2h) \text{ and}$$

$$a_c = \frac{q_c}{h^2}(4h-6h_c),\ b_c = \frac{q_c}{h^2}(6h_c-2h)$$

4.9.5 *Pressure Due to Wall Inertia*

Pressure on tank wall due to its inertia is given by:

$$P_{ww} = (A_h)_i\,t\,\rho_m\,g$$

where

ρ_m = mass density of tank wall; and

t = wall thickness.

4.10 Effect of Vertical Ground Acceleration

Due to vertical ground acceleration, effective weight of liquid increases, this induces additional pressure on tank wall, whose distribution is similar to that of hydrostatic pressure.

4.10.1 Hydrodynamic pressure on tank wall due to vertical ground acceleration may be taken as:

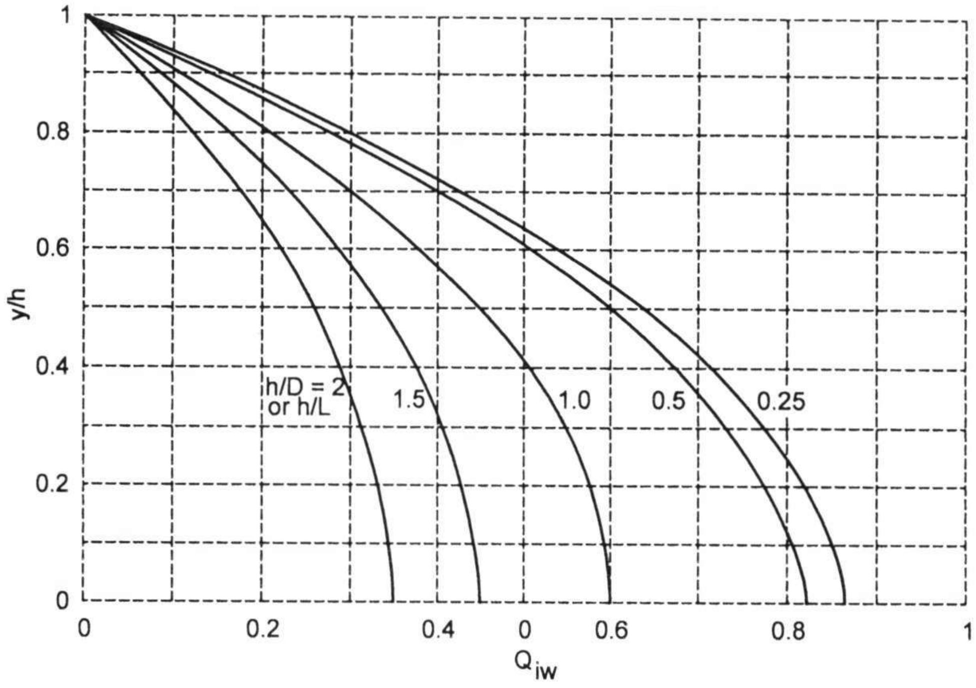
$$P_v = (A_v)\,\rho\,g\,h\,(1-y/h)$$

$$A_v = \frac{2}{3}\left(\frac{Z}{2}\times\frac{I}{R}\times\frac{S_a}{g}\right)$$

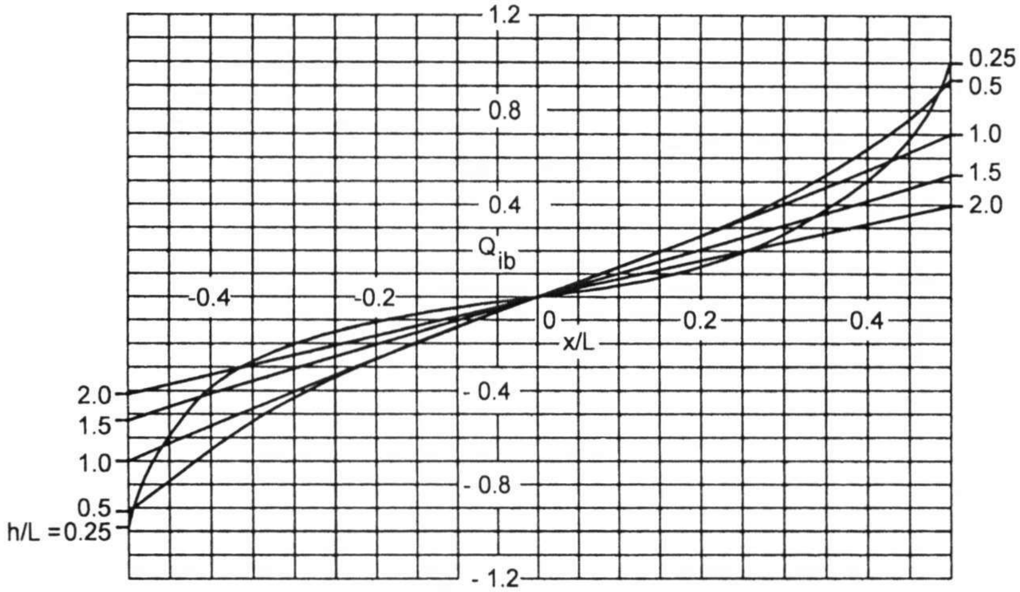
where

y = vertical distance of point under consideration from bottom of tank wall, and

(S_a/g) = Average response acceleration coefficient given by Fig. 2 and Table 3 of IS 1893 (Part 1) and subject to **4.5.2** and **4.5.3**.

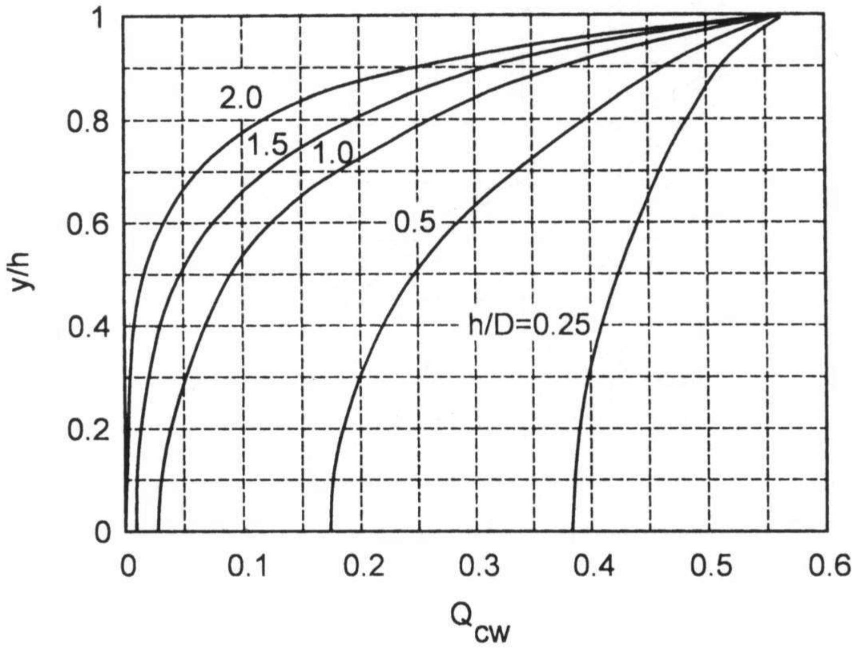


(a) On Wall of Circular and Rectangular Tank

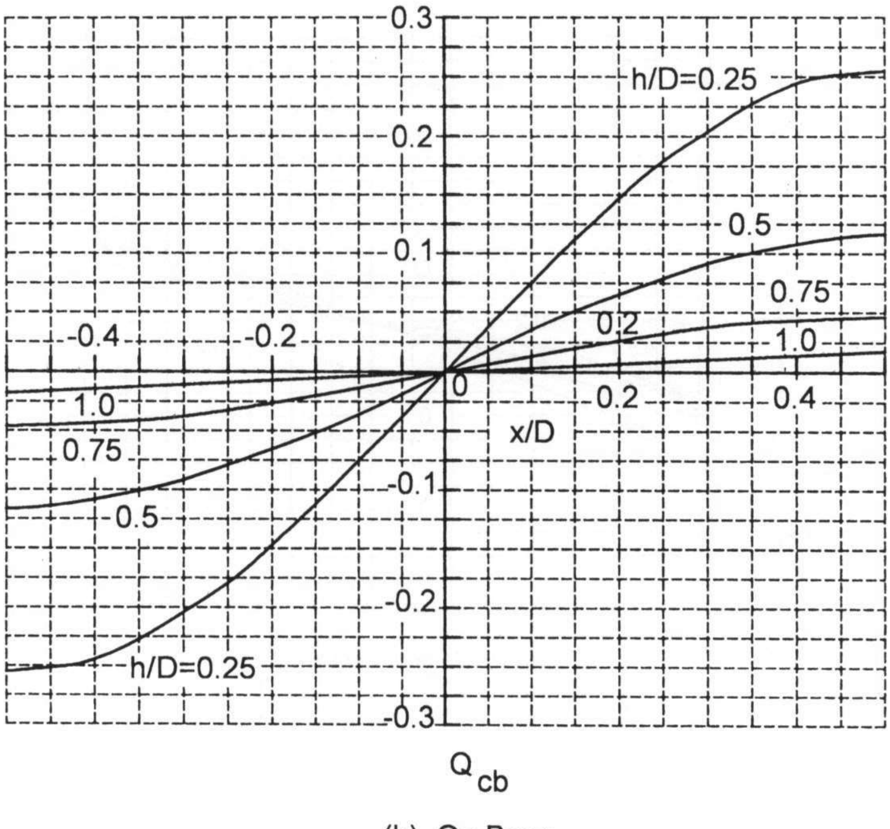


(b) On Base of Rectangular Tank

FIG. 9 IMPULSIVE PRESSURE COEFFICIENT (a) ON WALL (C) (b) ON BASE

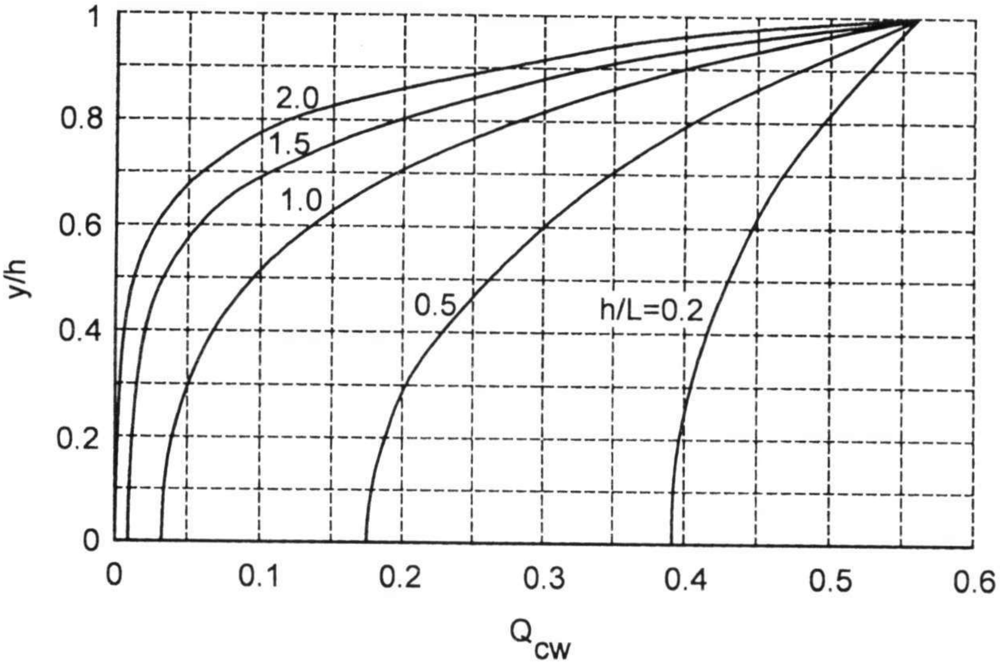


(a) On Wall

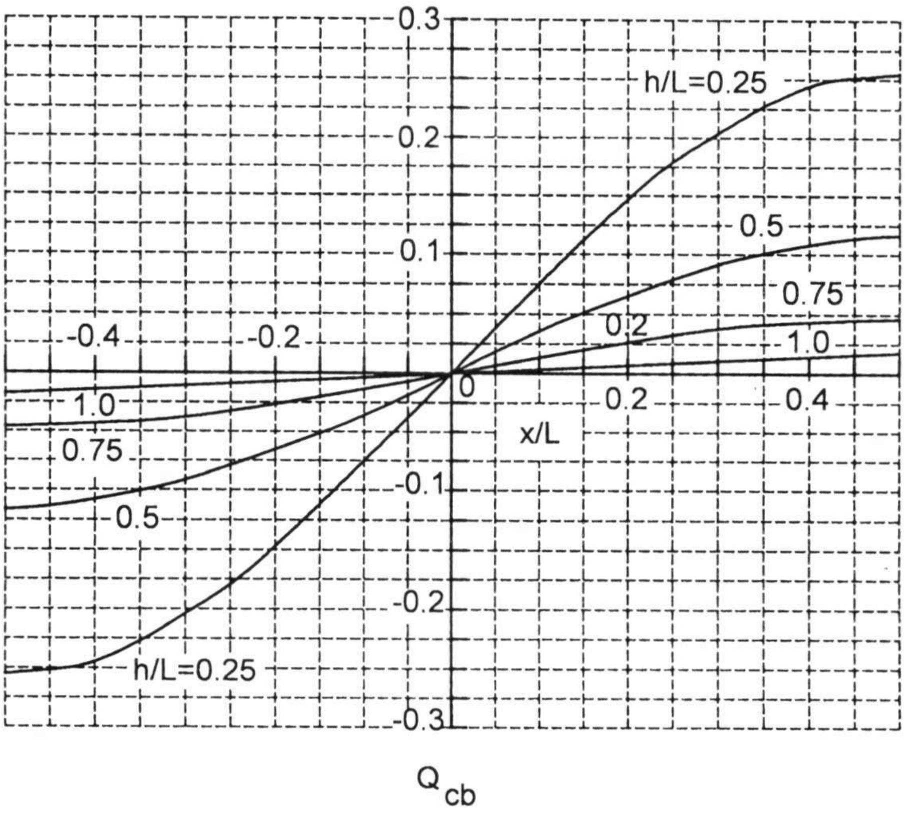


(b) On Base

FIG. 10 CONVECTIVE PRESSURE COEFFICIENT FOR CIRCULAR TANK
(a) ON WALL (b) ON BASE

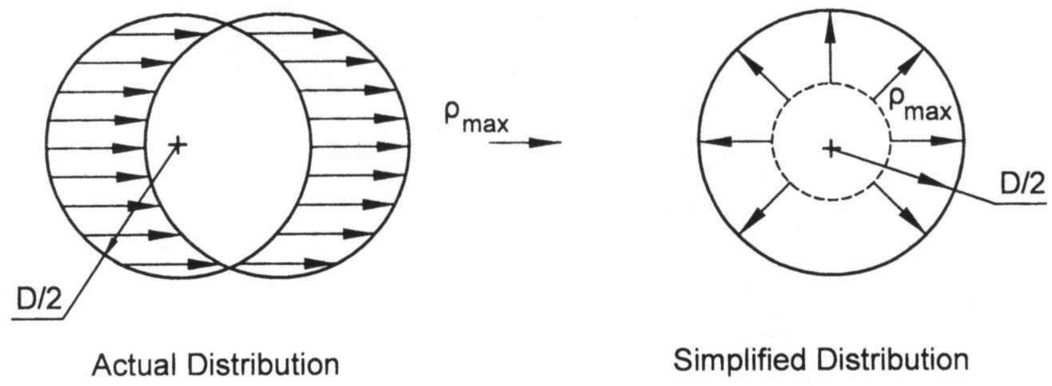


(a) On Wall

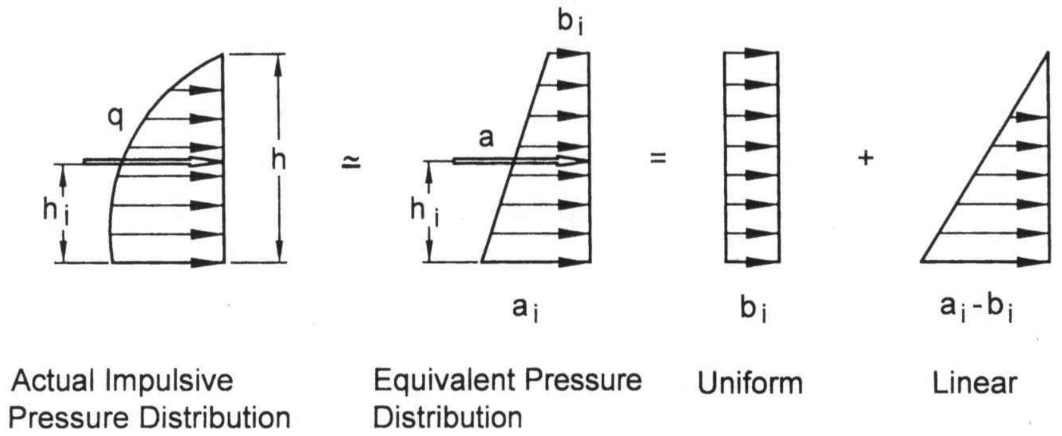


(b) On Base

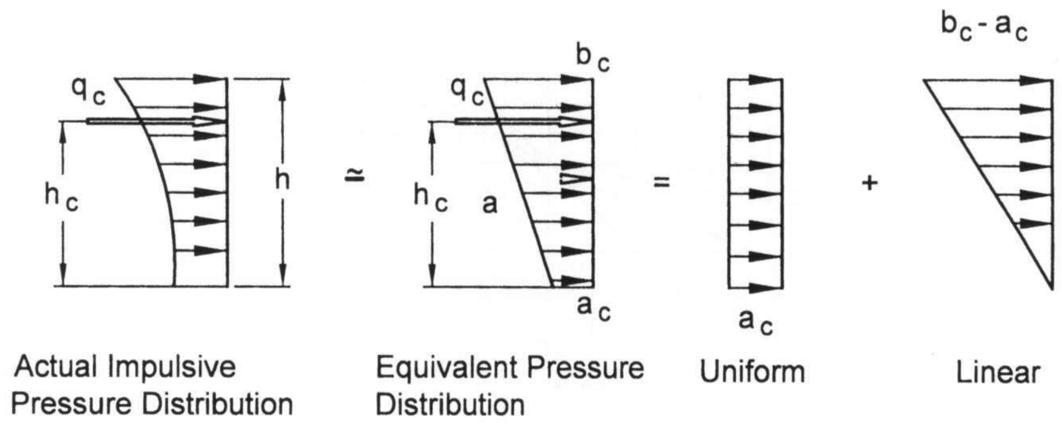
FIG. 11 CONVECTIVE PRESSURE COEFFICIENT FOR RECTANGULAR TANK
(a) ON WALL (b) ON BASE



(a) Simplified Pressure Distribution in Circumferential Direction on Tank Wall



(b) Equivalent Linear Distribution Along Wall Height for Impulsive Pressure



(c) Equivalent Linear Distribution Along Wall Height for Convective Pressure

FIG. 12 HYDRODYNAMIC PRESSURE DISTRIBUTION FOR WALL ANALYSIS
(a) ON WALL (b) ON BASE

In absence of more refined analysis, time period of vertical mode of vibration for all types of tank may be taken as 0.3 s.

4.10.2 The maximum value of hydrodynamic pressure should be obtained by combining pressure due to horizontal and vertical excitation through square root of sum of squares (SRSS) rule, which can be given as:

$$P = \sqrt{(P_{iw} + P_{ww})^2 + P_{cw}^2 + P_v^2}$$

4.11 Sloshing Wave Height

Maximum sloshing wave height is given by:

a) For circular tank:

$$d_{max} = (A_h)_c \ R\frac{D}{2}$$

b) For rectangular tank:

$$d_{max} = (A_h)_c \ R\frac{L}{2}$$

where

$(A_h)_c$ = design horizontal seismic coefficient corresponding to convective time period.

4.12 Anchorage Requirement

Circular ground supported tanks shall be anchored to their foundation (*see* Fig. 13) when

$$\frac{h}{D} > \frac{1}{(A_h)_i}$$

In case of rectangular tank, the same expression may be used with L instead of D.

5 SEISMIC DESIGN OF LIQUID RETAINING TANKS

5.1 Two Mass Idealization

The rational method of analysis using the two masses – impulsive and convective – as presented in **4.1** to **4.12** may be used for determining the seismic design forces on ground supported as well as elevated water tanks of any capacity and material of construction namely, reinforced concrete or steel.

5.2 One Mass Approximation

In the light of the on going practice for construction of large number of water tanks, it is considered expedient to permit the option of one mass idealization, in certain cases, as stated here below, in which the whole water mass is taken as if in impulsive mode.

5.2.1 Ground supported or elevated liquid retaining RC structure of up to and including 1 000 kl capacity, wall of the container if in concrete, which can be regarded as rigid.

5.2.2 Wall in steel may not be regarded as rigid, hence for design of steel tanks by one mass model, the capacity should not exceed 200 kl and h/D or h/L should be 0.4 or higher.

5.2.3 For one mass model, water mass in convective mode shall not be considered. Total water mass shall be assumed in impulsive mode and the impulsive force

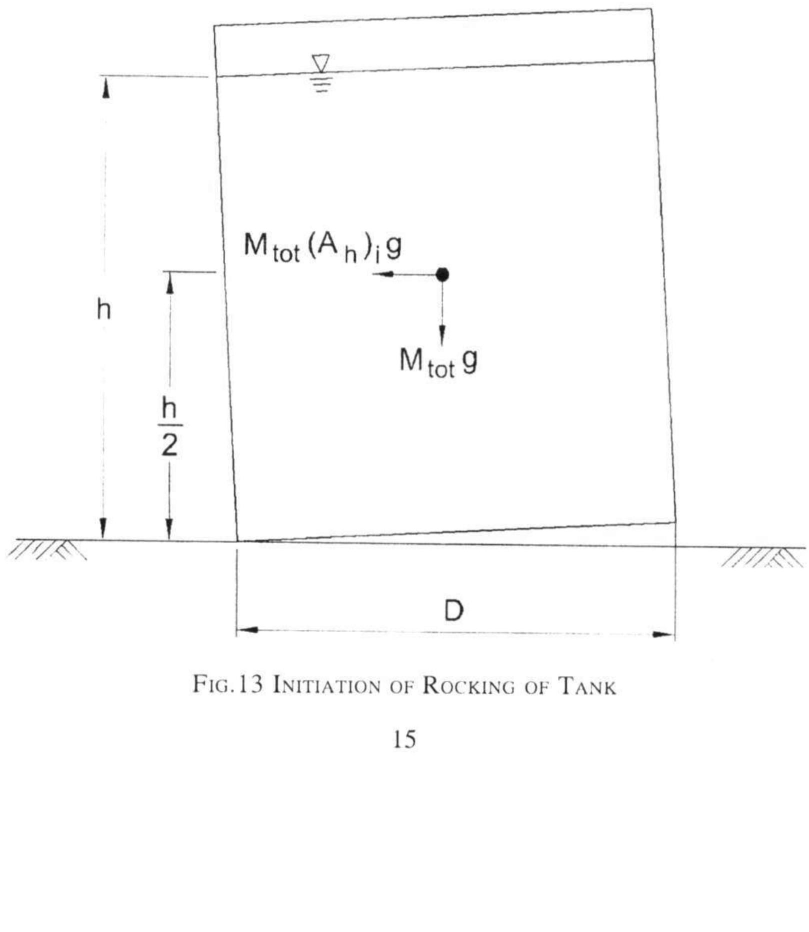


FIG. 13 INITIATION OF ROCKING OF TANK

shall be assumed to act at centre of gravity of the whole water mass.

5.2.4 The design shall be worked out both when the tank is full and when empty. When empty, the weight *W* used in the design shall consist of the dead load of the tank and one-third the weight of the staging lumped at the centre of gravity of the tank. When full, the weight of the fluid contents is to be added to the weight under empty conditions.

6 MISCELLANEOUS

6.1 Piping

Piping system connected to tanks should be given consideration of potential vibration and movement at the pipe joints during earthquakes, and sufficient flexibility should be introduced by proper detailing of pipe joints to avoid de-function. Piping system and its connection to the tank should be designed to comply with the assumptions made and the likely performance; merely neglecting the weight of piping system may not be adequate in all cases.

The piping system shall be designed so as not to impart significant mechanical loading on tank. Local loads at pipe connections can be considered in the design of the tank. Mechanical devices, which add flexibility to piping such as bellows, expansion joints and other special couplings, may be used in the connections.

6.2 Buckling of Shell

Ground supported tanks (particularly, steel tanks) shall be checked for failure against buckling of tank walls under vertical board. Similarly, safety of shaft type of staging of elevated tanks against buckling shall be ensured.

6.3 Buried Tanks

Dynamic earth pressure shall be taken into account while computing the base shear of a partially or fully buried tank. Earth pressure shall also be considered in the design of walls. In buried tanks, dynamic earth pressure shall not be relied upon to reduce dynamic effects due to liquid.

6.4 Shear Transfer

The lateral earthquake force generates shear between wall and base slab and between roof and wall. Wall-to-base slab, wall-to-roof slab and wall-to-wall joints shall be suitably designed to transfer shear forces. Similarly in elevated tanks, connection between container and staging should be suitably designed to transfer the shear force.

6.5 P-Δ Effect

All staging of columns and braces (or beams) for elevated tanks shall be designed for P-Δ effect.

7 AESTHETICS

Elevated water tanks are prominently in public view and visible from near as well as long distances. They often become landmarks on the landscape. It is therefore important that the shape and form of the container and the supporting structure must receive due attention from the point of aesthetics. Innovations in the shape and form should be encouraged when they improve the ambience and enhance the quality of the environment.

Where unusual shapes and forms for supporting structures are used, the designer may use some discretion in choosing the value of response reduction factor *R* consistent with expected seismic performances and ductility. It will be incumbent on the designer, however, to justify the choice of *R* value *vis-à-vis* the seismic safety.

8 QUALITY CONTROL IN REINFORCED CONCRETE TANKS

Quality control in design and constructions are particularly important for elevated tanks in view of several collapses of water tanks during testing. It is necessary that quality of materials and construction tolerances are strictly adhered to during construction phase. Some construction tolerances and detailing are listed below. The information given is not exhaustive and designers and construction engineers are expected to have competence to take adequate measures to ensure required structural performance.

NOTE — The design/construction details for reinforced concrete tanks should strictly follow IS 456, IS 3370 (Parts 1 to 4), and IS 11682. The recommendations are made here to ensure safety under normal as well as service loads.

8.1 RC Frame Staging

8.1.1 *Columns*

a) Minimum size of column should be 400 mm (diameter and/or side of rectangular column) except for tanks having 200 m^3 or less capacity, columns of 300 mm size may be used.
b) Clear height of column between braces should not be more than ten times the size of column.
c) Reinforcement detailing including overlaps in longitudinal bars should follow as shown in IS 13920.
d) During construction and casting of columns, some eccentricity in the verticality of column

may develop. Eccentricity up to 20 mm may be allowed in column between two brace levels. Additional moment due to this eccentricity should be considered in the analysis.

8.1.2 *Braces*

a) Minimum width of unflanged brace shall not be less than 1/30th of its clear length between junctions.

b) In Zones IV and V, use of diagonal bracings in vertical plane shall be encouraged. Information on detailing of RC and steel diagonal bracings is given in IS 11682.

8.1.3 *Foundation*

For isolated footings, tie beam near top of footing shall be provided as per IS 4326.

8.2. RC Shaft Staging

8.2.1 *Thickness of Shaft*

a) Minimum thickness of shaft shall be suitable for constructability which depends on height of formwork for one lift of concrete. Minimum thickness of shaft shall be 150 mm for shaft diameter up to 4 m. For larger diameter shafts, following equations shall be used to arrive at minimum thickness:

1) For shafts with diameter less than 8 m,

$$t_{min} = 150 + (D - 4\,000)/80 \text{ mm}$$

2) For shafts with diameter equal to or greater than 8 m,

$$t_{min} = 200 + (D - 8\,000)/120 \text{ mm}$$

where

D = diameter of shaft, in mm.

b) Additional thickening of shaft and extra vertical and circumferential reinforcement shall be provided at top and bottom level of shaft (that is, at junctions with foundation and with container). This is required to account for secondary moments and eccentricities. Additional vertical and circumferential reinforcement shall be same as that required as per design calculations.

8.2.2 *Reinforcement in Shaft*

a) Minimum vertical reinforcement shall be 0.25 percent of concrete area. The reinforcement shall be provided in two layers. The minimum diameter of vertical bars shall be 10 mm. Maximum centre-to-centre distance between vertical reinforcement in each layer shall not exceed 300 mm.

b) Circumferential reinforcement shall not be less than 0.2 percent of concrete area in vertical section. Since vertical reinforcement is provided in two layers, circumferential reinforcement shall be divided equally in two layers. The spacing of circumferential bars in each layer shall not be more than 300 mm or shell thickness, whichever is less. Circumferential reinforcement shall be placed nearer the faces of shell.

c) At horizontal construction joints in shaft, one additional layer of vertical bars projecting on either side of the joint with L_d anchorage length shall be provided. Continuity of concreting at construction joint shall be done with application of neat cement slurry.

d) *Openings in shaft %* Detailing of shaft at the opening shall take into consideration effective continuity of reinforcement at all sides. More information on detailing near openings is given in IS 11682. At vertical edges of door opening stiffeners may be required.

e) In the tank ring beams, reinforcement bars in direct tension shall have lap length twice the development length in tension. The spliced length of the ring beams in tension shall be enclosed in spirals made of bars not less than 6 mm dia with pitch not more than 100 mm, or enclosed in stirrups of 8 mm dia with pitch not more than 150 mm, the stirrups shall have 135° hooks bent into the core concrete with minimum 50 mm extension. If diameter is more than 22 mm, couplers may be used.

8.2.3 *Construction Control*

a) Vertical Alignment — The centre point of shaft shall not vary from its vertical axis by more than 0.2 percent of shaft height.

b) Over any height of 1.6 m, wall of shaft shall not be out of plumb by more than 10 mm.

c) *Shaft diameter* — The measured centerline diameter of shaft at any section shall not vary from the specified diameter by more than 20 mm plus 0.1 percent of the specified theoretical diameter.

d) *Shaft thickness* — The measured wall thickness shall not vary from the specified wall thickness by more than –5 mm or +10 mm.

8.2.4 *Mat Foundation*

In case of mat foundations, lifting of mat on tension side can be allowed at soil contact. The maximum eccentricity at base may be permitted up to 0.25 times the base diameter provided the maximum compression remains within permissible limits.

8.3 RC Tank and Shaft

a) In the tank ring beams, reinforcement bars in direct tension shall have lap length twice the development length in tension. The spliced length of the ring beams in tension shall be enclosed in spirals made of bars not less than 6 mm diameter with pitch not more than 100 mm, or enclosed in stirrups of 8 mm diameter with pitch not more than 150 mm, the stirrups shall have 135° hooks bent into the core concrete with minimum 50 mm extension.

b) In tank wall or shaft, not more than one-third of vertical bars shall be spliced at any section. For circumferential bars, lap length shall be 1.4 times development length in tension; the laps shall be staggered so that not more than one-third the bars shall be spliced at any one section.

8.4 Strong Column – Weak Beam

For column and beam type of staging of elevated tank, sum of moment of resistance of column at a junction should not be less than 1.1 times the sum of moment of resistance of beams in any one plane taken at a time. This check is to be applied by limit state method.

8.5 Staircase Design

Provisions of IS 11682 shall be followed for the staircase design.

(*Continued from second cover*)

c) Housner, G. W., 1963a, 'Dynamic analysis of fluids in containers subjected to acceleration', Nuclear Reactors and Earthquakes, Report No. TID 7024, U. S. Atomic Energy Commission, Washington D.C.

d) Housner, G. W., 1963b, 'The dynamic behavior of water tanks', Bulletin of Seismological Society of America, Vol. 53, No. 2, 381-387.

e) Jain, S. K. and Medhekar, M. S., 1993, 'Proposed provisions for aseismic design of liquid storage tanks: Part I – Codal provisions', Journal of Structural Engineering, Vol. 20, No. 3, 119-128.

f) Jain, S. K. and Medhekar, M. S., 1994, 'Proposed provisions for a seismic design of liquid storage tanks: Part II – Commentary and examples', Journal of Structural Engineering, Vol. 20, No. 4, 167-175.

g) Jaiswal, O. R. Rai, D. C. and Jain, S.K., 2004a, 'Codal provisions on design seismic forces for liquid storage tanks: a review', Report No. IITK-GSDMA-EQ-01- V1.0, Indian Institute of Technology, Kanpur.

h) Jaiswal, O. R., Rai, D. C. and Jain, S.K., 2004b, 'Codal provisions on seismic analysis of liquid storage tanks: a review' Report No. IITK-GSDMA-EQ-04-V1.0, Indian Institute of Technology, Kanpur.

i) Priestley, M. J. N., et al., 1986, 'Seismic design of storage tanks', Recommendations of a study group of the New Zealand National Society for Earthquake Engineering.

10. Veletsos, A. S., 1984, 'Seismic response and design of liquid storage tanks', Standards for the seismic design of oil and gas pipeline systems, Technical Council on Lifeline Earthquake Engineering, ASCE, N.Y., 255-370, 443-461.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value observed or calculated expressing the result of a test or analysis, shall be round off in the accordance with IS 2 : 1960 Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded value should be the same as that of the specified value in this Standard.

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IS 1893 (Part 1) : 2016

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भाग 1 सामान्य प्रावधान और भवन

(छठा पुनरीक्षण)

Criteria for Earthquake Resistant Design of Structures

Part 1 General Provisions and Buildings

(Sixth Revision)

ICS 91.120.25

भारतीय मानक ब्यूरो
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Earthquake Engineering Sectional Committee, CED 39

FOREWORD

This Indian Standard (Part 1) (Sixth Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Earthquake Engineering Sectional Committee had been approved by the Civil Engineering Division Council.

India is prone to strong earthquake shaking, and hence earthquake resistant design is essential. The Committee has considered an earthquake zoning map based on the maximum intensities at each location as recorded from damage surveys after past earthquakes, taking into account,

a) known magnitudes and the known epicentres (*see* Annex A) assuming all other conditions as being average; and

b) tectonics (*see* Annex B) and lithology (*see* Annex C) of each region.

The Seismic Zone Map (*see* Fig. 1) is broadly associated with 1964 MSK Intensity Scale (*see* Annex D) corresponding to VI (or less), VII, VIII and IX (and above) for Seismic Zones II, III, IV and V, respectively. Seismic Zone Factors for some important towns are given in Annex E.

Structures designed as per this standard are expected to sustain damage during strong earthquake ground shaking. The provisions of this standard are intended for earthquake resistant design of only normal structures (without energy dissipation devices or systems in-built). This standard provides the minimum design force for earthquake resistant design of special structures (such as large and tall buildings, large and high dams, long-span bridges and major industrial projects). Such projects require rigorous, site-specific investigation to arrive at more accurate earthquake hazard assessment.

To control loss of life and property, base isolation or other advanced techniques may be adopted. Currently, the Indian Standard is under formulation for design of such buildings; until the standard becomes available, specialist literature should be consulted for design, detail, installation and maintenance of such buildings.

IS 1893 : 1962 'Recommendations for earthquake resistant design of structures' was first published in 1962, and revised in 1966, 1970, 1975 and 1984. Further, in 2002, the Committee decided to present the provisions for different types of structures in separate parts, to keep abreast with rapid developments and extensive research carried out in earthquake-resistant design of various structures. Thus, IS 1893 was split into five parts. The other parts in the series are:

Part 1 General provisions and buildings

Part 2 Liquid retaining tanks — Elevated and ground supported

Part 3 Bridges and retaining walls

Part 4 Industrial structures, including stack-like structures

Part 5 Dams and embankments (*to be formulated*)

This standard (Part 1) contains general provisions on earthquake hazard assessment applicable to all buildings and structures covered in Parts 2 to 5. Also, Part 1 contains provisions specific to earthquake-resistant design of buildings. Unless stated otherwise, the provisions in Parts 2 to 5 are to be read necessarily in conjunction with the general provisions as laid down in Part 1.

In this revision, the following changes have been included:

a) Design spectra are defined for natural period up to 6 s;

b) Same design response spectra are specified for all buildings, irrespective of the material of construction;

c) Bases of various load combinations to be considered have been made consistent for earthquake effects, with those specified in the other codes;

d) Temporary structures are brought under the purview of this standard;

e) Importance factor provisions have been modified to introduce intermediate importance category of buildings, to acknowledge the density of occupancy of buildings;

f) A provision is introduced to ensure that all buildings are designed for at least a minimum lateral force;

g) Buildings with flat slabs are brought under the purview of this standard;

h) Additional clarity is brought in on how to handle different types of irregularity of structural system;

j) Effect of masonry infill walls has been included in analysis and design of frame buildings;

k) Method is introduced for arriving at the approximate natural period of buildings with basements, step back buildings and buildings on hill slopes;

m) Provisions on torsion have been simplified; and

n) Simplified method is introduced for liquefaction potential analysis.

In the formulation of this standard, effort has been made to coordinate with standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. Assistance has particularly been derived from the following publications:

1) IBC 2015, International Building Code, International Code Council, USA, 2015

2) NEHRP 2009, NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, Report No. FEMA P-750, Federal Emergency Management Agency, Washington, DC, USA, 2009

3) ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, USA, 2010

4) NZS 1170.5: 2004, Structural Design Actions, Part 5: Earthquake Actions – New Zealand, Standards New Zealand, Wellington, New Zealand, 2004

Also, considerable assistance has been given by Indian Institutes of Technology, Jodhpur, Madras, Bombay, Roorkee and Kanpur; Geological Survey of India; India Meteorological Department, National Centre for Seismology (Ministry of Earth Sciences, Govt of India) and several other organizations. Significant improvements have been made to the standard based on findings of a project entitled, ‘Review of Building Codes and Preparation of Commentary and Handbooks’ awarded to IIT Kanpur by the Gujarat State Disaster Management Authority (GSDMA), Gandhinagar, through World Bank finances during 2003-2004.

The units used with the items covered by the symbols shall be consistent throughout this standard, unless specifically noted otherwise.

The composition of the Committee responsible for the formulation of this standard is given in Annex G.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2 : 1960 ‘Rules for rounding off numerical values (*revised*)’. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 1 GENERAL PROVISIONS AND BUILDINGS

(Sixth Revision)

1 SCOPE

1.1 This standard (Part 1) primarily deals with earthquake hazard assessment for earthquake-resistant design of (1) buildings, (2) liquid retaining structures, (3) bridges, (4) embankments and retaining walls, (5) industrial and stack-like structures, and (6) concrete, masonry and earth dams. Also, this standard (Part 1) deals with earthquake-resistant design of buildings; earthquake-resistant design of the other structures is dealt with in Parts 2 to 5.

1.2 All structures, like parking structures, security cabins and ancillary structures need to be designed for appropriate earthquake effects as per this standard.

1.3 Temporary elements, such as scaffolding and temporary excavations, need to be designed as per this standard.

1.4 This standard does not deal with construction features relating to earthquake-resistant buildings and other structures. For guidance on earthquake-resistant construction of buildings, reference may be made to the latest revisions of the following Indian Standards: IS 4326, IS 13827, IS 13828, IS 13920, IS 13935 and IS 15988.

1.5 The provisions of this standard are applicable even to critical and special structures, like nuclear power plants, petroleum refinery plants and large dams. For such structures, additional requirements may be imposed based on special studies, such as site-specific hazard assessment. In such cases, the earthquake effects specified by this standard shall be taken as at least the minimum.

2 REFERENCES

The standards listed below contain provisions, which, through reference in this text, constitute provisions of this standard. At the time of publication, the editions indicated were valid. All standards are subject to revision, and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards indicated below:

IS No.	*Title*
456 : 2000	Code of practice for plain and reinforced concrete (*fourth revision*)
800 : 2007	Code of practice for general construction in steel (*second revision*)
875	Code of practice for design loads (other than earthquake) for buildings and structures:
(Part 1 : 1987)	Dead loads — Unit weights of building material and stored materials (*second revision*)
(Part 2 : 1987)	Imposed loads (*second revision*)
(Part 3 : 2015)	Wind loads (*third revision)*
(Part 4 : 1987)	Snow loads (*second revision)*
(Part 5 : 1987)	Special loads and load combinations (*second revision*)
1343 : 2012	Code of practice for prestressed concrete (*second revision*)
1498 : 1970	Classification and identification of soils for general engineering purposes (*first revision*)
1888 : 1982	Method of load test on soils (*second revision*)
1893	Criteria for earthquake resistant design of structures:
(Part 2) : 2014	Liquid retaining tanks
(Part 3) : 2014	Bridges and retaining walls
(Part 4) : 2015	Industrial structures including stack-like structures (*first revision*)
1905 : 1987	Code of practice for structural use of unreinforced masonry (*third revision*)
2131 : 1981	Method of standard penetration test for soils (*first revision*)
2809 : 1972	Glossary of terms and symbols relating to soil engineering (*first revision*)
2810 : 1979	Glossary of terms relating to soil dynamics (*first revision*)
2974	Code of practice for design and construction of machine foundations:
(Part 1) : 1982	Foundation for reciprocating type machines
(Part 2) : 1980	Foundations for impact type machines (Hammer foundations)
(Part 3) : 1992	Foundations for rotary type machines (Medium and high frequency)
(Part 4) : 1979	Foundations for rotary type machines of low frequency

IS No.	*Title*
(Part 5) : 1987	Foundations for impact machines other than hammer (Forging and stamping press, pig breaker, drop crusher and jolter)
4326 : 2013	Earthquake resistant design and construction of buildings—Code of Practice (*third revision*)
6403 : 1981	Code of practice for determination of bearing capacity of shallow foundations (*first revision*)
13827 : 1993	Improving earthquake resistance of earthen buildings — Guidelines
13828 : 1993	Improving earthquake resistance of low strength masonry buildings — Guidelines
13920 : 2016	Ductile design and detailing of reinforced concrete structures subjected to seismic forces — Code of practice (*first revision*)
13935 : 1993	Repair and seismic strengthening of buildings — Guidelines
15988 : 2013	Seismic evaluation and strengthening of existing reinforced concrete building — Guidelines
SP 7 : 2016 (Part 6/Sec 4)	National Building Code of India: Part 6 Structural Design, Section 4 Masonry

3 TERMINOLOGY

For the purpose of this standard, definitions given below shall apply to all structures, in general. For definition of terms pertaining to soil mechanics and soil dynamics, reference may be made to IS 2809 and IS 2810, and for definition of terms pertaining to 'loads', reference may be made to IS 875 (Parts 1 to 5).

3.1 Closely-Spaced Modes — Closely-spaced modes of a structure are those of the natural modes of oscillation of a structure, whose natural frequencies differ from each other by 10 percent or less of the lower frequency.

3.2 Critical Damping — The damping beyond which the free vibration motion will not be oscillatory.

3.3 Damping — The effect of internal friction, inelasticity of materials, slipping, sliding, etc, in reducing the amplitude of oscillation; it is expressed as a fraction of critical damping (*see* **3.2**).

3.4 Design Acceleration Spectrum — Design acceleration spectrum refers to an average smoothened graph of maximum acceleration as a function of natural frequency or natural period of oscillation for a specified damping ratio for the expected earthquake excitations at the base of a single degree of freedom system.

3.5 Design Horizontal Acceleration Coefficient (A_h) — It is a horizontal acceleration coefficient that shall be used for design of structures.

3.6 Design Horizontal Force — It is the horizontal seismic force prescribed by this standard that shall be used to design a structure.

3.7 Ductility — It is the capacity of a structure (or its members) to undergo large inelastic deformations without significant loss of strength or stiffness.

3.8 Epicentre — It is the geographical point on the surface of earth vertically above the point of origin of the earthquake.

3.9 Floor Response Spectrum — It is the response spectrum (for a chosen material damping value) of the time history of the shaking generated at a floor of a structure, when the structure is subjected to a given earthquake ground motion at its base.

3.10 Importance Factor *(I)* — It is a factor used to estimate design seismic force depending on the functional use of the structure, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance.

3.11 Intensity of Earthquake — It is the measure of the strength of ground shaking manifested at a place during the earthquake, and is indicated by a roman capital numeral on the MSK scale of seismic intensity (*see* Annex D).

3.12 Liquefaction — It is a state primarily in saturated cohesionless soils wherein the effective shear strength is reduced to negligible value for all engineering purposes, when the pore pressure approaches the total confining pressure during earthquake shaking. In this condition, the soil tends to behave like a fluid mass (*see* Annex F).

3.13 Lithological Features — Features that reflect the nature of the geological formation of the earth's crust above bed rock characterized on the basis of structure, mineralogical composition and grain size.

3.14 Modal Mass (M_k) in Mode (k) of a Structure — It is a part of the total seismic mass of the structure that is effective in natural mode k of oscillation during horizontal or vertical ground motion.

3.15 Modal Participation Factor (P_k) in Mode (k) of a Structure — The amount by which natural mode k contributes to overall oscillation of the structure during horizontal or vertical earthquake ground motion. Since the amplitudes of mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for defining mode shapes.

3.16 Modes of Oscillation — *See* **3.19**.

3.17 Mode Shape Coefficient (ϕ_{ik}) — It is the spatial deformation pattern of oscillation along degree of freedom i, when the structure is oscillating in its natural mode k. A structure with N degrees of freedom possesses N natural periods and N associated natural mode shapes. These natural mode shapes are together presented in the form of a mode shape matrix [ϕ], in which each column represents one natural mode shape. The element ϕ_{ik} is called the mode shape coefficient associated with degree of freedom i, when the structure is oscillating in mode k.

3.18 Natural Period (T_k) in Mode (k) of Oscillation — The time taken (in second) by the structure to complete one cycle of oscillation in its natural mode k of oscillation.

3.18.1 *Fundamental Lateral Translational Natural Period* (T_1) — It is the longest time taken (in second) by the structure to complete one cycle of oscillation in its lateral translational mode of oscillation in the considered direction of earthquake shaking. This mode of oscillation is called the fundamental lateral translational natural mode of oscillation. A three-dimensional model of a structure will have one such fundamental lateral translational mode of oscillation along each of the two orthogonal plan directions.

3.19 Normal Mode of Oscillation — The mode of oscillation in which there are special undamped free oscillations in which all points on the structure oscillate harmonically at the same frequency (or period), such that all these points reach their individual maximum responses simultaneously.

3.20 Peak Ground Acceleration — It is the maximum acceleration of the ground in a given direction of ground shaking. Here, the acceleration refers to that of the horizontal motion, unless specified otherwise.

3.21 Response Reduction Factor (R) — It is the factor by which the base shear induced in a structure, if it were to remain elastic, is reduced to obtain the design base shear. It depends on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations, redundancy in the structure, or overstrength inherent in the design process.

3.22 Response Spectrum — It is the representation of maximum responses of a spectrum of idealized single degree freedom systems of different natural periods but having the same damping, under the action of the same earthquake ground motion at their bases. The response referred to here can be maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

3.23 Response Acceleration Coefficient of a Structure (S_a/g) — It is a factor denoting the normalized design acceleration spectrum value to be considered for the design of structures subjected to earthquake ground shaking; this value depends on the natural period of oscillation of the structure and damping to be considered in the design of the structure.

3.24 Seismic Mass of a Floor — It is the seismic weight of the floor divided by acceleration due to gravity.

3.25 Seismic Mass of a Structure — It is the seismic weight of a structure divided by acceleration due to gravity.

3.26 Seismic Weight of a Floor (W) — It is the sum of dead load of the floor, appropriate contributions of weights of columns, walls and any other permanent elements from the storeys above and below, finishes and services, and appropriate amounts of specified imposed load on the floor.

3.27 Seismic Weight of a Structure (W) — It is the sum of seismic weights of all floors.

3.28 Seismic Zone Factor (Z) — It is the value of peak ground acceleration considered by this standard for the design of structures located in each seismic zone.

3.29 Time History Analysis — It is an analysis of the dynamic response of the structure at each instant of time, when its base is subjected to a specific ground motion time history.

4 SPECIAL TERMINOLOGY FOR BUILDINGS

4.1 The definitions given below shall apply for the purpose of earthquake resistant design of buildings, as enumerated in this standard.

4.2 Base — It is the level at which inertia forces generated in the building are considered to be transferred to the ground through the foundation. For buildings with basements, it is considered at the bottommost basement level. For buildings resting on,

a) pile foundations, it is considered to be at the top of pile cap;

b) raft, it is considered to be at the top of raft; and

c) footings, it is considered to be at the top of the footing.

For buildings with combined types of foundation, the base is considered as the bottom-most level of the bases of the constituent individual foundations as per definitions above.

4.3 Base Dimension (d) — It is the dimension (in metre) of the base of the building along a direction of shaking.

4.4 Centre of Mass (CM) — The point in the floor of a building through which the resultant of the inertia force of the floor is considered to act during earthquake shaking. Unless otherwise stated, the inertia force considered is that associated with the horizontal shaking of the building.

4.5 Centre of Resistance (*CR*)

4.5.1 *For Single Storey Buildings* — It is the point on the roof of a building through which when the resultant internal resistance acts, the building undergoes,

a) pure translation in the horizontal direction; and

b) no twist about vertical axis passing through the *CR*.

4.5.2 *For Multi-Storey Buildings* — It is the set of points on the horizontal floors of a multi-storey building through which, when the resultant incremental internal resistances across those floors act, all floors of the building undergo,

a) pure translation in the horizontal direction; and

b) no twist about vertical axis passing through the *CR*.

4.6 Eccentricity

4.6.1 *Design Eccentricity* (**e_{di}**) — It is the value of eccentricity to be used for floor *i* in calculations of design torsion effects.

4.6.2 *Static Eccentricity* (**e_{si}**) — It is the distance between centre of mass (*CM*) and centre of resistance (*CR*) of floor *i*.

4.7 Design Seismic Base Shear (V_B) — It is the horizontal lateral force in the considered direction of earthquake shaking that the structure shall be designed for.

4.8 Diaphragm — It is a horizontal or nearly horizontal structural system (for example, reinforced concrete floors and horizontal bracing systems), which transmits lateral forces to vertical elements connected to it.

4.9 Height of Floor (h_i) — It is the difference in vertical elevations (in metre) of the base of the building and top of floor *i* of the building.

4.10 Height of Building (*h*) — It is the height of building (in metre) from its base to top of roof level,

a) excluding the height of basement storeys, if basement walls are connected with the ground floor slab or basement walls are fitted between the building columns, but

b) including the height of basement storeys, if basement walls are not connected with the ground floor slab and basement walls are not fitted between the building columns.

In step-back buildings, it shall be taken as the average of heights of all steps from the base, weighted with their corresponding floor areas. And, in buildings founded on hill slopes, it shall be taken as the height of the roof from the top of the highest footing level or pile cap level.

4.11 Horizontal Bracing System — It is a horizontal truss system that serves the same function as a diaphragm.

4.12 Joints — These are portions of columns that are common to beams/braces and columns, which frame into columns.

4.13 Lateral Force Resisting System — It is part of the structural system, and consists of all structural members that resist lateral inertia forces induced in the building during earthquake shaking.

4.14 Moment-Resisting Frame — It is an assembly of beams and columns that resist induced and externally applied forces primarily by flexure.

4.14.1 *Ordinary Moment-Resisting Frame (OMRF)* — It is a moment-resisting frame designed and detailed as per IS 456 or IS 800, but not meeting special detailing requirements for ductile behaviour as per IS 13920 or IS 800, respectively.

4.14.2 *Special Moment-Resisting Frame (SMRF)* — It is a moment-resisting frame designed and detailed as per IS 456 or IS 800, and meeting special detailing requirements for ductile behaviour as per IS 13920 or IS 800, respectively.

4.15 Number of Storeys (*n*) — It is the number of levels of a building above the base at which mass is present in substantive amounts. This,

a) excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns; and

b) includes the basement storeys, when they are not so connected.

4.16 Core Structural Walls, Perimeter Columns, Outriggers and Belt Truss System — It is a structural system comprising of a core of structural walls and perimeter columns, resisting the vertical and lateral loads, with

a) the core structural walls connected to select perimeter column element(s) (often termed outrigged columns) by deep beam elements, known as outriggers, at discrete locations along the height of the building; and

b) the outrigged columns connected by deep beam elements (often known as belt truss), typically at the same level as the outrigger elements.

A structure with this structural system has enhanced lateral stiffness, wherein core structural walls and perimeter columns are mobilized to act with each other through the outriggers, and the perimeter columns themselves through the belt truss. The global lateral stiffness is sensitive to: flexural stiffness of the core element, the flexural stiffness of the outrigger element(s), the axial stiffness of the outrigged column(s), and the flexural stiffness of the outrigger elements connecting the core structural walls to the perimeter columns.

4.17 Principal Plan Axes — These are two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

4.18 P-Δ Effect — It is the secondary effect on shear forces and bending moments of lateral force resisting elements generated under the action of the vertical loads, interacting with the lateral displacement of building resulting from seismic effects.

4.19 RC Structural Wall — It is a wall designed to resist lateral forces acting in its own plane.

4.19.1 *Ordinary RC Structural Wall* — It is a reinforced concrete (RC) structural wall designed and detailed as per IS 456, but not meeting special detailing requirements for ductile behaviour as per IS 13920.

4.19.2 *Special RC Structural Wall* — It is a RC structural wall designed and detailed as per IS 13920, and meeting special detailing requirements for ductile behaviour as per IS 13920.

4.20 Storey — It is the space between two adjacent floors.

4.20.1 *Soft Storey* — It is one in which the lateral stiffness is less than that in the storey above. The storey lateral stiffness is the total stiffness of all seismic force resisting elements resisting lateral earthquake shaking effects in the considered direction.

4.20.2 *Weak Storey* — It is one in which the storey lateral strength [cumulative design shear strength of all structural members other than that of unreinforced masonry (URM) infills] is less than that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the lateral storey shear in the considered direction.

4.21 Storey Drift — It is the relative displacement between the floors above and/or below the storey under consideration.

4.22 Storey Shear (V_i) — It is the sum of design lateral forces at all levels above the storey *i* under consideration.

4.23 Storey Lateral Shear Strength (S_i) — It is the total lateral strength of all lateral force resisting elements in the storey considered in a principal plan direction of the building.

4.24 Storey Lateral Translational Stiffness (K_i) — It is the total lateral translational stiffness of all lateral force resisting elements in the storey considered in a principal plan direction of the building.

4.25 RC Structural Wall Plan Density (ρ_{sw}) — It is the ratio of the cross-sectional area at the plinth level of RC structural walls resisting the lateral load and the plinth of the building, expressed as a percentage.

5 SYMBOLS

The symbols and notations given below apply to the provisions of this standard:

A_h	Design horizontal earthquake acceleration coefficient
A_k	Design horizontal earthquake acceleration spectrum value for mode *k* of oscillation
b_i	Plan dimension of floor *i* of the building, perpendicular to direction of earthquake shaking
C	Index for the closely-spaced modes
d	Base dimension (in metre) of the building in the direction in which the earthquake shaking is considered
DL	Response quantity due to dead load
e_{di}	Design eccentricity to be used at floor *i* calculated as per **7.8.2**
e_{si}	Static eccentricity at floor *i* defined as the distance between centre of mass and centre of resistance
EL_X	Response quantity due to earthquake load for horizontal shaking along *X*-direction
EL_Y	Response quantity due to earthquake load for horizontal shaking along *Y*-direction
EL_Z	Response quantity due to earthquake load for horizontal shaking along *Z*-direction
F_{roof}	Design lateral forces at the roof due to all modes considered
F_i	Design lateral forces at the floor *i* due to all modes considered
g	Acceleration due to gravity
h	Height (in metre) of structure
h_i	Height measured from the base of the building to floor *i*
I	Importance factor
IL	Response quantity due to imposed load
K_i	Lateral translational stiffness of storey *i*

L	Dimension of a building in a considered direction
M_k	Modal mass of mode k
n	Number of storeys or floors
N	Corrected SPT value for soil
N_m	Number of modes to be considered as per **7.7.5.2**
P_k	Mode participation factor of mode k
Q_i	Lateral force at floor i
Q_{ik}	Design lateral force at floor i in mode k
R	Response reduction factor
S_a/g	Design / Response acceleration coefficient for rock or soil sites as given by Fig. 2 and **6.4.2** based on appropriate natural period
S_i	Lateral shear strength of storey i
T	Undamped natural period of oscillation of the structure (in second)
T_a	Approximate fundamental period (in second)
T_k	Undamped natural period of mode k of oscillation (in second)
T_1	Fundamental natural period of oscillation (in second)
V_B	Design seismic base shear
$\bar{V}_B$	Design base shear calculated using the approximate fundamental period T_a
V_i	Peak storey shear force in storey i due to all modes considered
V_{ik}	Shear force in storey i in mode k
V_{roof}	Peak storey shear force in the top storey due to all modes considered
W	Seismic weight of the building
W_i	Seismic weight of floor i
Z	Seismic zone factor
ϕ_{ik}	Mode shape coefficient at floor i in mode k
λ	Peak response (for example, member forces, displacements, storey forces, storey shears or base reactions) due to all modes considered
λ_k	Absolute value of maximum response in mode k
λ_c	Absolute value of maximum response in mode c, where mode c is a closely-spaced mode
λ^*	Peak response due to the closely-spaced modes only
ρ_{ji}	Coefficient used in complete quadratic combination (CQC) method while combining responses of modes i and j
ω_i	Circular frequency (in rad/s) in mode i

6 GENERAL PRINCIPLES AND DESIGN CRITERIA

6.1 General Principles

6.1.1 *Ground Motion*

The characteristics (intensity, duration, frequency content, etc) of seismic ground vibrations expected at any site depend on magnitude of earthquake, its focal depth, epicentral distance, characteristics of the path through which the seismic waves travel, and soil strata on which the structure is founded. The random earthquake ground motions, which cause the structure to oscillate, can be resolved in any three mutually perpendicular directions. The predominant direction of ground vibration is usually horizontal.

Effects of earthquake-induced vertical shaking can be significant for overall stability analysis of structures, especially in structures (a) with large spans, and (b) those in which stability is a criterion for design. Reduction in gravity force due to vertical ground motions can be detrimental particularly in prestressed horizontal members, cantilevered members and gravity structures. Hence, special attention shall be paid to effects of vertical ground motion on prestressed or cantilevered beams, girders and slabs.

6.1.2 The response of a structure to ground vibrations depends on (a) type of foundation; (b) materials, form, size and mode of construction of structures; and (c) duration and characteristics of ground motion. This standard specifies design forces for structures founded on rocks or soils, which do not settle, liquefy or slide due to loss of strength during earthquake ground vibrations.

6.1.3 Actual forces that appear on structures during earthquakes are much higher than the design forces specified in the standard. Ductility arising from inelastic material behaviour with appropriate design and detailing, and overstrength resulting from the additional reserve strength in structures over and above the design strength are relied upon for the deficit in actual and design lateral loads. In other words, earthquake resistant design as per this standard relies on inelastic behaviour of structures. But, the maximum ductility that can be realized in structures is limited. Therefore, structures shall be designed for at least the minimum design lateral force specified in this standard.

6.1.4 Members and connections of reinforced and prestressed concrete structures shall be designed (as per IS 456 and IS 1343) such that premature failure does not occur due to shear or bond. Some provisions for appropriate ductile detailing of RC members are given in IS 13920. Members and their connections of steel structures should be so proportioned that high ductility is obtained in the structure, avoiding premature failure due to elastic or inelastic buckling of any type. Some provisions for appropriate ductile detailing of steel members are given in IS 800.

6.1.5 The soil-structure interaction refers to effects of the flexibility of supporting soil-foundation system on the response of structure. Soil-structure interaction may not be considered in the seismic analysis of structures supported on rock or rock-like material at shallow depth.

6.1.6 Equipment and other systems, which are supported at various floor levels of a structure, will be subjected to different motions at their support points. In such cases, it may be necessary to obtain floor response spectra for design of equipment and its supports. For details, reference may be made to IS 1893 (Part 4).

6.1.7 *Additions to Existing Structures*

Additions shall be made to existing structures only as follows:

a) An addition that is structurally independent from an existing structure shall be designed and constructed in accordance with the seismic requirements for new structures.

b) An addition that is structurally connected to an existing structure shall be designed and constructed such that the entire structure conforms to the seismic force resistance requirements for new structures, unless the following three conditions are complied with:

    1) Addition shall comply with the requirements for new structures,
    2) Addition shall not increase the seismic forces in any structural element of the existing structures by more than 5 percent, unless the capacity of the element subject to the increased force is still in compliance with this standard, and
    3) Addition shall not decrease the seismic resistance of any structural element of the existing structure unless reduced resistance is equal to or greater than that required for new structures.

6.1.8 *Change in Occupancy*

When a change of occupancy results in a structure being re-classified to a higher importance factor (I), the structure shall conform to seismic requirements laid down for new structures with the higher importance factor.

6.2 Assumptions

The following assumptions shall be made in the earthquake-resistant design of structures:

a) Earthquake ground motions are complex and irregular, consisting of several frequencies and of varying amplitudes each lasting for a small duration. Therefore, usually, resonance of the type as visualized under steady-state sinusoidal excitations will not occur, as it would need time to build up such amplitudes. But, there are exceptions where resonance-like conditions have been seen to occur between long distance waves and tall structures founded on deep soft soils.

b) Earthquake is not likely to occur simultaneously with high wind, maximum flood or maximum sea waves.

c) The values of elastic modulus of materials, wherever required, will be taken as for static analysis, unless more definite values are available for use in dynamic conditions [*see* IS 456, IS 800, IS 1343, IS 1905 and IS 2974 (Parts 1 to 5)].

6.3 Load Combinations and Increase in Permissible Stresses

6.3.1 *Load Combinations*

The load combinations shall be considered as specified in respective standards due to all load effects mentioned therein. In addition, those specified in this standard shall be applicable, which include earthquake effects.

6.3.1.1 Even when load combinations that do not contain earthquake effects, indicate larger demands than combinations including them, the provisions shall be adopted related to design, ductile detailing and construction relevant for earthquake conditions, which are given in this standard, IS 13920 and IS 800.

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.

6.3.2.2 When lateral load resisting elements are not oriented along mutually orthogonal horizontal directions [as per **7.1** and Table 5(e)], structure shall be designed for the simultaneous effects due to full design earthquake load in one horizontal direction plus 30 percent of design earthquake load along the other horizontal direction. Thus, structure should be designed for the following sets of combinations of earthquake effects:

a) $\pm EL_X \pm 0.3\, EL_Y$, and
b) $\pm 0.3\, EL_X \pm EL_Y$,

where X and Y are two orthogonal horizontal plan directions. Thus, EL in the load combinations given in **6.3.1** shall be replaced by ($EL_X \pm 0.3\ EL_Y$) or ($EL_Y \pm 0.3\ EL_X$). Hence, the sets of load combinations to be considered shall be as given below:

1) $1.2\ [DL + IL \pm (EL_X \pm 0.3\ EL_Y)]$ and $1.2\ [DL + IL \pm (EL_Y \pm 0.3\ EL_X)]$;
2) $1.5\ [DL \pm (EL_X \pm 0.3\ EL_Y)]$ and $1.5\ [DL \pm (EL_Y \pm 0.3\ EL_X)]$; and
3) $0.9\ DL \pm 1.5\ (EL_X \pm 0.3\ EL_Y)$ and $0.9\ DL \pm 1.5\ (EL_Y \pm 0.3\ EL_X)$.

6.3.3 *Design Vertical Earthquake Effects*

6.3.3.1 Effects due to vertical earthquake shaking shall be considered when any of the following conditions apply:

a) Structure is located in Seismic Zone IV or V;
b) Structure has vertical or plan irregularities;
c) Structure is rested on soft soil;
d) Bridges;
e) Structure has long spans; or
f) Structure has large horizontal overhangs of structural members or sub-systems.

6.3.3.2 When effects due to vertical earthquake shaking are to be considered, the design vertical force shall be calculated for vertical ground motion as detailed in **6.4.6**.

6.3.3.3 Where both horizontal and vertical seismic forces are taken into account, load combination specified in **6.3.4** shall be considered.

6.3.4 *Combinations to Account for Three Directional Earthquake Ground Shaking*

6.3.4.1 When responses from the three earthquake components are to be considered, the responses due to each component may be combined using the assumption that when the maximum response from one component occurs, the responses from the other two components are 30 percent each of their maximum. All possible combinations of three components (EL_X, EL_Y and EL_Z) including variations in sign (plus or minus) shall be considered. Thus, the structure should be designed for the following sets of combinations of earthquake load effects:

a) $\pm EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z$,
b) $\pm EL_Y \pm 0.3\ EL_Z \pm 0.3\ EL_X$, and
c) $\pm EL_Z \pm 0.3\ EL_X \pm 0.3\ EL_Y$,

where *X* and *Y* are orthogonal plan directions and *Z* vertical direction. Thus, *EL* in the above referred load combinations shall be replaced by ($EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z$), ($EL_Y \pm 0.3\ EL_Z \pm 0.3\ EL_X$) or ($EL_Z \pm 0.3\ EL_X \pm 0.3\ EL_Y$,). This implies that the sets of load combinations involving earthquake effects to be considered shall be as given below:

1) $1.2\ [\mathrm{DL} + \mathrm{IL} \pm (EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z)]$ and
$1.2\ [\mathrm{DL} + \mathrm{IL} \pm (EL_Y \pm 0.3\ EL_X \pm 0.3\ EL_Z)]$;
2) $1.5\ [\mathrm{DL} \pm (EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z)]$ and
$1.5\ [\mathrm{DL} \pm (EL_Y \pm 0.3\ EL_X \pm 0.3\ EL_Z)]$; and
3) $0.9\ \mathrm{DL} \pm 1.5\ (EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z)$ and
$0.9\ \mathrm{DL} \pm 1.5\ (EL_Y \pm 0.3\ EL_X \pm 0.3\ EL_Z)$.

6.3.4.2 As an alternative to the procedure in **6.3.4.1**, the net response (*EL*) due to the combined effect of the three components can be obtained by:

$$EL = \sqrt{(EL_X)^2 + (EL_Y)^2 + (EL_Z)^2}$$

Caution may be exercised on loss of sign especially of the axial force, shear force and bending moment quantities, when this procedure is used; it can lead to grossly uneconomical design of structures.

6.3.4.3 Procedure for combining shaking effects given by **6.3.4.1** and **6.3.4.2** apply to the same response quantity (say, bending moment in a column about its major axis, or storey shear force in a frame) due to different components of the ground motion.

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.

6.3.5 *Increase in Net Pressure on Soils in Design of Foundations*

6.3.5.1 In the design of foundations, unfactored loads shall be combined in line with IS 2974, while assessing the bearing pressure in soils.

6.3.5.2 When earthquake forces are included, net bearing pressure in soils can be increased as per Table 1, depending on type of foundation and type of soil. For determining the type of soil for this purpose, soils shall be classified in four types as given in Table 2. In soft soils, no increase shall be applied in bearing pressure, because settlements cannot be restricted by increasing bearing pressure.

6.3.5.3 In soil deposits consisting of submerged loose sands and soils falling under classification SP with corrected standard penetration test values *N*, less than 15 in Seismic Zones III, IV and V, and less than 10 in Seismic Zone II, the shaking caused by earthquake ground motion may cause liquefaction or excessive total and differential settlements. Such sites should be avoided preferably for locating new structures, and should be avoided for locating structures of important projects. Otherwise, settlements need to be investigated, and appropriate methods adopted of compaction or stabilization to achieve N values indicated in Note 4 of Table 1. Alternatively, deep pile foundations may be adopted and anchored at depths well below the underlying soil layers, which are likely to liquefy or undergo excessive settlements.

Also, marine clay layers and other sensitive clay layers are known to liquefy, undergo excessive settlements or even collapse, owing to low shear strength of the said soil; such soils will need special treatment according to site condition (*see* Table 2).

A simplified method is given in Annex F, for evaluation of liquefaction potential.

6.4 Design Acceleration Spectrum

6.4.1 For the purpose of determining design seismic force, the country is classified into four seismic zones as shown in Fig. 1.

6.4.2 The design horizontal seismic coefficient A_h for a structure shall be determined by:

$$A_h = \frac{\left(\frac{Z}{2}\right)\left(\frac{S_a}{g}\right)}{\left(\frac{R}{I}\right)}$$

where

Z = seismic zone factor given in Table 3;

I = importance factor given in IS 1893 (Parts 1 to 5) for the corresponding structures; when not specified, the minimum values of I shall be,

a) 1.5 for critical and lifeline structures;

b) 1.2 for business continuity structures; and

c) 1.0 for the rest.

R = response reduction factor given in IS 1893 (Parts 1 to 5) for the corresponding structures; and

$\left(\frac{S_a}{g}\right)$ = design acceleration coefficient for different soil types, normalized with peak ground acceleration, corresponding to natural period T of structure (considering soil-structure interaction, if required). It shall be as given in Parts 1 to 5 of IS 1893 for the corresponding structures; when not specified, it shall be taken as that corresponding to 5 percent damping, given by expressions below:

a) For use in equivalent static method [*see* Fig. 2(a)]:

$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 2.5 & 0 < T < 0.40\text{ s} \\ \dfrac{1}{T} & 0.40\text{ s} < T < 4.00\text{ s} \\ 0.25 & T > 4.00\text{ s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 2.5 & 0 < T < 0.55\text{ s} \\ \dfrac{1.36}{T} & 0.55\text{ s} < T < 4.00\text{ s} \\ 0.34 & T > 4.00\text{ s} \end{cases} \\ \text{For soft soil sites} & \begin{cases} 2.5 & 0 < T < 0.67\text{ s} \\ \dfrac{1.67}{T} & 0.67\text{ s} < T < 4.00\text{ s} \\ 0.42 & T > 4.00\text{ s} \end{cases} \end{cases}$$

b) For use in response spectrum method [*see* Fig. 2(b)]

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

6.4.2.1 For determining the correct spectrum to be used in the estimate of (S_a/g), the type of soil on which the structure is placed shall be identified by the classification given in Table 4, as:

a) *Soil type* I — Rock or hard soils;

b) *Soil type* II — Medium or stiff soils; and

c) *Soil type* III — Soft soils.

In Table 4, the value of N to be used shall be the weighted average of N of soil layers from the existing ground level to 30 m below the existing ground level; here, the N values of individual layers shall be the corrected values.

Table 1 Percentage Increase in Net Bearing Pressure and Skin Friction of Soils
(*Clause* 6.3.5.2)

Sl No. (1)	Soil Type (2)	Percentage Increase Allowable (3)
i)	*Type A:* Rock or hard soils	50
ii)	*Type B:* Medium or stiff soils	25
iii)	*Type C:* Soft soils	0

NOTES

1 The net bearing pressure shall be determined in accordance with IS 6403 or IS 1888.

2 Only corrected values of *N* shall be used.

3 If any increase in net bearing pressure has already been permitted for forces other than seismic forces, the increase in allowable bearing pressure, when seismic force is also included, shall not exceed the limits specified above.

4 The desirable minimum corrected field values of *N* shall be as specified below:

Seismic Zone	*Depth* (m) *below Ground Level*	*N Values*	*Remarks*
III, IV and V	≤ 5	15	For values of depths between 5 m and 10 m, linear interpolation is recommended
	≥10	25	
II	≤ 5	10	
	≥10	20	

If soils of lower *N* values are encountered than those specified in the table above, then suitable ground improvement techniques shall be adopted to achieve these values. Alternately, deep pile foundations should be used, which are anchored in stronger strata, underlying the soil layers that do not meet the requirement.

5 Piles should be designed for lateral loads neglecting lateral resistance of those soil layers (if any), which are liable to liquefy.

6 Indian Standards IS 1498 and IS 2131 may be referred for soil notation, and corrected *N* values shall be determined by applying correction factor C_N for effective overburden pressure σ'_{vo} using relation $N = C_N N_1$, where $C_N = \sqrt{P_a/\sigma'_{vo}} \leq 1.7$, P_a is the atmospheric pressure and N1 is the uncorrected SPT value for soil.

7 While using this table, the value of *N* to be considered shall be determined as below:

a) Isolated footings — Weighted average of *N* of soil layers from depth of founding, to depth of founding plus twice the breadth of footing;

b) Raft foundations — Weighted average of *N* of soil layers from depth of founding, to depth of founding plus twice the breadth of raft;

c) Pile foundation — Weighted average of *N* of soil layers from depth of bottom tip of pile, to depth of bottom tip of pile plus twice the diameter of pile;

d) Group pile foundation — Weighted average of *N* of soil layers from depth of bottom tip of pile group, to depth of bottom tip of pile group plus twice the width of pile group; and

e) Well foundation — Weighted average of *N* of soil layers from depth of bottom tip of well, to depth of bottom tip of well plus twice the width of well.

Table 2 Classification of Types of Soils for Determining Percentage Increase in Net Bearing Pressure and Skin Friction
(*Clause* 6.3.5.2)

Sl No. (1)	Soil Type (2)	Remarks (3)
i)	**Type A** Rock or hard soils	Well graded gravel (GW) or well graded sand (SW) both with less than 5 percent passing 75 mm sieve (Fines) Well graded gravel — sand mixtures with or without fines (GW-SW) Poorly-graded sand (SP) or Clayey sand (SC), all having *N* above 30 Stiff to hard clays having *N* above 30, where *N* is corrected standard penetration test value
ii)	**Type B** Medium or stiff soils	Poorly graded sands or poorly graded sands with gravel (SP) with little or no fines having *N* between 10 and 30 Stiff to medium stiff fine-grained soils, like silts of low compressibility (ML) or clays of low compressibility (CL) having *N* between 10 and 30
iii)	**Type C** Soft soils	All soft soils other than SP with *N*<10. The various possible soils are: Silts of intermediate compressibility (MI); Silts of high compressibility (MH); Clays of intermediate compressibility (CI); Clays of high compressibility (CH); Silts and clays of intermediate to high com-pressibility (MI-MH or CI-CH); Silt with clay of intermediate compressibility (MI-CI); and Silt with clay of high compressibility (MH-CH).
iv)	**Type D** Unstable, collapsible, liquefiable soils	Requires site-specific study and special treatment according to site condition (*see* **6.3.5.3**)

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

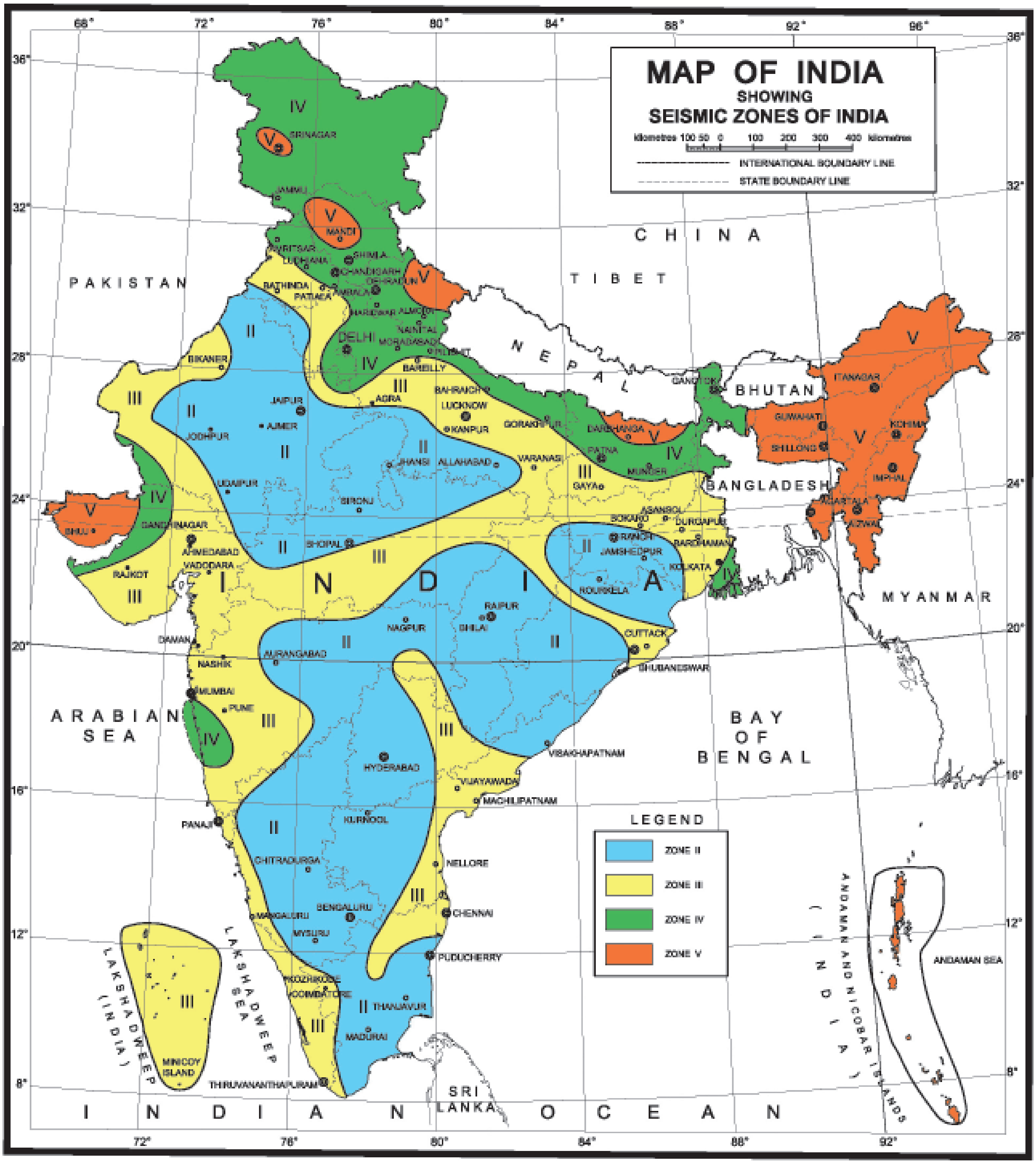
6.4.3 Effects of design earthquake loads applied on structures can be considered in two ways, namely:

a) Equivalent static method, and
b) Dynamic analysis method.

In turn, dynamic analysis can be performed in three ways, namely:

1) Response spectrum method,
2) Modal time history method, and
3) Time history method.

In this standard, Equivalent Static Method, Response Spectrum Method and Time History Method are

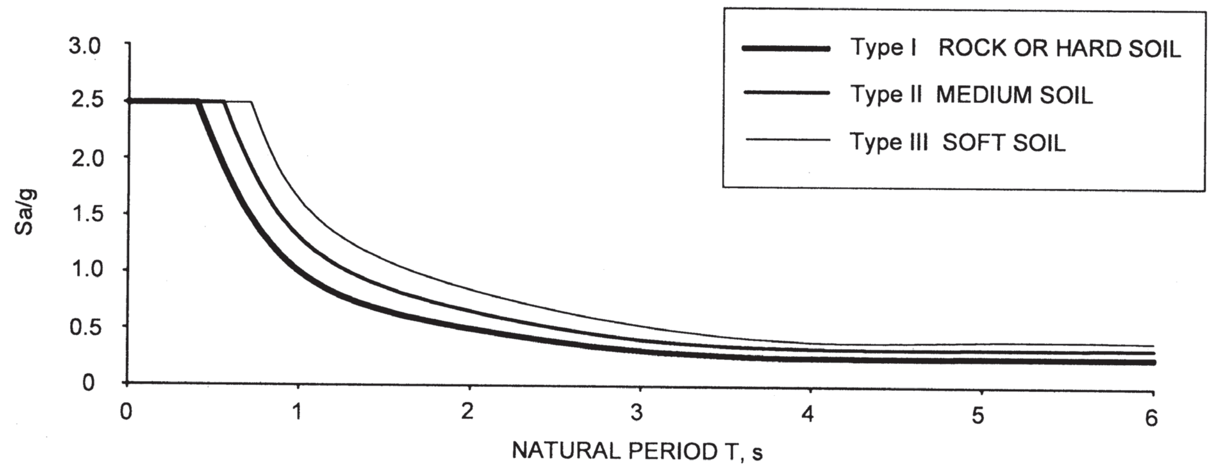


Based upon Survey of India Political map printed in 2002.

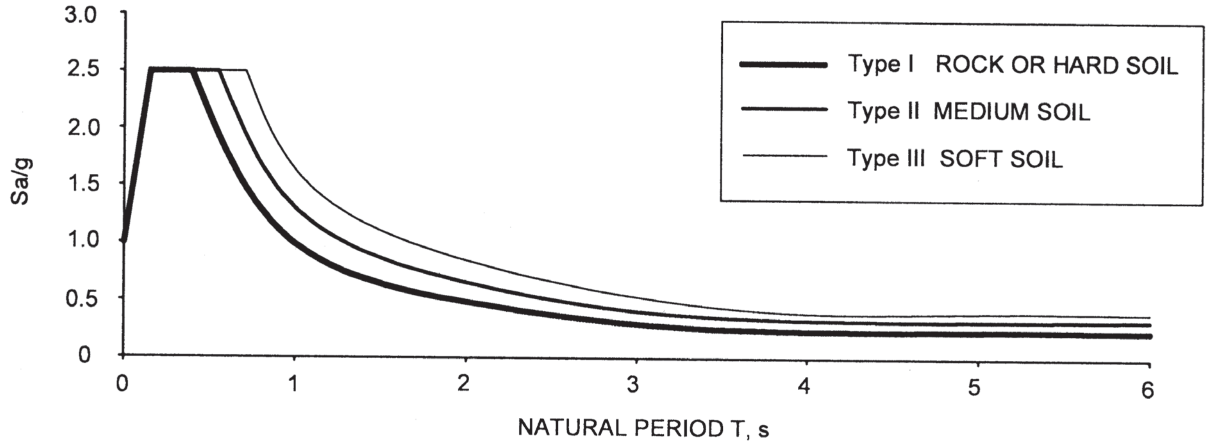
The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate baseline.
The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.
The state boundaries between Uttarakhand & Uttar Pradesh, Bihar & Jharkhand, and Chhattisgarh & Madhya Pradesh have not been verified by the Governments concerned.
The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.
The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.
The responsibility for the correctness of internal details rests with the publisher.

NOTE — Towns falling at the boundary of zones demarcation line between two zones shall be considered in higher zone.

FIG. 1 SEISMIC ZONES OF INDIA



2A SPECTRA FOR EQUIVALENT STATIC METHOD



2B SPECTRA FOR RESPONSE SPECTRUM METHOD

FIG. 2 DESIGN ACCELERATION COEFFICIENT (S_a/g) (CORRESPONDING TO 5 PERCENT DAMPING)

Table 4 Classification of Types of Soils for Determining the Spectrum to be Used to Estimate Design Earthquake Force

(*Clause* 6.4.2.1)

Sl No. (1)	Soil Type (2)	Remarks (3)
i)	**I** Rock or Hard soils	a) Well graded gravel (GW) or well graded sand (SW) both with less than 5 percent passing 75 µm sieve (Fines) b) Well graded gravel-sand mixtures with or without fines (GW-SW) c) Poorly graded sand (SP) or clayey sand (SC), all having *N* above 30 d) Stiff to hard clays having *N* above 30, where *N* is standard penetration test value
ii)	**II** Medium or Stiff soils	a) Poorly graded sands or poorly graded sands with gravel (SP) with little or no fines having *N* between 10 and 30 b) Stiff to medium stiff fine-grained soils, like silts of low compressibility (ML) or clays of low compressibility (CL) having *N* between 10 and 30
iii)	**III** Soft soils	All soft soils other than SP with *N*<10. The various possible soils are: a) Silts of intermediate compressibility (Ml); b) Silts of high compressibility (MH); c) Clays of intermediate compressibility (CI); d) Clays of high compressibility (CH); e) Silts and clays of intermediate to high compressibility (MI-MH or CI-CH); f) Silt with clay of intermediate compressibility (MI-CI); and g) Silt with clay of high compressibility (MH-CH).

adopted. Equivalent static method may be used for analysis of regular structures with approximate natural period T_a less than 0.4 s.

6.4.3.1 For structural analysis, the moment of inertia shall be taken as:

a) In RC and masonry structures: 70 percent of I_{gross} of columns, and 35 percent of I_{gross} of beams; and

b) In steel structures: I_{gross} of both beams and columns.

6.4.4 Where a number of modes are to be considered in response spectrum method, A_h as defined in **6.4.2** for each mode k shall be determined using natural period T_k of oscillation of that mode.

6.4.5 For underground structures and buildings whose base is located at depths of 30 m or more, A_h at the base shall be taken as half the value obtained from **6.4.2**. This reduced value shall be used only for estimating inertia effects due to masses at the corresponding levels below the ground; the inertia effects for the above ground portion of the building shall be estimated based on the unreduced value of A_h. For estimating inertia effects due to masses of structures and foundations placed between the ground level and 30 m depth, the design horizontal acceleration spectrum value shall be linearly interpolated between A_h and 0.5 A_h, where A_h is as specified in **6.4.2**.

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

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

The value of S_a/g shall be based on natural period T corresponding to the first vertical mode of oscillation, using **6.4.2**.

6.4.7 When design acceleration spectrum is developed specific to a project site, the same may be used for design of structures of the project. In such cases, effects of the site-specific spectrum shall not be less than those arising out of the design spectrum specified in this standard.

7 BUILDINGS

The four main desirable attributes of an earthquake resistant building are:

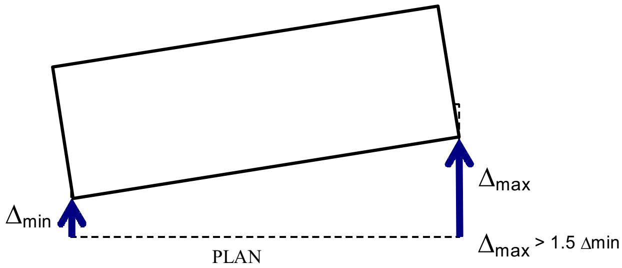
a) Robust structural configuration,
b) At least a minimum elastic lateral stiffness,
c) At least a minimum lateral strength, and
d) Adequate ductility.

7.1 Regular and Irregular Configurations

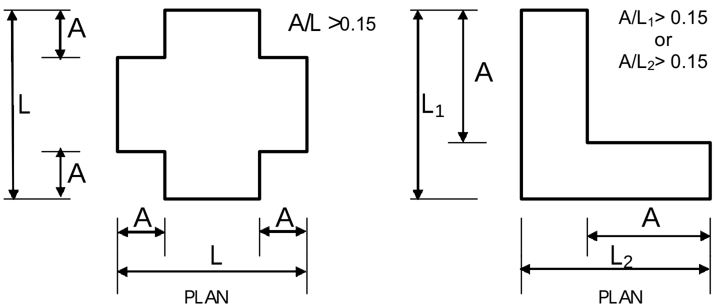
Buildings with simple regular geometry and uniformly distributed mass and stiffness in plan and in elevation, suffer much less damage, than buildings with irregular configurations. All efforts shall be made to eliminate irregularities by modifying architectural planning and structural configurations. A building shall be considered to be irregular for the purposes of this standard, even if any one of the conditions given in Tables 5 and 6 is applicable. Limits on irregularities for Seismic Zones III, IV and V and special requirements are laid out in Tables 5 and 6.

Table 5 Definitions of Irregular Buildings – Plan Irregularities (*see* Fig. 3)
(*Clause* 7.1)

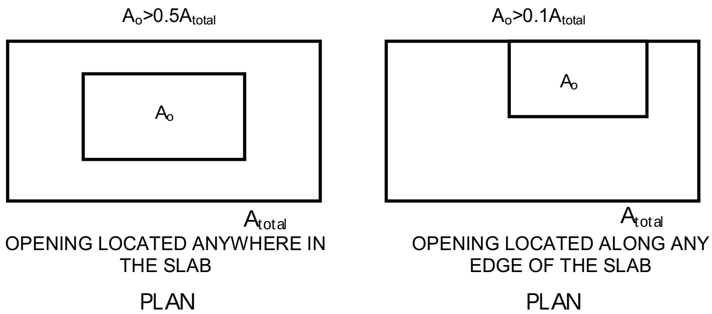
Sl No. (1)	Type of Plan Irregularity (2)
i)	**Torsional Irregularity** Usually, a well-proportioned building does not twist about its vertical axis, when a) the stiffness distribution of the vertical elements resisting lateral loads is balanced in plan according to the distribution of mass in plan (at each storey level); and b) the floor slabs are stiff in their own plane (this happens when its plan aspect ratio is less than 3) A building is said to be torsionally irregular, when, 1) the maximum horizontal displacement of any floor in the direction of the lateral force at one end of the floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction; and 2) the natural period corresponding to the fundamental torsional mode of oscillation is more than those of the first two translational modes of oscillation along each principal plan directions *In torsionally irregular buildings, when the ratio of maximum horizontal displacement at one end and the minimum horizontal displacement at the other end is,*



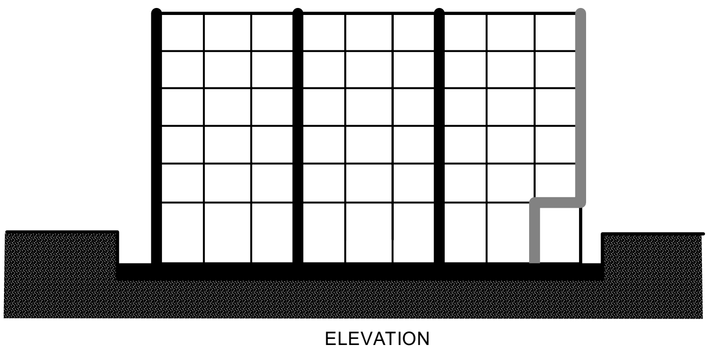
3A TORSIONAL IRREGULARITY



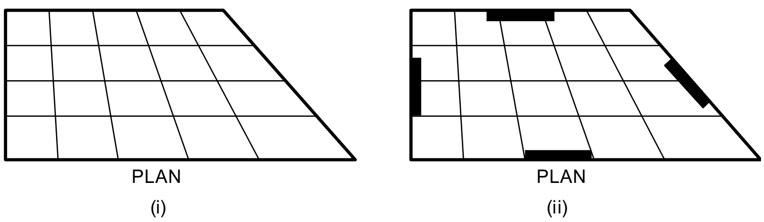
3B RE-ENTRANT CORNERS



3C FLOOR SLABS HAVING EXCESSIVE CUT-OUT AND OPENINGS



3D OUT-OF-PLANE OFFSETS IN VERTICAL ELEMENTS



3E NON-PARALLEL LATERAL FORCE SYSTEM:
(i) MOMENT FRAME BUILDING, and
(ii) MOMENT FRAME BUILDING WITH STRUCTURAL WALLS

FIG. 3 DEFINITIONS OF IRREGULAR BUILDINGS — PLAN IRREGULARITIES

Table 5 — (*Concluded*)

	i) *in the range 1.5 – 2.0, (a) the building configuration shall be revised to ensure that the natural period of the fundamental torsional mode of oscillation shall be smaller than those of the first two translational modes along each of the principal plan directions, and then (b) three dimensional dynamic analysis method shall be adopted; and* ii) *more than 2.0, the building configuration shall be revised*
ii)	**Re-entrant Corners** A building is said to have a re-entrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15 percent of its overall plan dimension in that direction *In buildings with re-entrant corners, three-dimensional dynamic analysis method shall be adopted.*
iii)	**Floor Slabs having Excessive Cut-Outs or Openings** Openings in slabs result in flexible diaphragm behaviour, and hence the lateral shear force is not shared by the frames and/or vertical members in proportion to their lateral translational stiffness. The problem is particularly accentuated when the opening is close to the edge of the slab. A building is said to have discontinuity in their in-plane stiffness, when floor slabs have cut-outs or openings of area more than 50 percent of the full area of the floor slab *In buildings with discontinuity in their in-plane stiffness, if the area of the geometric cut-out is,* a) *less than or equal to 50 percent, the floor slab shall be taken as rigid or flexible depending on the location of and size of openings; and* b) *more than 50 percent, the floor slab shall be taken as flexible.*
iv)	**Out-of-Plane Offsets in Vertical Elements** Out-of-plane offsets in vertical elements resisting lateral loads cause discontinuities and detours in the load path, which is known to be detrimental to the earthquake safety of the building. A building is said to have out-of-plane offset in vertical elements, when structural walls or frames are moved out of plane in any storey along the height of the building *In a building with out-of-plane offsets in vertical elements,* a) *specialist literature shall be referred for design of such a building, if the building is located in Seismic Zone II; and* b) *the following two conditions shall be satisfied, if the building is located in Seismic Zones III, IV and V:* 1) *Lateral drift shall be less than 0.2 percent in the storey having the offset and in the storeys below; and* 2) *Specialist literature shall be referred for removing the irregularity arising due to out-of-plane offsets in vertical elements.*
v)	**Non-Parallel Lateral Force System** Buildings undergo complex earthquake behaviour and hence damage, when they do not have lateral force resisting systems oriented along two plan directions that are orthogonal to each other. A building is said to have non-parallel system when the vertically oriented structural systems resisting lateral forces are not oriented along the two principal orthogonal axes in plan *Buildings with non-parallel lateral force resisting system shall be analyzed for load combinations mentioned in* **6.3.2.2** or **6.3.4.1**.

Table 6 Definition of Irregular Buildings – Vertical Irregularities (*see* Fig. 4)

(*Clause* 7.1)

Sl No. (1)	Type of Vertical Irregularity (2)
i)	**Stiffness Irregularity (Soft Storey)** A soft storey is a storey whose lateral stiffness is less than that of the storey above. *The structural plan density (SPD) shall be estimated when unreinforced masonry infills are used. When SPD of masonry infills exceeds 20 percent, the effect of URM infills shall be considered by explicitly modelling the same in structural analysis (as per* **7.9***). The design forces for RC members shall be larger of that obtained from analysis of:* *a) Bare frame, and* *b) Frames with URM infills,using 3D modelling of the structure. In buildings designed considering URM infills, the inter-storey drift shall be limited to 0.2 percent in the storey with stiffening and also in all storeys below.*
ii)	**Mass Irregularity** Mass irregularity shall be considered to exist, when the seismic weight *(as per* **7.7***)* of any floor is more than 150 percent of that of the floors below. *In buildings with mass irregularity and located in Seismic Zones III, IV and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per* **7.7***).*
iii)	**Vertical Geometric Irregularity** Vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 125 percent of the storey below. *In buildings with vertical geometric irregularity and located in Seismic Zones III, IV and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per* **7.7***).*
iv)	**In-Plane Discontinuity in Vertical Elements Resisting Lateral Force** In-plane discontinuity in vertical elements which are resisting lateral force shall be considered to exist, when in-plane offset of the lateral force resisting elements is greater than 20 percent of the plan length of those elements. *In buildings with in-plane discontinuity and located in Seismic Zones II, the lateral drift of the building under the design lateral force shall be limited to* 0.2 *percent of the building height; in Seismic Zones III, IV and V, buildings with in-plane discontinuity shall not be permitted.*
v)	**Strength Irregularity (Weak Storey)** A weak storey is a storey whose lateral strength is less than that of the storey above. *In such a case, buildings in Seismic Zones III, IV and V shall be designed such that safety of the building is not jeopardized; also, provisions of* 7.10 *shall be followed.*
vi)	**Floating or Stub Columns** Such columns are likely to cause concentrated damage in the structure. *This feature is undesirable, and hence should be prohibited, if it is part of or supporting the primary lateral load resisting system.*
vii)	**Irregular Modes of Oscillation in Two Principal Plan Directions** Stiffnesses of beams, columns, braces and structural walls determine the lateral stiffness of a building in each principal plan direction. A building is said to have lateral storey irregularity in a principal plan direction, if

Table 6 — (*Concluded*)

a) the first three modes contribute less than 65 percent mass participation factor in each principal plan direction, and b) the fundamental lateral natural periods of the building in the two principal plan directions are closer to each other by 10 percent of the larger value. *In buildings located in Seismic Zones II and III, it shall be ensured that the first three modes together contribute at least 65 percent mass participation factor in each principal plan direction. And, in buildings located in Seismic Zones IV and V, it shall be ensured that,* 1) *the first three modes together contribute at least 65 percent mass participation factor in each principal plan direction, and* 2) *the fundamental lateral natural periods of the building in the two principal plan directions are away from each other by at least 10 percent of the larger value.*

7.2 Lateral Force

7.2.1 *Design Lateral Force*

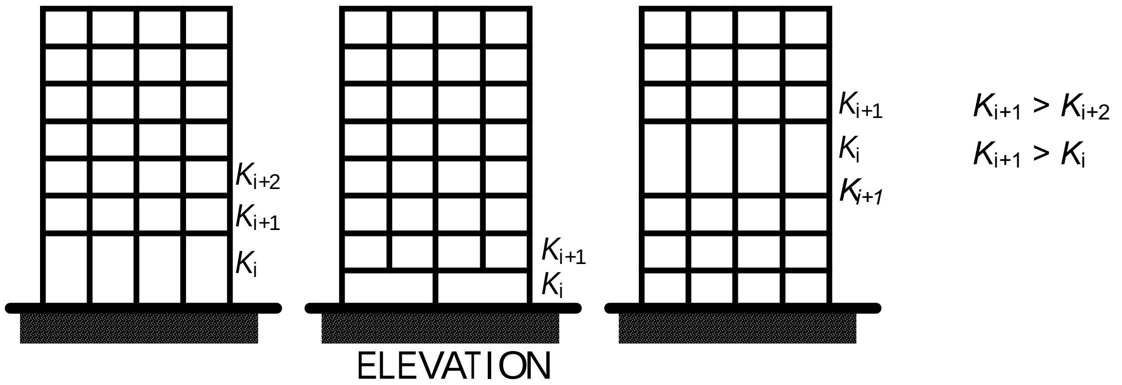
Buildings shall be designed for the design lateral force V_B given by:

$$V_B = A_h W$$

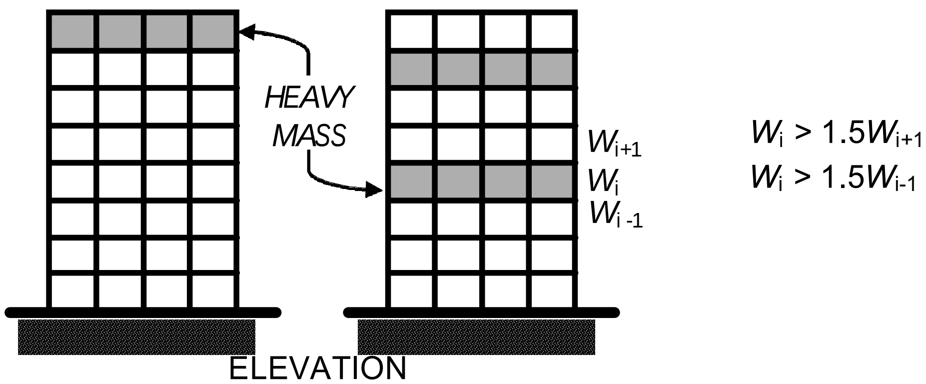
where A_h shall be estimated as per **6.4.2**, and W as per **7.4**.

7.2.2 *Minimum Design Lateral Force*

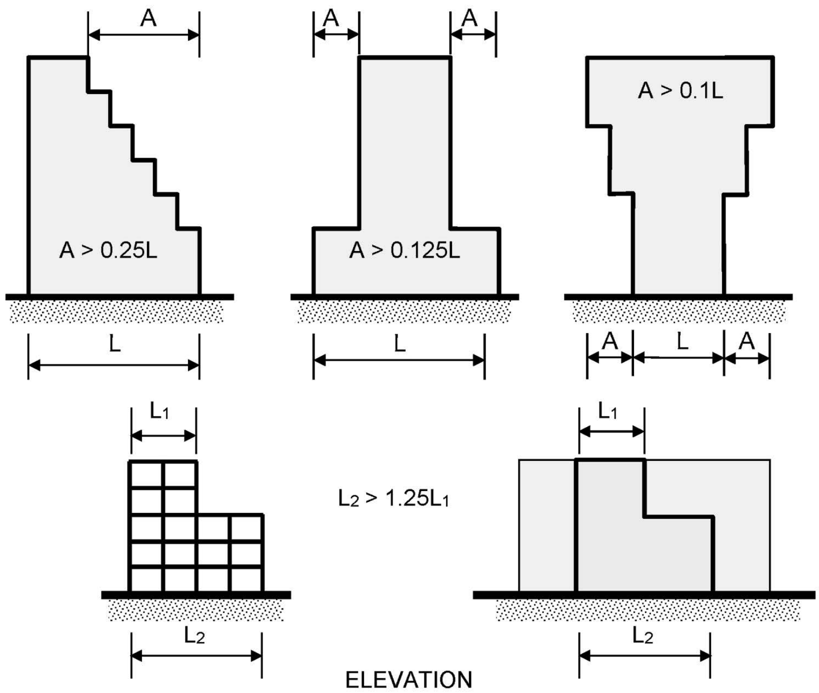
Buildings and portions there of shall be designed and constructed to resist at least the effects of design lateral force specified in **7.2.1**. But, regardless of design earthquake forces arrived at as per **7.3.1**, buildings shall have lateral load resisting systems capable of resisting a horizontal force not less than $(V_B)_{min}$ given in Table 7.



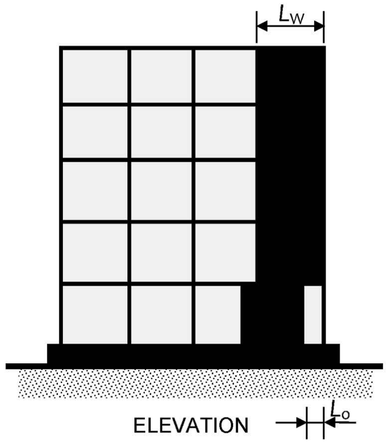
4A STIFFNESS IRREGULARITY (SOFT STOREY)



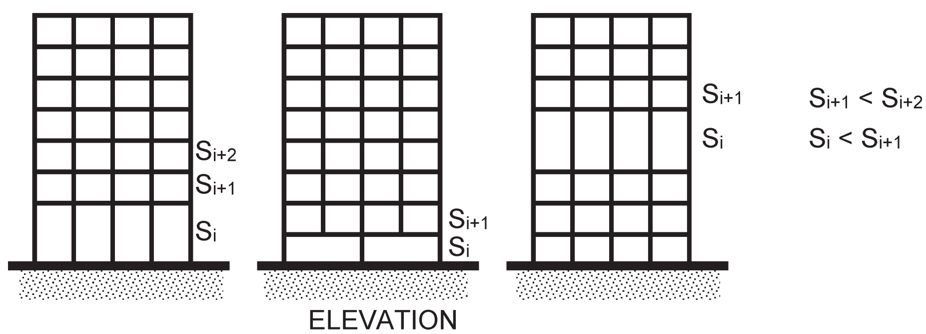
4B MASS IRREGULARITY



4C VERTICAL GEOMETRIC IRREGULARITY



4D IN-PLANE DISCONTINUITY IN VERTICAL ELEMENTS RESISTING LATERAL FORCE



4E STRENGTH IRREGULARITY (WEAK STOREY)

FIG. 4 DEFINITIONS OF IRREGULAR BUILDINGS — VERTICAL IRREGULARITIES

Table 7 Minimum Design Earthquake Horizontal Lateral Force for Buildings
(*Clause* 7.2.2)

Sl No. (1)	Seismic Zone (2)	ρ Percent (3)
i)	II	0.7
ii)	III	1.1
iii)	IV	1.6
iv)	V	2.4

7.2.3 *Importance Factor (I)*

In estimating design lateral force V_B of buildings as per **7.2.1**, the importance factor *I* of buildings shall be taken as per Table 8.

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

NOTES

1 Owners and design engineers of buildings or structures may choose values of importance factor *I* more than those mentioned above.

2 Buildings or structures covered under Sl No. (iii) may be designed for higher value of importance factor *I*, depending on economy and strategy.

3 In Sl No. (ii), when a building is composed of more than one structurally independent unit, the occupancy size shall be for each of the structurally independent unit of the building.

4 In buildings with mixed occupancies, wherein different *I* factors are applicable for the respective occupancies, larger of the importance factor *I* values shall be used for estimating the design earthquake force of the building.

7.2.4 *Damping Ratio*

The value of damping shall be taken as 5 percent of critical damping for the purposes of estimating A_h in the design lateral force V_B of a building as per **7.2.1**, irrespective of the material of construction (namely steel, reinforced concrete, masonry, or a combination thereof of these three basic materials) of its lateral load resisting system, considering that buildings experience inelastic deformations under design level earthquake effects, resulting in much higher energy dissipation than that due to initial structural damping in buildings. This value of damping shall be used, irrespective of the method of the structural analysis employed, namely Equivalent Static Method (as per **7.6**) or Dynamic Analysis Method (as per **7.7**).

7.2.5 *Design Acceleration Spectrum*

Design acceleration coefficient S_a/g corresponding to 5 percent damping for different soil types, normalized to peak ground acceleration, corresponding to natural period *T* of structure considering soil-structure interaction, irrespective of the material of construction of the structure. S_a/g shall be as given by expressions in **6.4.2**.

7.2.6 *Response Reduction Factor (R)*

Response reduction factor, along with damping during extreme shaking and redundancy: (a) influences the nonlinear behaviour of buildings during strong earthquake shaking, and (b) accounts for inherent system ductility, redundancy and overstrength normally available in buildings, if designed and detailed as per this standard and the associated Indian Standards.

For the purpose of design as per this standard, response reduction factor *R* for different building systems shall be as given in Table 9. The values of *R* shall be used for design of buildings with lateral load resisting elements, and NOT for just the lateral load resisting elements, which are built in isolation.

7.2.7 *Dual System*

Buildings with dual system consist of moment resisting frames and structural walls (or of moment resisting frames and bracings) such that both of the following conditions are valid:

a) Two systems are designed to resist total design lateral force in proportion to their lateral stiffness, considering interaction of two systems at all floor levels; and

b) Moment resisting frames are designed to resist independently at least 25 percent of the design base shear.

7.3 Design Imposed Loads for Earthquake Force Calculation

7.3.1 For various loading classes specified in IS 875 (Part 2), design seismic force shall be estimated using full dead load plus percentage of imposed load as given in Table 10. The same shall be used in the three-dimensional dynamic analysis of buildings also.

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
iii)	**Structural Wall Systems** (*see* Note 3)	
	a) Load bearing masonry buildings	
	1) Unreinforced masonry (designed as per IS 1905) without horizontal RC seismic bands (*see* Note 1)	1.5
	2) Unreinforced masonry (designed as per IS 1905) with horizontal RC seismic bands	2.0
	3) Unreinforced masonry (designed as per IS 1905) with horizontal RC seismic bands and vertical reinforcing bars at corners of rooms and jambs of openings (with reinforcement as per IS 4326)	2.5
	4) Reinforced masonry [*see* SP 7 (Part 6) Section 4]	3.0
	5) Confined masonry	3.0
	b) Buildings with ordinary RC structural walls (*see* Note 1)	3.0
	c) Buildings with ductile RC structural walls	4.0
iv)	**Dual Systems** (*see* Note 3)	
	a) Buildings with ordinary RC structural walls and RC OMRFs (*see* Note 1)	3.0
	b) Buildings with ordinary RC structural walls and RC SMRFs (*see* Note 1)	4.0
	c) Buildings with ductile RC structural walls with RC OMRFs (*see* Note 1)	4.0
	d) Buildings with ductile RC structural walls with RC SMRFs	5.0
v)	**Flat Slab – Structural Wall Systems** (*see* Note 4)	
	RC building with the three features given below: a) Ductile RC structural walls (which are designed to resist 100 percent of the design lateral force), b) Perimeter RC SMRFs (which are designed to independently resist 25 percent of the design lateral force), and preferably c) An outrigger and belt truss system connecting the core ductile RC structural walls and the perimeter RC SMRFs (*see* Note 1).	3.0

NOTES

1 RC and steel structures in Seismic Zones III, IV and V shall be designed to be ductile. Hence, this system is not allowed in these seismic zones.

2 Eccentric braces shall be used only with SBFs.

3 Buildings with structural walls also include buildings having structural walls and moment frames, but where,
a) frames are not designed to carry design lateral loads, or
b) frames are designed to carry design lateral loads, but do not fulfill the requirements of 'Dual Systems'.

4 In these buildings, (a) punching shear failure shall be avoided, and (b) lateral drift at the roof under design lateral force shall not exceed 0.1 percent.

7.3.2 For calculation of design seismic forces of buildings, imposed load on roof need not be considered. But, weights of equipment and other permanently fixed facilities should be considered; in such a case, the reductions of imposed loads mentioned in Table 10 are not applicable to that part of the load.

Table 10 Percentage of Imposed Load to be Considered in Calculation of Seismic Weight
(*Clause* 7.3.1)

Sl No. (1)	Imposed Uniformity Distributed Floor Loads kN/m^2 (2)	Percentage of Imposed Load (3)
i)	Up to and including 3.0	25
ii)	Above 3.0	50

7.3.3 Imposed load values indicated in Table 10 for calculating design earthquake lateral forces are applicable to normal conditions. When loads during earthquakes are more accurately assessed, designers may alter imposed load values indicated or even replace the entire imposed load given in Table 10 with actual assessed load values, subject to the values given in Table 7 as the minimum values. Where imposed load is not assessed as per **7.3.1** and **7.3.2**,

a) only that part of imposed load, which possesses mass, shall be considered; and
b) lateral earthquake design force shall not be calculated on contribution of impact effects from imposed loads.

7.3.4 Loads other than those given above (for example, snow and permanent equipment) shall be considered appropriately.

7.3.5 In regions of severe snow loads and sand storms exceeding intensity of 1.5 kN/m^2, 20 percent of uniform design snow load or sand load, respectively shall be included in the estimation of seismic weight. In case the minimum values of seismic weights corresponding to these load effects given in IS 875 are higher, the higher values shall be used.

7.3.6 In buildings that have interior partitions, the weight of these partitions on floors shall be included in the estimation of seismic weight; this value shall not be less than 0.5 kN/m^2. In case the minimum values of seismic weights corresponding to partitions given in parts of IS 875 are higher, the higher values shall be used. It shall be ensured that the weights of these partitions shall be considered only in estimating inertial effects of the building.

7.4 Seismic Weight

7.4.1 *Seismic Weight of Floors*

Seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in **7.3**. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be appropriately apportioned to the floors above and below the storey.

7.4.2 Any weight supported in between storeys shall be distributed to floors above and below in inverse proportion to its distance from the floors.

7.6 Equivalent Static Method

As per this method, first, the design base shear V_B shall be computed for the building as a whole. Then, this V_B shall be distributed to the various floor levels at the corresponding centres of mass. And, finally, this design seismic force at each floor level shall be distributed to individual lateral load resisting elements through structural analysis considering the floor diaphragm action. This method shall be applicable for regular buildings with height less than 15 m in Seismic Zone II.

7.6.1 The design base shear V_B along any principal direction of a building shall be determined by:

$$V_B = A_h W$$

where

A_h = design horizontal acceleration coefficient value as per **6.4.2**, using approximate fundamental natural period T_a as per **7.6.2** along the considered direction of shaking; and

W = seismic weight of the building as per **7.4**.

7.6.2 The approximate fundamental translational natural period T_a of oscillation, in second, shall be estimated by the following expressions:

a) Bare MRF buildings (without any masonry infills):

$$T_a = \begin{cases} 0.075h^{0.75} & \text{(for RC MRF building)} \\ 0.080h^{0.75} & \text{(for RC-Steel Composite MRF building)} \\ 0.085h^{0.75} & \text{(for steel MRF building)} \end{cases}$$

where

h = height (in m) of building (*see* Fig. 5). This excludes the basement storeys, where basement storey, walls are connected with the ground floor deck or fitted between the building columns, but includes the basement storeys, when they are not so connected.

b) Buildings with RC structural walls:

$$T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq \frac{0.09h}{\sqrt{d}}$$

where A_w is total effective area (m^2) of walls in the first storey of the building given by:

$$A_w = \sum_{i=1}^{N_w}\left[A_{wi}\left\{0.2+\left(\frac{L_{wi}}{h}\right)^2\right\}\right]$$

where

h = height of building as defined in **7.6.2**(a), in m;

A_{wi} = effective cross-sectional area of wall i in first storey of building, in m^2;

L_{wi} = length of structural wall i in first storey in the considered direction of lateral forces, in m;

d = base dimension of the building at the plinth level along the considered direction of earthquake shaking, in m; and

N_w = number of walls in the considered direction of earthquake shaking.

The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

c) All other buildings:

$$T_a = \frac{0.09h}{\sqrt{d}}$$

where

h = height of building, as defined in **7.6.2**(a), in m; and

d = base dimension of the building at the plinth level along the considered direction of earthquake shaking, in m.

7.6.3 The design base shear (V_B) computed in **7.6.1** shall be distributed along the height of the building and in plan at each floor level as below:

a) *Vertical distribution of base shear to different floor levels* — The design base shear V_B computed in **7.6.1** shall be distributed along the height of the building as per the following expression:

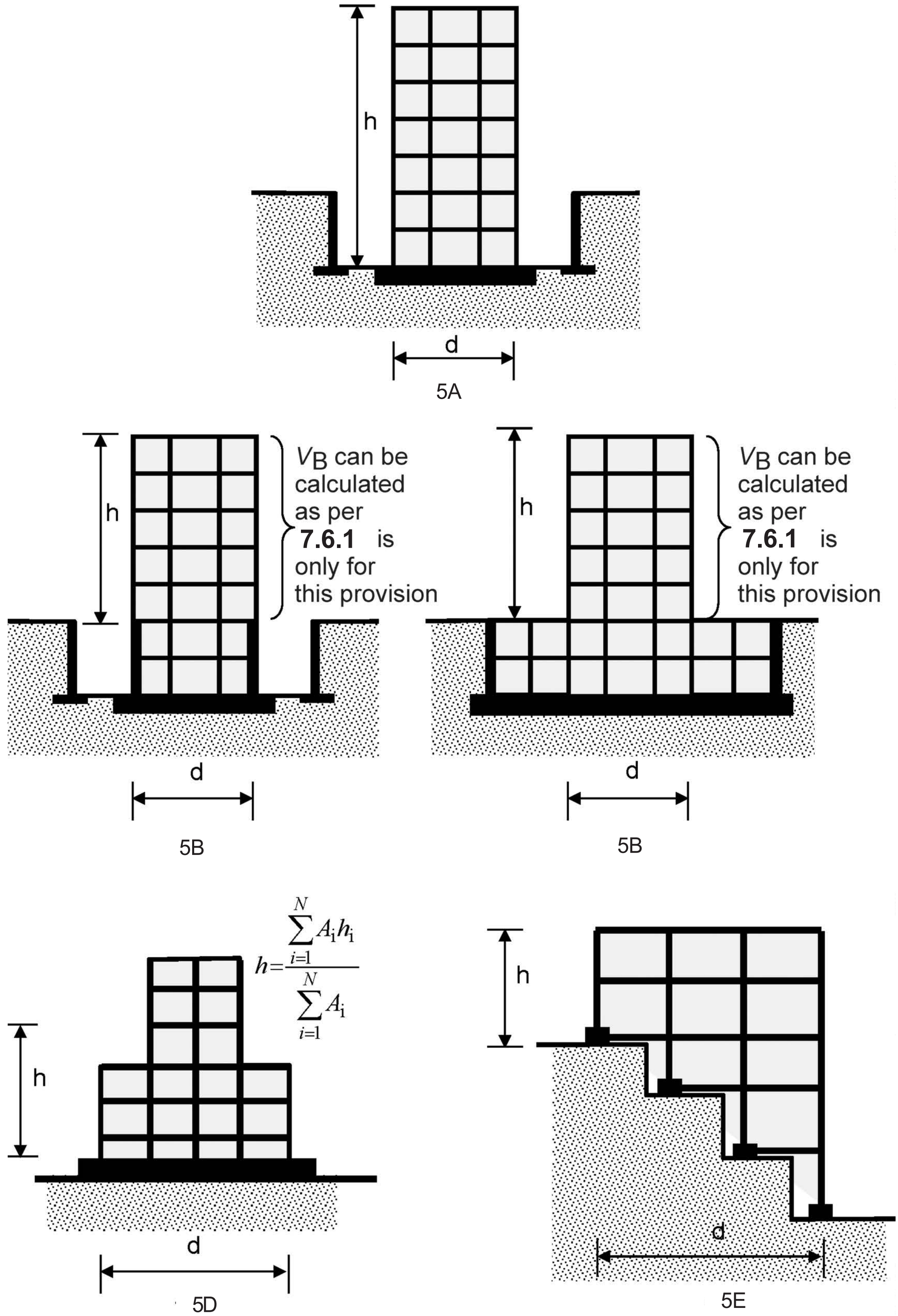


FIG. 5 DEFINITIONS OF HEIGHT AND BASE WIDTH OF BUILDINGS

$$Q_i = \left(\frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \right) V_B$$

where

Q_i = design lateral force at floor i;

W_i = seismic weight of floor i;

h_i = height of floor i measured from base; and

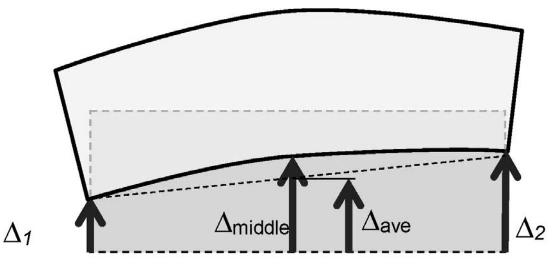
n = number of storeys in building, that is, number of levels at which masses are located.

b) *In-plan distribution of design lateral force at floor i to different lateral force resisting elements* — The design storey shear in any storey shall be calculated by summing the design lateral forces at all floor above that storey. In buildings whose floors are capable of providing rigid horizontal diaphragm action in their own plane, the design storey shear shall be distributed to the various vertical elements of lateral force resisting system in proportion to the lateral stiffness of these vertical elements.

7.6.4 *Diaphragm*

In buildings whose floor diaphragms cannot provide rigid horizontal diaphragm action in their own plane, design storey shear shall be distributed to the various vertical elements of lateral force resisting system considering the in-plane flexibility of the diaphragms.

A floor diaphragm shall be considered to be flexible, if it deforms such that the maximum lateral displacement measured from the chord of the deformed shape at any point of the diaphragm is more than 1.2 times the average displacement of the entire diaphragm (*see* Fig. 6).



$\Delta_{middle} > 1.2\Delta_{ave}$
where $\Delta_{ave} = (\Delta_1 + \Delta_2)/2$

PLAN

FIG. 6 DEFINITION OF FLEXIBLE FLOOR DIAPHRAGM

Usually, reinforced concrete monolithic slab-beam floors or those consisting of prefabricated or precast elements with reasonable reinforced screed concrete (at least a minimum of 50 mm on floors and of 75 mm on roof, with at least a minimum reinforcement of 6 mm bars spaced at 150 mm centres) as topping, and of plan aspect ratio less than 3, can be considered to be providing rigid diaphragm action.

7.7 Dynamic Analysis Method

7.7.1 Linear dynamic analysis shall be performed to obtain the design lateral force (design seismic base shear, and its distribution to different levels along the height of the building, and to various lateral load resisting elements) for all buildings, other than regular buildings lower than 15 m in Seismic Zone II.

7.7.2 The analytical model for dynamic analysis of buildings with unusual configuration should be such that it adequately represents irregularities present in the building configuration.

7.7.3 Dynamic analysis may be performed by either the Time History Method or the Response Spectrum Method. When either of the methods is used, the design base shear V_B estimated shall not be less than the design base shear $\bar{V}_B$ calculated using a fundamental period T_a, where T_a is as per **7.6.2**.

When V_B is less than $\bar{V}_B$, the force response quantities (for example member stress resultants, storey shear forces, and base reactions) shall be multiplied by $\bar{V}_B/V_B$. For earthquake shaking considered along,

a) the two mutually perpendicular plan directions X and Y, separate multiplying factors shall be calculated, namely $\bar{V}_{BX}/V_{BX}$ and $\bar{V}_{BY}/V_{BY}$, respectively; and

b) the vertical Z direction, the multiplying factor shall be taken as $Max\left[\bar{V}_{BX}/V_{BX};\bar{V}_{BY}/V_{BY}\right]$.

7.7.4 *Time History Method*

Time history method shall be based on an appropriate ground motion (preferably compatible with the design acceleration spectrum in the desired range of natural periods) and shall be performed using accepted principles of earthquake structural dynamics.

7.7.5 *Response Spectrum Method*

Response spectrum method may be performed for any building using the design acceleration spectrum specified in **6.4.2**, or by a site-specific design acceleration spectrum mentioned in **6.4.7**.

7.7.5.1 *Natural modes of oscillation*

Undamped free vibration analysis of the entire building shall be performed as per established methods of structural dynamics using appropriate mass and elastic stiffness of the structural system, to obtain natural periods T_k and mode shapes $\{\phi\}_k$ of those of its N_m modes of oscillation [$k \in (1, N_m)$] that need to be considered as per **7.7.5.2**.

7.7.5.2 *Number of modes to be considered*

The number of modes N_m to be used in the analysis for earthquake shaking along a considered direction, should be such that the sum total of modal masses of these modes considered is at least 90 percent of the total seismic mass.

If modes with natural frequencies beyond 33 Hz are to be considered, the modal combination shall be carried out only for modes with natural frequency less than 33 Hz; the effect of modes with natural frequencies more than 33 Hz shall be included by the missing mass correction procedure following established principles of structural dynamics. If justified by rigorous analysis, designers may use a cut off frequency other than 33 Hz.

7.7.5.3 *Combination of modes*

The responses of different modes considered shall be combined by one of the two methods given below:

a) Peak response quantities (for example, member forces, displacements, storey forces, storey shears, and base reactions) may be combined as per Complete Quadratic Combination (CQC) method, as given below:

$$\lambda = \sqrt{\sum_{i=1}^{N_m} \sum_{j=1}^{N_m} \lambda_i \ \rho_{ij} \ \lambda_j}$$

where

λ = estimate of peak response quantity;

λ_i = response quantity in mode i (with sign);

λ_j = response quantity in mode j (with sign);

ρ_{ij} = cross-modal correlation co-efficient

$$= \frac{8 \ \zeta^2 (1+\beta) \ \beta^{1.5}}{(1-\beta^2)^2 + 4 \ \zeta^2 \beta \ (1+\beta)^2};$$

N_m = number of modes considered;

ζ = modal damping coefficient ratio which shall be taken as 0.05;

β = natural frequency ratio = $\dfrac{\omega_j}{\omega_i}$;

ω_j = circular natural frequency in mode j; and

ω_i = circular natural frequency in mode i.

b) Alternatively, the peak response quantities may be combined as follows:

1) If building does not have closely-spaced modes, then net peak response quantity λ due to all modes considered shall be estimated as:

$$\lambda = \sqrt{\sum_{k=1}^{N_m} (\lambda_k)^2}$$

where

λ_k = peak response quantity in mode k, and

N_m = number of modes considered.

2) If building has a few closely-spaced modes, then net peak response quantity λ^* due to these closely space modes alone shall be obtained as:

$$\lambda^* = \sum_c |\lambda_c|$$

where

λ_c = peak response quantity in closely spaced mode c. The summation is for closely spaced modes only. Then, this peak response quantity λ^* due to closely spaced modes is combined with those of remaining well-separated modes by method described above.

7.7.5.4 *Simplified method of dynamic analysis of buildings*

Regular buildings may be analyzed as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In such a case, the following expressions shall hold in the computation of the various quantities:

a) *Modal mass* — Modal mass M_k of mode k is given by:

$$M_k = \frac{\left[\sum_{i=1}^{n} W_i \phi_{ik}\right]^2}{g \sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

where

g = acceleration due to gravity,

ϕ_{ik} = mode shape coefficient at floor i in mode k,

W_i = seismic weight of floor i of the structure, and

n = number of floors of the structure.

b) *Mode participation factor* — Mode participation factor P_k of mode k is given by:

$$P_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}}{\sum_{i=1}^{n} W_i (\phi_{ik})^2}$$

c) *Design lateral force at each floor in each mode* — Peak lateral force Q_{ik} at floor i in mode k is given by:

$$Q_{ik} = A_k \phi_{ik} P_k W_i$$

where

A_k = design horizontal acceleration spectrum value as per **6.4.2** using natural period of oscillation T_k of mode k obtained from dynamic analysis.

d) *Storey shear forces in each mode* — Peak *shear force* V_{ik} acting in storey i in mode k is given by:

$$V_{ik} = \sum_{j=i+1}^{n} Q_{ik}$$

e) *Storey shear force due to all modes considered* — Peak storey shear force V_i in storey i due to all modes considered, shall be obtained by combining those due to each mode in accordance with **7.7.5.3**.

f) *Lateral forces at each storey due to all modes considered* — Design lateral forces F_{roof} at roof level and F_i at level of floor i shall be obtained as:

$F_{roof} = V_{roof}$, and

$F_i = V_i - V_{i+1}$.

7.8 Torsion

7.8.1 Provision shall be made in all buildings for increase in shear forces on the lateral force resisting elements resulting from twisting about the vertical axis of the building, arising due to eccentricity between the centre of mass and centre of resistance at the floor levels. The design forces calculated as in **7.6** and **7.7.5**, shall be applied at the displaced centre of mass so as to cause design eccentricity (as given by **7.8.2**) between the displaced centre of mass and centre of resistance.

7.8.2 *Design Eccentricity*

While performing structural analysis by the Seismic Coefficient Method or the Response Spectrum Method, the design eccentricity e_{di} to be used at floor i shall be taken as:

$$e_{di} = \begin{cases} 1.5 e_{si} + 0.05 b_i \\ e_{si} - 0.05 b_i \end{cases}$$

whichever gives the more severe effect on lateral force resisting elements;

where

e_{si} = static eccentricity at floor *i*,
= distance between centre of mass and centre of resistance, and

b_i = floor plan dimension of floor *i*, perpendicular to the direction of force.

The factor 1.5 represents dynamic amplification factor, and $0.05b_i$ represents the extent of accidental eccentricity. The above amplification of 1.5 need not be used, when performing structural analysis by the Time History Method.

7.9 RC Frame Buildings with Unreinforced Masonry Infill Walls

7.9.1 In RC buildings with moment resisting frames and unreinforced masonry (URM) infill walls, variation of storey stiffness and storey strength shall be examined along the height of the building considering in-plane stiffness and strength of URM infill walls. If storey stiffness and strength variations along the height of the building render it to be irregular as per Table 6, the irregularity shall be corrected especially in Seismic Zones III, IV and V.

7.9.2 The estimation of in-plane stiffness and strength of URM infill walls shall be based on provisions given hereunder.

7.9.2.1 The modulus of elasticity E_m (in MPa) of masonry infill wall shall be taken as:

$$E_m = 550 f_m$$

where f_m is the compressive strength of masonry prism (in MPa) obtained as per IS 1905 or given by expression:

$$f_m = 0.433 f_b^{0.64} f_{mo}^{0.36}$$

where

f_b = compressive strength of brick, in MPa; and

f_{mo} = compressive strength of mortar, in MPa.

7.9.2.2 URM infill walls shall be modeled by using equivalent diagonal struts as below:

a) Ends of diagonal struts shall be considered to be pin-jointed to RC frame;

b) For URM infill walls without any opening, width w_{ds} of equivalent diagonal strut (*see* Fig. 7) shall be taken as:

$$w_{ds} = 0.175 \alpha_h^{-0.4} L_{ds}$$

where

$$\alpha_h = h \left(\sqrt[4]{\frac{E_m t \sin 2\theta}{4 E_f I_c h}} \right)$$

where E_m and E_f are the modulii of elasticity of the materials of the URM infill and RC MRF, I_c the moment of inertia of the adjoining column, t the thickness of the infill wall, and θ the angle of the diagonal strut with the horizontal;

c) For URM infill walls with openings, no reduction in strut width is required; and

d) Thickness of the equivalent diagonal strut shall be taken as thickness t of original URM infill wall, provided $h/t < 12$ and $l/t < 12$, where h is clear height of URM infill wall between the top beam and bottom floor slab, and l clear length of the URM infill wall between the vertical RC elements (columns, walls or a combination thereof) between which it spans.

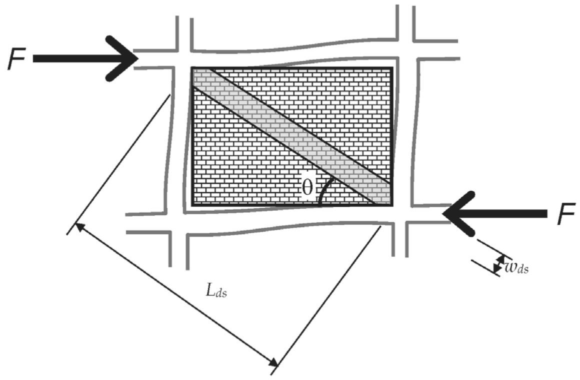


FIG. 7 EQUIVALENT DIAGONAL STRUT OF URM INFILL WALL

7.10 RC Frame Buildings with Open Storeys

7.10.1 RC moment resisting frame buildings, which have open storey(s) at any level, such as due to discontinuation of unreinforced masonry (URM) infill walls or of structural walls, are known to have flexible and weak storeys as per Table 6. In such buildings, suitable measures shall be adopted, which increase both stiffness and strength to the required level in the open storey and the storeys below. These measures shall be taken along both plan directions as per requirements laid down under **7.10.2** to **7.10.4**. The said increase may be achieved by providing measures, like:

a) RC structural walls, or

b) Braced frames, in select bays of the building.

7.10.2 When the RC structural walls are provided, they shall be,

a) founded on properly designed foundations;

b) continuous preferably over the full height of the building; and

c) connected preferably to the moment resisting frame of the building.

7.10.3 When the RC structural walls are provided, they shall be designed such that the building does NOT have:

a) Additional torsional irregularity in plan than that already present in the building. In assessing this, lateral stiffness shall be included of all elements that resist lateral actions at all levels of the building;

b) Lateral stiffness in the open storey(s) is less than 80 percent of that in the storey above; and

c) Lateral strength in the open storey(s) is less than 90 percent of that in the storey above.

7.10.4 When the RC structural walls are provided, the RC structural wall plan density ρ_{sw} of the building shall be at least 2 percent along each principal direction in Seismic Zones III, IV and V. These walls shall be well distributed in the plan of the building along each plan direction. RC structural walls of this measure can be adopted even in regular buildings that do not have open storey(s).

7.10.5 RC structural walls in buildings located in Seismic Zones III, IV and V shall be designed and detailed to comply with all requirements of IS 13920.

7.11 Deformation

Deformation of RC buildings shall be obtained from structural analysis using a structural model based on section properties given in **6.4.3**.

7.11.1 *Storey Drift Limitation*

7.11.1.1 Storey drift in any storey shall not exceed 0.004 times the storey height, under the action of design base of shear V_B with no load factors mentioned in **6.3**, that is, with partial safety factor for all loads taken as 1.0.

7.11.1.2 Displacement estimates obtained from dynamic analysis methods shall not be scaled as given in **7.7.3.**

7.11.2 *Deformation Capability of Non-Seismic Members*

For buildings located in Seismic Zones III, IV and V, it shall be ensured that structural components, that are not a part of seismic force resisting system in considered direction of ground motion but are monolithically connected, do not lose their vertical load-carrying capacity under induced net stress resultants, including additional bending moments and shear forces resulting from storey deformations equal to R times storey displacements calculated as per **7.11.1**, where R is specified in Table 9.

7.11.3 *Separation between Adjacent Units*

Two adjacent buildings, or two adjacent units of the same building with separation joint between them, shall be separated by a distance equal to R times sum of storey displacements Δ_1 and Δ_2 calculated as per **7.11.1** of the two buildings or two units of the same building, to avoid pounding as the two buildings or two units of the same building oscillate towards each other.

When floor levels of the adjacent units of a building or buildings are at the same level, the separation distance shall be calculated as $(R_1\Delta_1 + R_2\Delta_2)$, where R_1 and Δ_1 correspond to building 1, and R_2 and Δ_2 to building 2.

7.12 Miscellaneous

7.12.1 *Foundations*

Isolated RC footings without tie beams or unreinforced strip foundations, shall not be adopted in buildings rested on soft soils (with corrected $N < 10$) in any Seismic Zone. Use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided in buildings located in Seismic Zones III, IV and V.

Individual spread footings or pile caps shall be interconnected with ties (*see* **5.3.4.1** of IS 4326), except when individual spread footings are directly supported on rock, in buildings located in Seismic Zones IV and V. All ties shall be capable of carrying, in tension and in compression, an axial force equal to $A_h/4$ times the larger of the column or pile cap load, in addition to the otherwise computed forces, subject to a minimum of 5 percent of larger of column or pile cap loads. Here, A_h is as per **6.4.2**.

Pile shall be designed and constructed to withstand maximum curvature imposed (structural response) by earthquake ground shaking. Design of anchorage of piles into the pile cap shall consider combined effects, including that of axial forces due to uplift and bending moments due to fixity to pile cap.

7.12.2 *Cantilever Projections*

7.12.2.1 *Vertical projections*

Small-sized facilities (like towers, tanks, parapets, smoke stacks/chimneys) and other vertical cantilever projections attached to buildings and projecting vertically above the roof, but not a part of the structural system of the building, shall be designed and checked for stability for five times the design horizontal seismic coefficient A_h specified in **6.4.2** for that building. In the analysis of the building, weights of these projecting elements shall be lumped with the roof weight.

7.12.2.2 *Horizontal projections*

All horizontal projections of buildings (like cantilever structural members at the porch level or higher) or attached to buildings (like brackets, cornices and balconies) shall be designed for five times the design vertical coefficient A_v specified in **6.4.6** for that building.

7.12.2.3 The increased design forces specified in **7.12.2.1** and **7.12.2.2** are only for designing the projecting parts and their connections with the main structures, and NOT for the design of the main structure.

7.12.3 *Compound Walls*

Compound walls shall be designed for the design horizontal coefficient A_h of $1.25Z$, that is, A_h calculated using **6.4.2** with $I = 1$, $R = 1$ and $S_a/g = 2.5$.

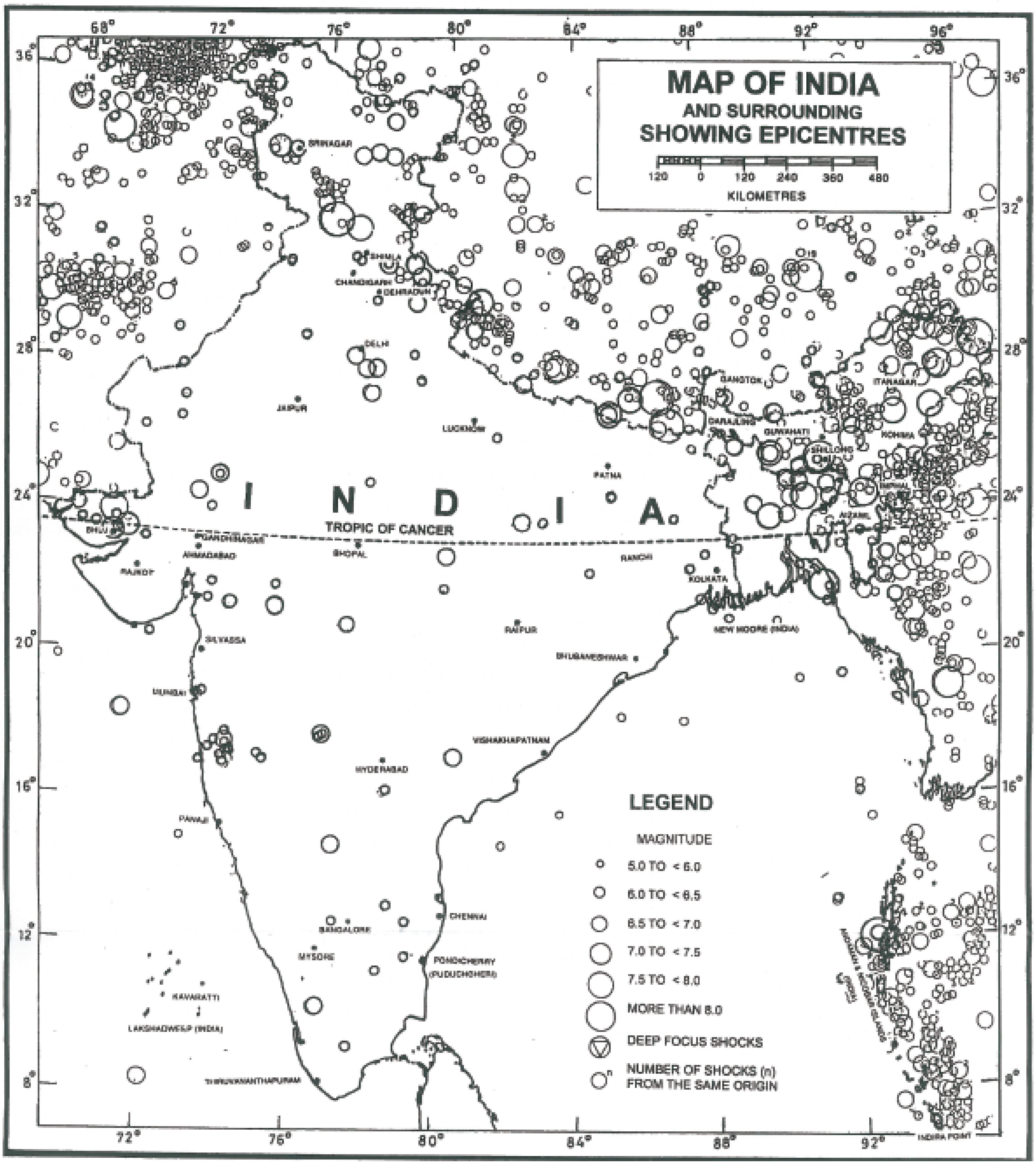
7.12.4 *Connections between Parts*

All small items and objects of a building shall be tied to the building or to each other to act as single unit, except those between the separation joints and seismic joints. These connections shall be made capable of transmitting the forces induced in them, but not less than 0.05 times weight of total dead and imposed load reactions; frictional resistance shall not be relied upon in these calculations.

ANNEX A

(*Foreword*)

MAP OF INDIA SHOWING EPICENTRES OF PAST EARTHQUAKES IN INDIA
(From Catalog of 2015)



Based upon Survey of India Political map printed in 2002.

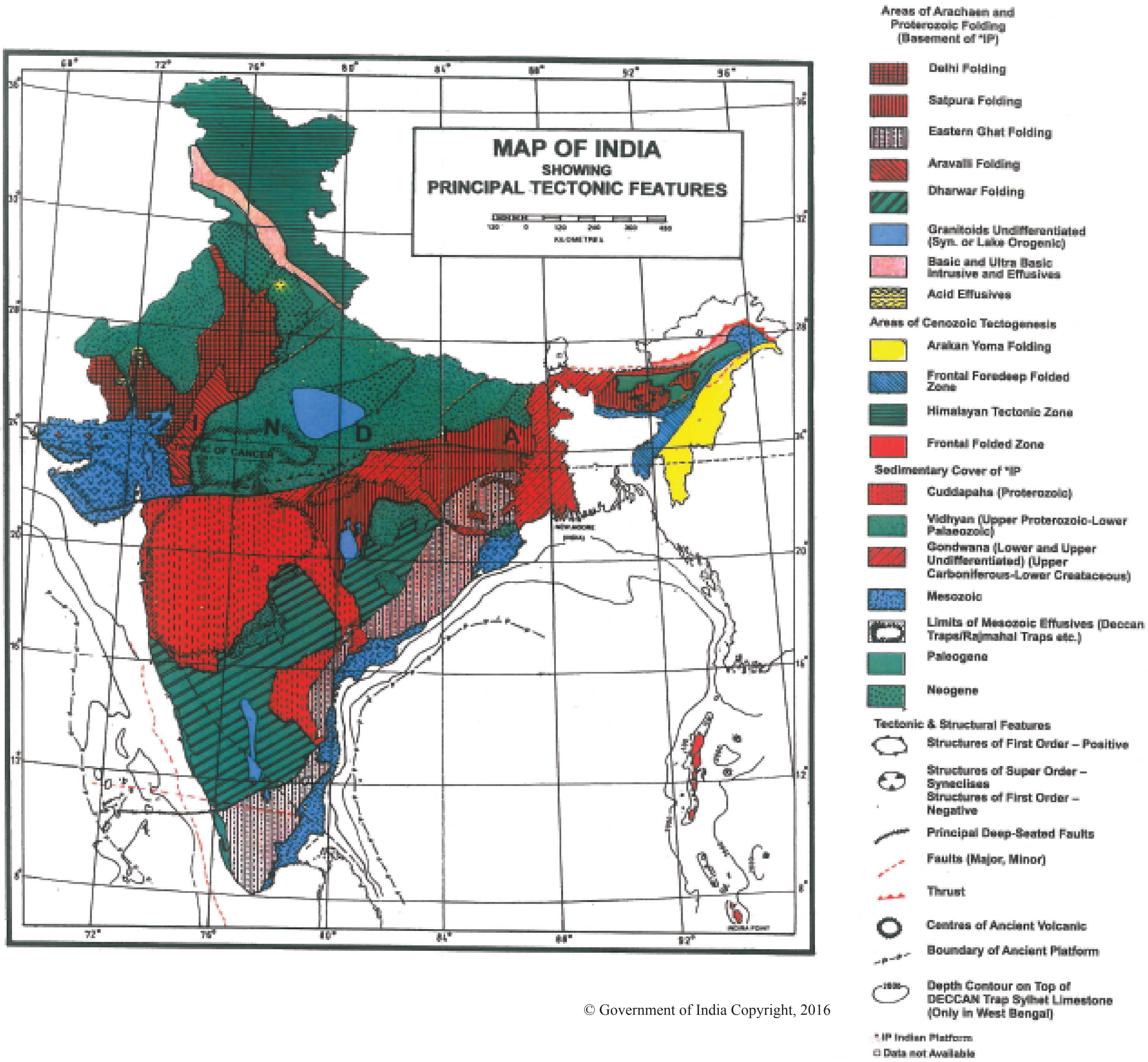
The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate baseline.
The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.
The state boundaries between Uttarakhand & Uttar Pradesh, Bihar & Jharkhand, and Chhattisgarh & Madhya Pradesh have not been verified by the Governments concerned.
The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.
The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.
The responsibility for the correctness of internal details rests with the publisher.

NOTE — For details regarding the up-to-date seismic activity (plotted on the Map of India), please visit the online portal of the National Centre for Seismology (NCS), Ministry of Earth Sciences, New Delhi.

ANNEX B

(*Foreword*)

MAP OF INDIA SHOWING PRINCIPAL TECTONIC FEATURES IN INDIA
(From Catalog of 2001)



Based upon Survey of India Political map printed in 2002.

The territorial waters of India extend into the sea to a distance of twelve nautical miles measured from the appropriate baseline.
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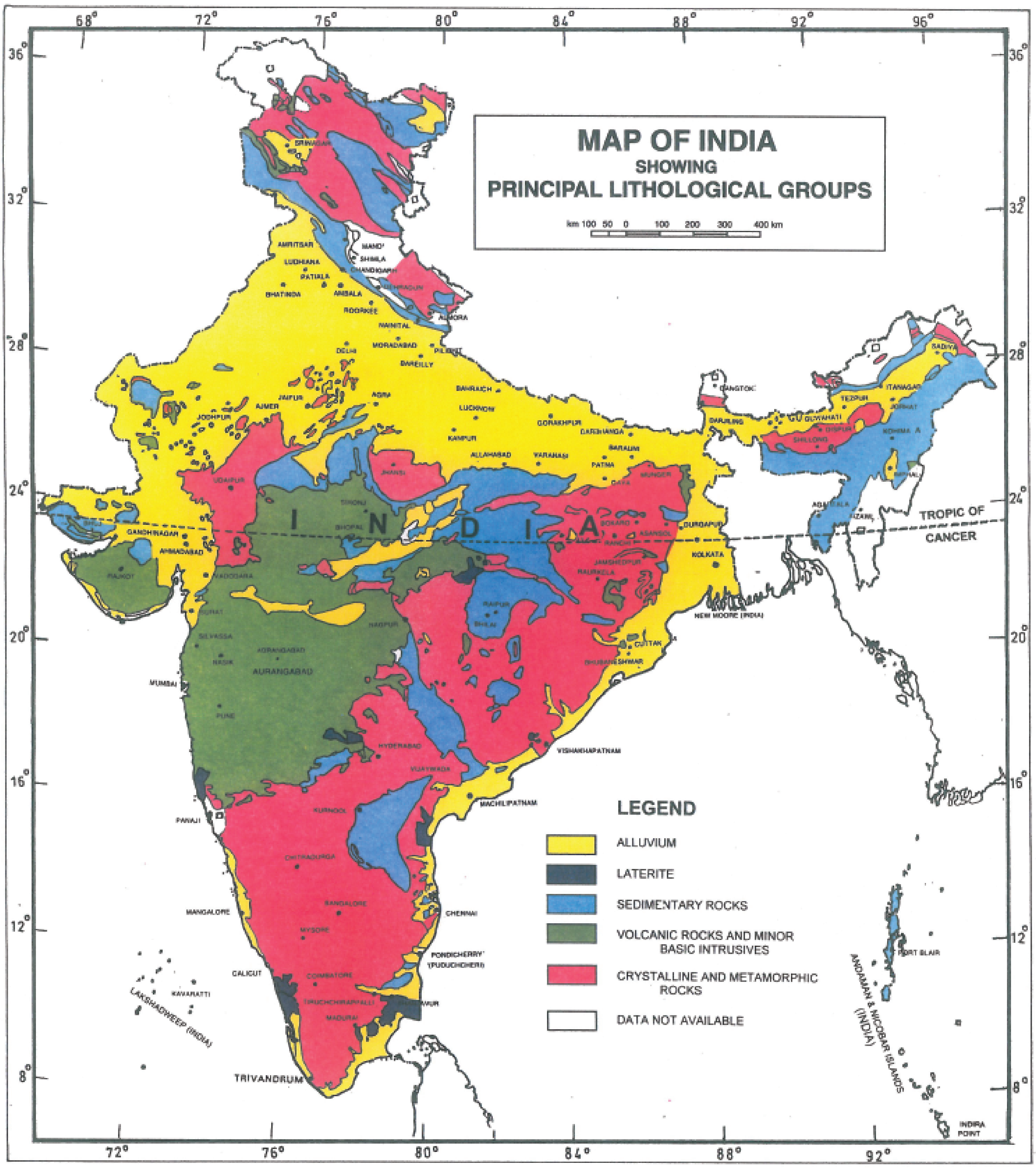
ANNEX C

(*Foreword*)

MAP OF INDIA SHOWING PRINCIPAL LITHOLOGICAL GROUPS



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ANNEX D

(*Foreword and Clause* 3.11)

MSK 1964 INTENSITY SCALE

D-1 The following description shall be applicable.

a) *Type of Structures (Buildings)*

Type A — Building in field-stone, rural structures, un-burnt brick houses, clay houses

Type B — Ordinary brick buildings, buildings of large block and prefabricated type, half timbered structures, buildings in natural hewn stone

Type C — Reinforced buildings, well built wooden structures

b) *Definition of Quantity*

Single, few	:	About 5 percent
Many	:	About 50 percent
Most	:	About 75 percent

c) *Classification of Damage to Buildings*

Classification	*Damage*	*Description*
Grade 1	Slight damage	Fine cracks in plaster; fall of small pieces of plaster
Grade 2	Moderate damage	Small cracks in walls; fall of fairly larger pieces of plaster; pantiles slip off; cracks in chimneys parts of chimney fall down
Grade 3	Heavy damage	Large and deep cracks in walls; fall of chimneys
Grade 4	Destruction	Gaps in walls; parts of buildings may collapse; separate parts of the buildings lose their cohesion; and inner walls collapse
Grade 5	Total damage	Total collapse of the building

D-2 MSK INTENSITY SCALE

D-2.1 The following introductory letters (i), (ii) and (iii) have been used throughout the intensity scales (I to XII), describing the following:

i) Persons and surroundings,

ii) Structures of all kinds, and

iii) Nature.

I Not Noticeable

i) The intensity of the vibration is below the limits of sensibility; the tremor is detected and recorded by seismograph only.

ii) —

iii) —

II Scarcely Noticeable (Very Slight)

i) Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.

ii) —

iii) —

III Weak, Partially Observed

i) The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects.

ii) —

iii) —

IV Largely Observed

i) The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors, and dishes rattle. Floors and walls crack. Furniture begins to shake. Hanging objects swing slightly. Liquid in open vessels are slightly disturbed. In standing motor cars the shock is noticeable.

ii) —

iii) —

V Awakening

i) The earthquake is felt indoors by all, outdoors by many. Many people awake. A few run outdoors. Animals become uneasy. Buildings tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Unstable objects overturn or shift. Open doors and windows are thrust open and slam back again. Liquids spill in small amounts from well-filled open containers. The sensation of vibration is like that due to heavy objects falling inside the buildings.

ii) Slight damages in buildings of Type A are possible.

iii) Slight waves on standing water. Sometimes changes in flow of springs.

VI Frightening

i) Felt by most indoors and outdoors. Many people in buildings are frightened and run outdoors. A few persons loose their balance. Domestic animals run out of their stalls. In few instances, dishes and glassware may break, and books fall down, pictures move, and unstable objects overturn. Heavy furniture may possibly move and small steeple bells may ring.

ii) Damage of Grade 1 is sustained in single buildings of Type B and in many of Type A. Damage in some buildings of Type A is of Grade 2.

iii) Cracks up to widths of 10 mm possible in wet ground; in mountains occasional landslips: change in flow of springs and in level of well water are observed.

VII Damage of Buildings

i) Most people are frightened and run outdoors. Many find it difficult to stand. The vibration is noticed by persons driving motor cars. Large bells ring.

ii) In many buildings of Type C damage of Grade 1 is caused; in many buildings of Type B damage is of Grade 2. Most buildings of Type A suffer damage of Grade 3, few of Grade 4. In single instances, landslides of roadway on steep slopes: crack in roads; seams of pipelines damaged; cracks in stone walls.

iii) Waves are formed on water, and water is made turbid by mud stirred up. Water levels in wells change, and the flow of springs changes. Sometimes dry springs have their flow restored and existing springs stop flowing. In isolated instances parts of sand and gravelly banks slip off.

VIII Destruction of Buildings

i) Fright and panic; also persons driving motor cars are disturbed. Here and there branches of trees break off. Even heavy furniture moves and partly overturns. Hanging lamps are damaged in part.

ii) Most buildings of Type C suffer damage of Grade 2, and few of Grade 3. Most buildings of Type B suffer damage of Grade 3. Most buildings of Type A suffer damage of Grade 4. Occasional breaking of pipe seams. Memorials and monuments move and twist. Tombstones overturn. Stone walls collapse.

iii) Small landslips in hollows and on banked roads on steep slopes; cracks in ground up to widths of several centimetres. Water in lakes become turbid. New reservoirs come into existence. Dry wells refill and existing wells become dry. In many cases, change in flow and level of water is observed.

IX General Damage of Buildings

i) General panic; considerable damage to furniture. Animals run to and fro in confusion and cry.

ii) Many buildings of Type C suffer damage of Grade 3, and a few of Grade 4. Many buildings of Type B show a damage of Grade 4 and a few of Grade 5. Many buildings of Type A suffer damage of Grade 5. Monuments and columns fall. Considerable damage to reservoirs; underground pipes partly broken. In individual cases, railway lines are bent and roadway damaged.

iii) On flat land overflow of water, sand and mud is often observed. Ground cracks to widths of up to 10 cm, on slopes and river banks more than 10 cm. Furthermore, a large number of slight cracks in ground; falls of rock, many land slides and earth flows; large waves in water. Dry wells renew their flow and existing wells dry up.

X General Destruction of Buildings

i) —

ii) Many buildings of Type C suffer damage of Grade 4, and a few of Grade 5. Many buildings of Type B show damage of Grade 5. Most of Type A has destruction of Grade 5. Critical damage to dykes and dams. Severe damage to bridges. Railway lines are bent slightly. Underground pipes are bent or broken. Road paving and asphalt show waves.

iii) In ground, cracks up to widths of several centimetres, sometimes up to 1 m, parallel to water courses occur broad fissures. Loose ground slides from steep slopes. From river banks and steep coasts, considerable landslides are possible. In coastal areas, displacement of sand and mud; change of water level in wells; water from canals, lakes, rivers, etc, thrown on land. New lakes occur.

XI Destruction

i) —

ii) Severe damage even to well built buildings, bridges, water dams and railway lines. Highways become useless. Underground pipes destroyed.

iii) Ground considerably distorted by broad cracks and fissures, as well as movement in horizontal and vertical directions. Numerous landslips and falls of rocks. The intensity of the earthquake requires to be investigated specifically.

XII Landscape Changes

i) —

ii) Practically all structures above and below ground are greatly damaged or destroyed.

iii) The surface of the ground is radically changed. Considerable ground cracks with extensive vertical and horizontal movements are observed. Falling of rock and slumping of river banks over wide areas, lakes are dammed; waterfalls appear and rivers are deflected. The intensity of the earthquake requires to be investigated specially.

ANNEX E

(*Foreword*)

LIST OF SOME TOWNS WITH POPULATION MORE THAN 3 LAKHS (as per CENSUS 2011) AND THEIR SEISMIC ZONE FACTOR Z

Town	Zone	Z
Agra	III	0.16
Ahmedabad	III	0.16
Ajmer	II	0.10
Allahabad	II	0.10
Almora	IV	0.24
Ambala	IV	0.24
Amritsar	IV	0.24
Asansol	III	0.16
Aurangabad	II	0.10
Bahraich	IV	0.24
Bangalore (Bengaluru)	II	0.10
Barauni	IV	0.24
Bareilly	III	0.16
Belgaum	III	0.16
Bhatinda	III	0.16
Bhilai	II	0.10
Bhopal	II	0.10
Bhubaneswar	III	0.16
Bhuj	V	0.36
Bijapur	III	0.16
Bikaner	III	0.16
Bokaro	III	0.16
Bulandshahr	IV	0.24
Burdwan	III	0.16
Calicut (Kozhikode)	III	0.16
Chandigarh	IV	0.24
Chennai	III	0.16
Chitradurga	II	0.10
Coimbatore	III	0.16
Cuddalore	II	0.10
Cuttack	III	0.16
Darbhanga	V	0.36
Darjeeling	IV	0.24
Dharwad	III	0.16
Dehra Dun	IV	0.24
Dharampuri	III	0.16
Delhi	IV	0.24
Durgapur	III	0.16
Gangtok	IV	0.24
Guwahati	V	0.36
Gulbarga	II	0.10
Gaya	III	0.16
Gorakhpur	IV	0.24
Hyderabad	II	0.10
Imphal	V	0.36
Jabalpur	III	0.16
Jaipur	II	0.10
Jamshedpur	II	0.10

Town	Zone	Z	Town	Zone	Z
Jhansi	II	0.10	Patna	IV	0.24
Jodhpur	II	0.10	Pilibhit	IV	0.24
Jorhat	V	0.36	Pondicherry (Puducherry)	II	0.10
Kakrapara	III	0.16	Pune	III	0.16
Kalpakkam	III	0.16	Raipur	II	0.10
Kanchipuram	III	0.16	Rajkot	III	0.16
Kanpur	III	0.16	Ranchi	II	0.10
Karwar	III	0.16	Roorkee	IV	0.24
Kochi	III	0.16	Rourkela	II	0.10
Kohima	V	0.36	Sadiya	V	0.36
Kolkata	III	0.16	Salem	III	0.16
Kota	II	0.10	Shillong	V	0.36
Kurnool	II	0.10	Shimla	IV	0.24
Lucknow	III	0.16	Sironj	II	0.10
Ludhiana	IV	0.24	Solapur	III	0.16
Madurai	II	0.10	Srinagar	V	0.36
Mandi	V	0.36	Surat	III	0.16
Mangaluru	III	0.16	Tarapur	III	0.16
Mungher	IV	0.24	Tezpur	V	0.36
Moradabad	IV	0.24	Thane	III	0.16
Mumbai	III	0.16	Thanjavur	II	0.10
Mysuru	II	0.10	Thiruvananthapuram	III	0.16
Nagpur	II	0.10	Tiruchirappalli	II	0.10
Nagarjunasagar	II	0.10	Tiruvannamalai	III	0.16
Nainital	IV	0.24	Udaipur	II	0.10
Nashik	III	0.16	Vadodara	III	0.16
Nellore	III	0.16	Varanasi	III	0.16
Osmanabad	III	0.16	Vellore	III	0.16
Panjim	III	0.16	Vijayawada	III	0.16
Patiala	III	0.16	Vishakhapatnam	II	0.10

ANNEX F

(*Clauses* 3.12 *and* 6.3.5.3)

SIMPLIFIED PROCEDURE FOR EVALUATION OF LIQUEFACTION POTENTIAL

F-1 Due to the difficulties in obtaining and testing undisturbed representative samples from potentially liquefiable sites, *in-situ* testing is the approach preferred widely for evaluating the liquefaction potential of a soil deposit. Liquefaction potential assessment procedures involving both the SPT and CPT are widely used in practice. The most common procedure used in engineering practice for the assessment of liquefaction potential of sands and silts is the simplified procedure. The procedure may be used with either standard penetration test (SPT) blow count or cone penetration test (CPT) tip resistance or shear wave velocity V_s measured within the deposit as described below:

Step 1 — The subsurface data used to assess liquefaction susceptibility should include the location of the water table, either SPT blow count N or tip resistance q_c of a CPT cone or shear wave velocity V_s, unit weight, and fines content of the soil (percent by weight passing the IS Standard Sieve No. 75 µ).

Step 2 — Evaluate total vertical overburden stress σ_{vo} and effective vertical overburden stress σ'_{vo} at different depths for all potentially liquefiable layers within the deposit.

Step 3 — Evaluate stress reduction factor r_d using:

$$r_d = \begin{cases} 1 - 0.00765z & 0 < z \le 9.15 \text{ m} \\ 1.174 - 0.0267z & 9.15 \text{ m} < z \le 23.0 \text{ m} \end{cases}$$

where z is the depth (in metre) below the ground surface.

Step 4 — Calculate cyclic stress ratio CSR induced by the earthquake using:

$$CSR = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right)r_d ,$$

where

a_{max} = peak ground acceleration (PGA) preferably in terms of g,

g = acceleration due to gravity, and

r_d = stress reduction factor.

If value of PGA is not available, the ratio (a_{max}/g) may be taken equal to seismic zone factor Z (as per Table 3).

Step 5 — Obtain cyclic resistance ratio CRR by correcting standard cyclic resistance ratio $CRR_{7.5}$ for earthquake magnitude, high overburden stress level and high initial static shear stress using:

$$CRR = CRR_{7.5}\left(MSF\right)K_\sigma K_\alpha ,$$

where

$CRR_{7.5}$ = standard cyclic resistance ratio for a 7.5 magnitude earthquake obtained using values of SPT or CPT or shear wave velocity (as per Step 6), and

MSF = magnitude scaling factor given by following equation:

$$MSF = 10^{2.24}/M_W^{2.56}$$

This factor is required when the magnitude is different than 7.5. The correction for high overburden stresses K_σ is required when overburden pressure is high (depth > 15 m) and can be found using following equation:

$$K_\sigma = \left(\sigma'_{vo}/P_a\right)^{(f-1)}$$

where σ'_{vo} effective overburden pressure and P_a atmospheric pressure are measured in the same units and f is an exponent and its value depends on the relative density D_r. For D_r = 40 percent ~ 60 percent, f = 0.8 ~ 0.7 and for D_r = 60 percent ~ 80 percent, f = 0.7 ~ 0.6. The correction for static shear stresses K_α is required only for sloping ground and is not required in routine engineering practice. Therefore, in the scope of this standard, value of K_α shall be assumed unity.

For assessing liquefaction susceptibility using:

a) SPT, go to Step 6(a) or

b) CPT, go to Step 6(b) or

c) Shear wave velocity, go to Step 6(c).

Step 6 — Obtain cyclic resistance ratio $CRR_{7.5}$,

6(a) *Using values of SPT*

Evaluate the SPT (standard penetration test) blow count N_{60}, for a hammer efficiency of 60 percent. Specifications for standardized equipment are given in Table 11. If equipment used is of non-standard type, N_{60} shall be obtained using measured value (N):

$$N_{60} = NC_{60} ,$$

where

$$C_{60} = C_{HT}C_{HW}C_{SS}C_{RL}C_{BD} .$$

Factors C_{HT}, C_{HW}, C_{SS}, C_{RL} and C_{BD} recommended by various investigators for some common non-standard SPT configurations are provided in Table 12. For SPT conducted as per IS 2131, the energy delivered to the drill rod is about 60 percent therefore, C_{60} may be assumed as 1. The computed N_{60} is normalized to an effective overburden pressure of approximately 100 kPa using overburden correction factor C_N using:

$$\left(N_1\right)_{60} = C_N N_{60},$$

where

$$C_N = \sqrt{\frac{P_a}{\sigma'_{vo}}} \leq 1.7,$$

The cyclic resistance ratio $CRR_{7.5}$ is estimated from Fig. 8, using $(N_1)_{60}$ value.

Effect of fines content FC (in percent) can be rationally accounted by correcting $(N_1)_{60}$ and finding $(N_1)_{60CS}$ as follows:

$$\left(N_1\right)_{60CS} = \alpha + \beta\left(N_1\right)_{60},$$

where

$\alpha = 0$	$\beta = 1$	for $FC \leq 5$ percent
$\alpha = e^{\left[1.76 - \left(\frac{190}{FC^2}\right)\right]}$	$\beta = 0.99 + \frac{FC^{1.5}}{1\,000}$	for 5 percent $< FC < 35$ percent
$\alpha = 0.5$	$\beta = 1.2$	for $FC \geq 35$ percent

Again, Fig. 8 can be used to estimate $CRR_{7.5}$, where $(N_1)_{60CS}$ shall be used instead of $(N_1)_{60}$ and only SPT clean sand based curve shall be used irrespective of fines contents. The $CRR_{7.5}$ can be estimated using following equation, instead of Fig. 8:

$$CRR_{7.5} = \frac{1}{34 - \left(N_1\right)_{60CS}} + \frac{\left(N_1\right)_{60CS}}{135} + \frac{50}{\left[10 \times \left(N_1\right)_{60CS} + 45\right]^2} - \frac{1}{200}$$

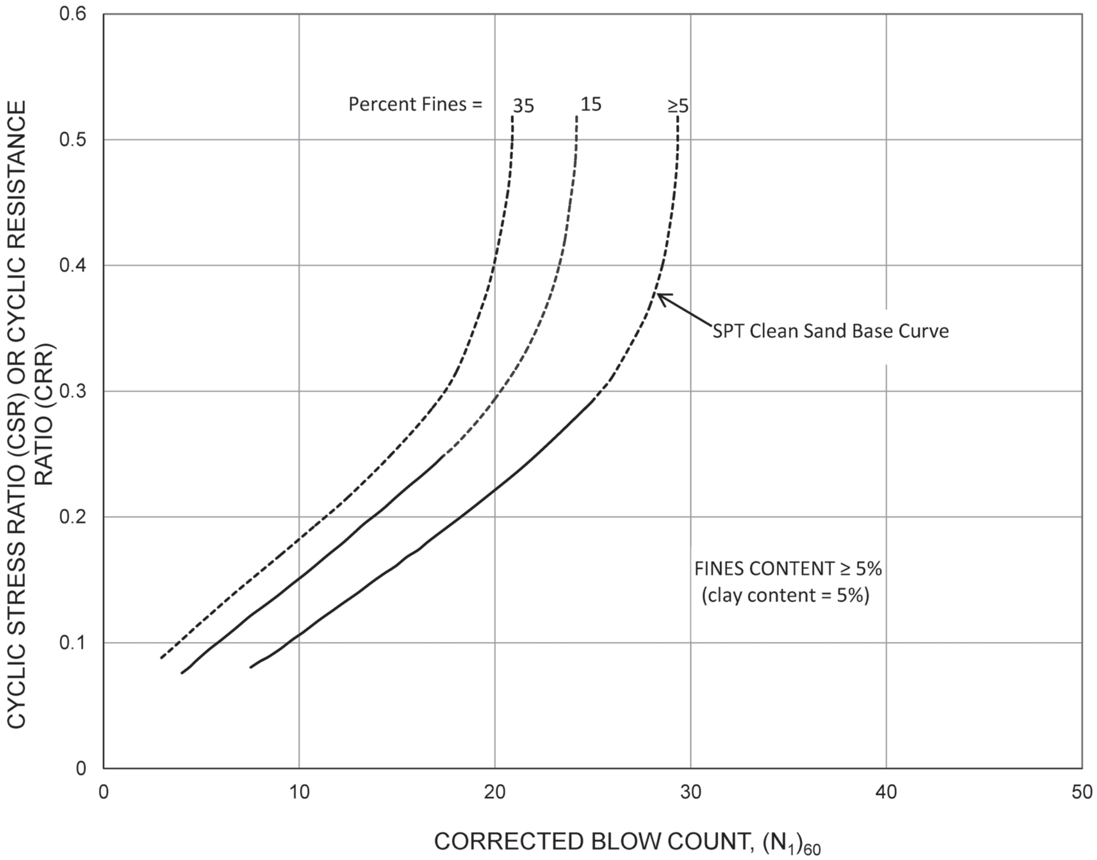

FIG. 8 RELATION BETWEEN CRR AND $(N_1)_{60}$ FOR SAND FOR M_w7.5 EARTHQUAKES

6(b) *Using values of CPT*

The CPT procedure requires normalization of measured cone tip resistance q_c using atmospheric pressure P_a and correction for overburden pressure C_Q as follows:

$$q_{C1N} = C_Q\left(\frac{q_c}{P_a}\right),$$

where q_{C1N} is normalized dimensionless cone penetration resistance, and

$$C_Q = \left(\frac{P_a}{\sigma'_{vo}}\right)^n$$

$$n = \begin{cases} 0.5 & \text{for sand} \\ 1 & \text{for clay} \end{cases}$$

The normalized penetration resistance q_{C1N} for silty sands is corrected to an equivalent clean sand value $(q_{C1N})_{CS}$ by the following relation:

$$(q_{C1N})_{CS} = k_C\, q_{C1N}$$

where

k_C = Correction factor to account for grain characteristics

$$= \begin{cases} 1.0 \quad (\text{for } I_c \leq 1.64) \\ -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75\, I_c - 17.88 \end{cases}$$

(for $I_c > 1.64$), and

$$I_c = \sqrt{(3.47 - \log Q)^2 + (1.22 - \log F)^2}$$

$$Q = \left(\frac{q_c - \sigma_{vo}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n$$

$$F = 100\left(\frac{f_s}{q_c - \sigma_{vo}}\right) \text{percent, and}$$

where f_s = measured sleeve friction.

Using $(q_{C1N})_{CS}$ find the value of $CRR_{7.5}$ using Fig. 9. Alternatively, the $CRR_{7.5}$ can be found using following equations:

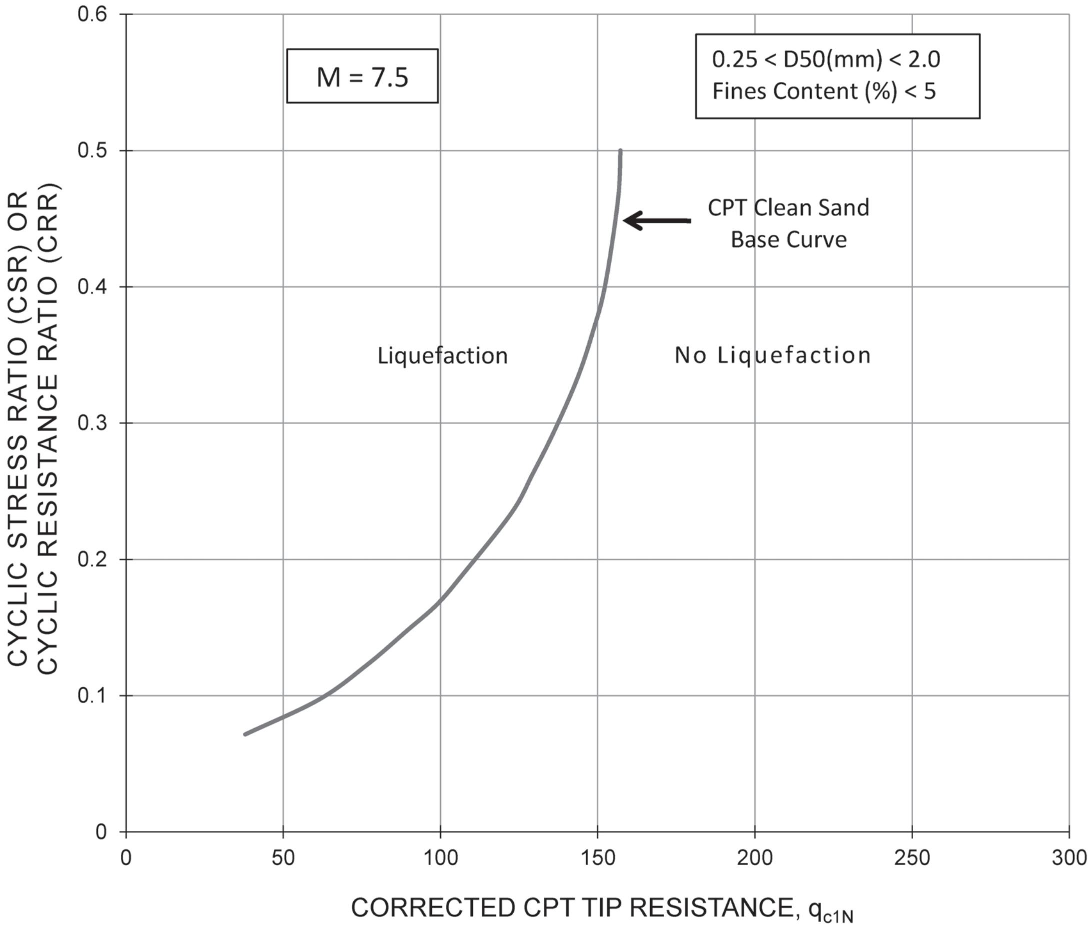

FIG. 9 RELATION BETWEEN *CRR* AND $(q_{C1N})_{CS}$ FOR M_w7.5 EARTHQUAKES

$$CRR_{7.5} = \begin{cases} 0.833\left(\dfrac{(q_{\mathrm{C1N}})_{\mathrm{CS}}}{1\,000}\right) + 0.05, & 0 < (q_{\mathrm{C1N}})_{\mathrm{CS}} < 50 \\ 93\left(\dfrac{(q_{\mathrm{C1N}})_{\mathrm{CS}}}{1\,000}\right)^3 + 0.08, & 50 \le (q_{\mathrm{C1N}})_{\mathrm{CS}} < 160 \end{cases}$$

6(c) *Using shear wave velocity:*

Apply correction for overburden stress to shear wave velocity V_s for clean sands using:

$$V_{s1} = \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.25} V_s$$

where (V_{s1}) is overburden stress corrected shear wave velocity. Using V_{s1} find the value of $CRR_{7.5}$ using Fig. 10. Alternatively, the $CRR_{7.5}$ can be found using following equation:

$$CRR_{7.5} = a\left(\frac{V_{s1}}{100}\right)^2 + b\left(\frac{1}{V_{s1}^* - V_{s1}} - \frac{1}{V_{s1}^*}\right)$$

where V_{s1}^* is limiting upper value of V_{s1} for liquefaction occurrence; a and b are curve fitting parameters. The values of a and b in Fig. 10 are 0.022 and 2.8, respectively. V_{s1}^* can be assumed to vary linearly from 200 m/s for soils with fine content of 35 percent, to 215 m/s for soils with fine contents of 5 percent or less.

Step 7 — Calculate the factor of safety *FS* against initial liquefaction using:

$$FS = \frac{CRR}{CSR},$$

where *CSR* is as estimated in Step 4 and *CRR* in Step 5. When the design ground motion is conservative, earthquake related permanent ground deformation is generally small, if $FS \ge 1.2$.

Step 8 — If $FS < 1$, then the soil is assumed to liquefy.

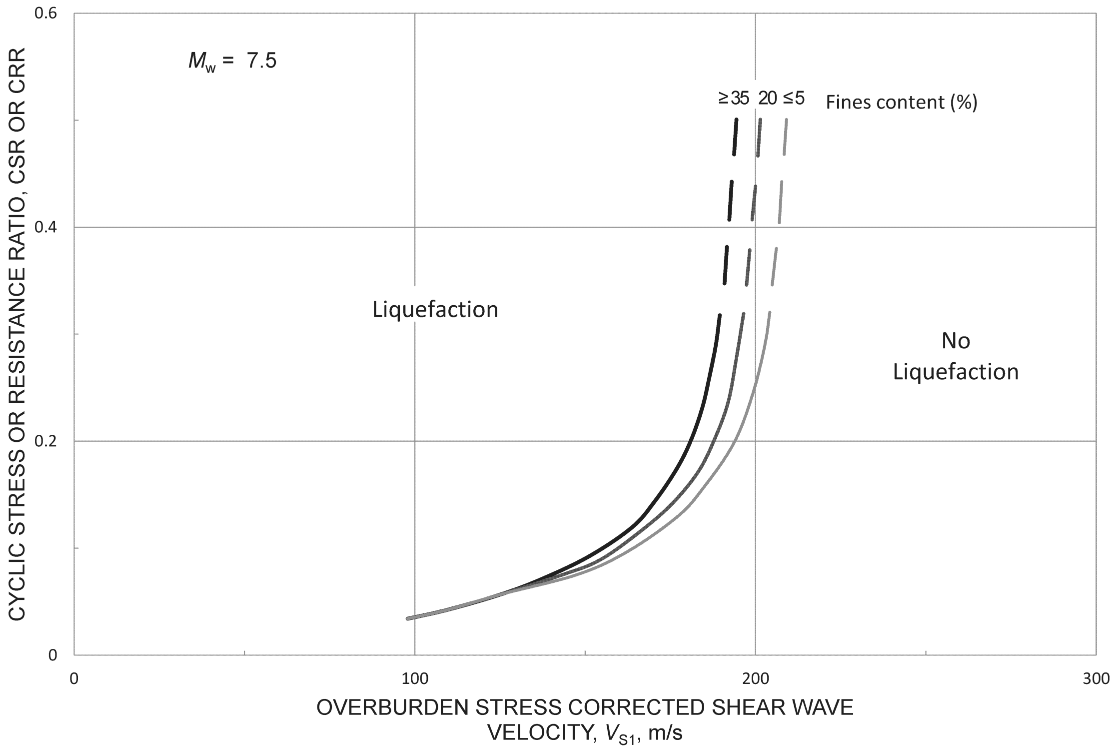

FIG. 10 RELATION BETWEEN CRR AND V_{S1} FOR M_w7.5 EARTHQUAKES

Table 11 Recommended Standardized SPT Equipment (*see* IS 2131)
[*Clause* F-1, *Step*: 6(a)]

Sl No. (1)	Element (2)	Standard Specification (3)
i)	Sampler	Standard split-spoon sampler with, outside diameter, *OD* = 51 mm; and inside diameter, *ID* = 35 mm (constant, that is, no room for liners in the barrel)
ii)	Drill rods	*A* or *AW* type for depths less than 15.2 m; *N* or *NW* type for greater depths
iii)	Hammer	Standard (safety) hammer with, a) weight = 63.5 kg; and b) drop height = 762 mm (delivers 60 percent of theoretical free fall energy)
iv)	Rope	Two wraps of rope around the pulley
v)	Borehole	100-130 mm diameter rotary borehole with bentonite mud for borehole stability (hollow stem augers where SPT is taken through the stem)
vi)	Drill bit	Upward deflection of drilling mud (tricone or baffled drag bit)
vii)	Blow count rate	30 to 40 blows per minute
viii)	Penetration resistant count	Measured over range of 150 mm – 450 mm of penetration into the ground

Table 12 Correction Factors for Non-Standard SPT Procedures and Equipment
[*Clause* F-1, *Step*: 6(a)]

Sl No. (1)	Correction for (2)	Correction Factor (3)
i)	Non-standard hammer weight or height of fall	$C_{HT} = \begin{matrix} 0.75 \text{ (for Donut hammer with rope and pulley)} \\ 1.33 \text{ (for Donut hammer with trip/auto)} \end{matrix}$ and Energy ratio = 80 percent
ii)	Non-standard hammer weight or height of fall	$C_{HW} = \dfrac{HW}{48\,387}$ where H = height of fall (mm), and W = hammer weight (kg)
iii)	Non-standard sampler setup (standard samples with room for liners, but used without liners)	$C_{SS} = \begin{cases} 1.1 \text{ (for loose sand)} \\ 1.2 \text{ (for dense sand)} \end{cases}$
iv)	Non-standard sampler setup (standard samples with room for liners, but liners are used)	$C_{SS} = \begin{cases} 0.9 \text{ (for loose sand)} \\ 0.8 \text{ (for dense sand)} \end{cases}$
v)	Short rod length	$C_{RL} = \begin{cases} = 0.75 \text{ (for rod length 0-3 m)} \\ = 0.80 \text{ (for rod length 3-4 m)} \\ = 0.85 \text{ (for rod length 4-6 m)} \\ = 0.95 \text{ (for rod length 6-10 m)} \\ = 1.0 \text{ (for rod length 10-30 m)} \end{cases}$
vi)	Nonstandard borehole diameter	$C_{BD} = \begin{cases} 1.00 \text{ (for bore hole diameter of 65-115 mm)} \\ = 1.05 \text{ (for bore hole diameter of 150 mm)} \\ = 1.15 \text{ (for bore hole diameter of 200 mm)} \end{cases}$

NOTES

1 N = Uncorrected SPT blow count.

2 $C_{60} = C_{HT} C_{HW} C_{SS} C_{RL} C_{BD}$

3 $N_{60} = N C_{60}$

4 C_N = Correction factor for overburden pressure $(N_1)_{60} = C_N C_{60} N$.

ANNEX G

(*Foreword*)

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