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Final Report: IS 1893 Code and Commentary
IITGN-World Bank Project on Seismic Codes

Proposed Modifications for Code on Criteria for Earthquake Resistant Design of Structures IS 1893: 2016 (Part 1- General Provisions and Buildings)

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- This document has been developed under the World Bank-sponsored Project on Improving Seismic Resilience of Built Environment in India at the Indian Institute of Technology Gandhinagar.
- This report presents the proposed modifications and commentary for the code on Criteria for Earthquake Resistant Design of Structures IS 1893: 2016(Part 1- General Provisions and Buildings).Proposed revisions in the code have been shown with strikethrough font when the content has been removed, or underlined when new content has been added.
- Some of the commentary content in this document has been taken from IITK-GSDMA document Proposed Draft Provisions and Commentary on Indian Seismic Code IS 1893 (Part 1) (EQ05- V.4.0 and EQ15-V3.0) (https://www.nicee.org/IITK-GSDMA_Codes.php). However, the original commentary has been significantly revised and expanded to address the current IS 1893 (Part 1): 2016 and proposed changes.
- The views and opinions expressed are those of the authors and not necessarily of the World Bank, IIT Gandhinagar, IIT Kanpur, or the Bureau of Indian Standards.
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- A seminar-cum-workshop was organized at IIT Gandhinagar, wherein more than 180 academicians, practising engineers and students participated from across the country to publicly discuss the proposed modifications in seismic codes IS 1893 (Part 1) and IS 13920, in addition to discussing the codal compliance in seismic design of a few real-life buildings. We sincerely appreciate their time and effort which proved extremely helpful in revising the report.

Foreword

In the seventh revision, a number of improvements made in the code; the significant changes are:

a) Design spectra are defined specifically for all three acceleration, velocity and displacement regions;

b) Design spectrum for the vertical earthquake response is included;

c) Definition of base is expanded to consider seismic load transfer mechanism from the building to the soil.

d) Simplified provision for including vertical earthquake effects at member and global level is included;

e) Provisions for the combination of different lateral force resisting structural systems;

f) Specific treatment for different types of irregularity has been rationalized and elaborated;

g) Improved relation for calculating approximate fundamental period of buildings with structural walls;

h) Minimum design base shear is enhanced in view of new findings;

i) Forces for strength design of diaphragm components is included;

j) Scope for the use of equivalent static method for analysis is defined;

k) Detail requirements for the response history analysis has been included;

l) Masonry infill modeling parameters has been revised and contribution of infill masonry to lateral strength has been restricted;

m) Explicit and rational treatment for plan and vertical irregularities including open ground story in masonry infilled frames;

n) Use of flat slab (slab without beams) as primary seismic force resisting system for buildings is included;

o) Separation requirements between adjacent buildings has been modified

p) Specific design requirements for temporary and parking structures;

q) Provisions for anchorage and support of non-structural elements;

r) New parameters for soil classification have been introduced.

s) Improvements in the simplified method for evaluation of liquefaction potential and design requirements for piles passing through liquefiable soil layers.

However, a number of issues have been identified which need to be addressed in future revisions of the code. These are summarized in the following:

a) List of definitions should include only those terms that have a more specific use and/or are not widely explained in the literature. Terms which are widely available in textbooks (like ‘damping’ or ‘critical damping’) and hence known to all qualified engineers, should be removed.

b) Design spectra should be expressed in terms of S_a, not in the non-dimensional form of S_a/g. Similarly, the seismic zones should be expressed in terms acceleration values as in the other international codes.

c) Many clauses contain non-normative content and mix with normative statements. They should be clearly identified and separated. Non-normative content can also be moved to commentary when it becomes the integral part of the code.

d) A more detailed treatment of SSI is should be provided.

e) The definition of dual system should be revised based on an easy assessment of the load sharing between the constituent lateral force resisting systems.

f) The parabolic distribution of floor forces of buildings of all heights for the Equivalent Static Method should be revisited. It should vary from linear for low-rise (fundamental period less than 1.5 s) to parabolic for buildings with fundamental period greater than 2.5 s.

g) The provisions of pounding should be be revisited especially for cases of adjacent moment frames with slabs at same elevations. For such cases, 30% reduction in seismic joint size can be considered while calculating lateral deflections.

1 – Scope

1.1

This standard(Part 1) primarily deals with assessment of seismic loads ~~earthquake hazard assessment~~ for earthquake-resistant design of various structures such as(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, temporary excavations need to be designed for earthquake forces.~~

3 – Terminology

3.5 Design Basis Earthquake

It is the earthquake level that forms the general basis of earthquake resistant design of structures as per the provision of this code. For normal structures, the code assumes the effect of the design basis earthquake motion to be one half of that due to the maximum considered earthquake motion.

3.11 Geotechnical Field Test Parameters

Foundation design and liquefaction assessment need at least one the following three field test parameters

a) *N*-value: Value of Standard Penetration Test at a depth, to which corrections are applied as per Annex~~ure~~ – F.
b) q_c value: Tip resistance from Cone Penetration Test at a depth
c) V_s value: Shear wave velocity of soil strata at a depth

~~3.12~~

3.14 Liquefaction

It is a state primarily of 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~~ decreases due to building of pore water pressure during earthquake shaking. ~~In this condition, the soil tends to behave like a fluid mass~~(see Annex G).

3.16 Maximum Considered Earthquake (MCE)

The most severe earthquake considered by this standard.

4 Special Terminology for Buildings

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.~~

It is level at which the horizontal earthquake ground motions are considered to be imparted to the structure.

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.~~

4.14 Moment-Resisting Frame

4.14.3– Intermediate Moment Resisting Frame (IMRF)

It is a moment-resisting frame with lower ductility and relaxed detailing requirements compared to SMRF and intended for application in lower seismic zones.

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.~~

4.16 Principal Orthogonal Plan (Horizontal) Axes

These are two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented that overlay the majority of seismic force resisting system.

~~4.19~~

4.17 RC Structural Wall

4.18.3–Intermediate Structural Wall

It is a structural wall with lower ductility and relaxed detailing requirements compared to Special Structural Wall and intended for application in lower seismic zones.

~~4.20~~

4.18 Storey

~~4.20.1~~

4.19.1– Soft Storey

It is one in which the lateral stiffness is less than70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys 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~~

4.19.2– 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

70 percent of 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.

~~4.19 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

A_B Area of base of the structure (in m^2)

a_p Component amplification factor

A_{wi} Effective cross-sectional area of structural wall *i* in the considered direction of lateral forces (in m^2)

C_w Effective structural wall area factor for calculation of natural period

D_p Seismic relative distance

F_p Design seismic force on a nonstructural element

h_{sx} Storey height below level x

h_x Height of level x to which upper connection point is attached

h_y Height of level y to which lower connection point is attached

I_p Importance factor of the nonstructural element

L_{wi} Length of structural wall i in the considered direction of lateral forces (in metre)

N measured (raw) SPT blow count

N_{60} Normalized SPT blow count for 60% energy efficiency

$(N_1)_{60}$ SPT blow count normalized for vertical effective stress of 1 atmosphere (i.e., about 100 kPa) and delivery of 60 % of theoretical hammer energy

$(N_1')_{60}$ Dilatancy corrected SPT blow count normalized for vertical effective stress or 1 atmosphere and delivery of 60 % of theoretical hammer energy

N_w Number of walls in the considered direction of earthquake shaking

R_p Component response modification factor

T_v Undamped natural period of vertical oscillation of the structure (in second)

V_B ~~Design seismic base shear~~ Design base shear calculated using the approximate fundamental period T_a

$\bar{V}_B$ Design base shear calculated using the ~~approximate fundamental period T_a~~ dynamic analysis method.

W_p Weight of the nonstructural element

x Height of point of attachment of the nonstructural element above top of the foundation of the building

Δ_{aA} Allowable storey drift for structure A

Δ_{aB} Allowable storey drift for structure B

δ_{xA} Deflection at building level x of structure A due to design seismic load

δ_{yA} Deflection at building level y of structure A due to design seismic load

δ_{yB} Deflection at building level y of structure B due to design seismic load

6 General Principles and Design Criteria

6.1 General Principles

6.1.3 –

The design approach adopted in this standard is to ensure that structures possess at least a minimum strength to withstand minor earthquakes (less than DBE), which occur frequently, without damage; resist moderate earthquakes (DBE) without significant structural damage though some non-structural damage may occur; and aims that structures withstand a major earthquake (MCE) without collapse.

~~Actual forces that appear on structures during earthquakes are much higher than the design forces specified in the standard~~. Actual ground accelerations that structures are subjected during earthquakes are much higher than the design accelerations specified in the standard. Consequently they are likely to experience inelastic deformations. 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 to account 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.

6.1.5 –

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

SSI effects may be considered for important and special structures.

6.1.7 – Additions to Existing Structures

Additions shall be made to existing structures only as follows, except for those structures in which provisions of additions had already been made:

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~~10percent, unless the capacity of the element

subject to the increased force is still in compliance with this standard, and

3. The 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 ~~importance factor~~*(I)*, the structure shall conform to the seismic requirements laid down for a new structure with the higher importance factor.

6.2 Assumptions

b) Earthquake is not likely to occur simultaneously with high wind, maximum flood or maximum sea waves. Therefore, these should not be combined with earthquake loads.

6.3 Load Combinations and Increase in Permissible Stresses.

6.3.2 – Design Horizontal Earthquake Load

6.3.2.1 –

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

For structural elements which are part of intersecting lateral force resisting systems of both orthogonal directions, the design actions shall be determined as per 6.3.2.2.

6.3.3 – Design Vertical Earthquake Load

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;~~

~~b) Bridges;~~

~~e)~~a)Structure has spans larger than 20 m; or

~~f)~~b)Structure has large horizontal overhangs of structural members or sub-systems,

more than 5 m, or

c) Prestressed beams and slabs.

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 and combined with horizontal seismic forces as per load combinations specified in 6.3.4.

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.~~

As an alternative to 6.3.3.2, a factor of 0.3 *ZI* on the dead loads either in additive or in counteracting manner can account for the vertical accelerations of ground motions and shall be combined with the effects of horizontal components of ground motions (obtained from Equivalent Static Method as per section 7.6 or Dynamic methods as per 7.7) using the following load combinations:

a) $1.2\ [(1.0 + 0.3\ ZI)\ DL + IL \pm (EL_X \pm 0.3\ EL_Y)]$

$1.2\ [(1.0 + 0.3\ ZI)\ DL + IL \pm (EL_Y \pm 0.3\ EL_X)]$

$1.2\ [(1.0 + ZI)\ DL + IL \pm (0.3\ EL_Y \pm 0.3\ EL_X)]$

b) $1.5\ [(1.0 + 0.3\ ZI)\ DL \pm (EL_X \pm 0.3\ EL_Y)]$

$1.5\ [(1.0 + 0.3\ ZI)\ DL \pm (EL_Y \pm 0.3\ EL_X)]$

$1.5\ [(1.0 + ZI)\ DL \pm (0.3\ EL_Y \pm 0.3\ EL_X)]$

c) $0.9\ [(1.0 - 0.3\ ZI)\ DL] \pm 1.5\ (EL_X \pm 0.3\ EL_Y)$

$0.9\ [(1.0 - 0.3\ ZI)\ DL] \pm 1.5\ (EL_Y \pm 0.3\ EL_X)$

$0.9\ [(1.0 - ZI)\ DL] \pm 1.5\ (0.3\ EL_Y \pm 0.3\ EL_X)$

6.3.4 –*Combination 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 the three components (EL_X, EL_Y, 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) $\pm EL_Y \pm 0.3\ EL_X \pm 0.3\ EL_Z$~~
~~c) $\pm EL_Z \pm 0.3\ EL_X \pm 0.3\ EL_Y$~~

~~where X and Y are two orthogonal plan directions and Z vertical direction.Thus, *EL* in the above referred load combinations shall be replaced by ($EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z$), ($EL_Y \pm 0.3\ EL_X \pm 0.3\ EL_Z$)or ($EL_Z \pm 0.3\ EL_X \pm 0.3\ EL_Y$). This implies that~~ Thus, for analysis using

Response History Method and Response Spectrum Method, the sets of load combinations involving earthquake effects to be considered shall be as given below:

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

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

3) 0.9 DL ± 1.5[$(EL_X \pm 0.3\ EL_Y \pm 0.3\ EL_Z)$], 0.9 DL ± 1.5[$(EL_Y \pm 0.3\ EL_X \pm 0.3\ EL_Z)$] and
   0.9 DL ± 1.5[$(EL_Z \pm 0.3\ EL_X \pm 0.3\ EL_Y)$]

6.3.5 – Special Load Combinations for Amplified Seismic Loads

The following special load combinations are required to ensure adequate strength in brittle elements of the structural system.

1.2 [(1.0+0.3 ZI) DL + IL] ± 0.5R ($EL_X \pm 0.3\ EL_Y$)

1.2 [(1.0+ 0.3 ZI) DL + IL] ± 0.5R ($EL_Y \pm 0.3\ EL_X$)

1.2 [(1.0+ ZI) DL + IL] ± 0.5R (0.3 $EL_Y \pm 0.3\ EL_X$)

~~6.3.5~~6.3.6– Increase in Allowable Net Bearing Pressure on Soils in Design of Foundations

~~6.3.5.2~~ 6.3.6.2–

When earthquake forces are included, net allowable 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~~ three 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.~~

Table 1 - Percentage Increase in Net Allowable Bearing Pressure and Skin Friction of Soils

(Clause 6.3.~~5~~6.2)

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

NOTES

1. The net allowable bearing pressure shall be determined in accordance with IS 6403 or ~~IS 1888,~~IS 8009.

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

2. If any increase in net allowable 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. Desirable minimum field values of N~~~~$(N_1')_{60}$~~~~shall be as specified below:~~

~~Seismic Zone Level~~	~~Depth below Ground (in meters)~~	~~$(N_1')_{60}$ Values~~	~~Remarks~~
~~III, IV and V~~	~~≤ 5~~ ~~≥ 10~~	~~15~~ ~~25~~	~~For values of depths between 5m and 10m, linear interpolation is recommended.~~
~~II (for important Structures only)~~	~~≤ 5~~ ~~≥ 10~~	~~10~~ ~~20~~	

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

~~5. The piles should be designed for lateral loads neglecting lateral resistance of 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 $CN = \sqrt{P_a/\sigma'_{vo}} \leq 1.7$; P_a is the atmospheric pressure and N_1 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 group, to depth of bottom tip of pile group plus twice the width of pile group; and~~

~~d) Well foundation – Weighted average of N of soil layers from depth of bottom tip of well, to the 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~~6.2)

Soil Type	Corrected SPT value $(N_1')_{60}$	Tip resistance of CPT q_c/P_a	Shear wave velocity V_s (m/s)
Type – A	>30	> 100	> 360
Type – B	15-30	50 –100	180 – 360
Type – C	< 15	< 50	< 180

Note: The value of $(N_1')_{60}q_c/P_a$, or V_s to be used shall be the weighted average of these values of individual soil layers from the existing ground level to 30 m below the existing ground level.

~~Soil Type~~	~~Definition~~
~~Type A: Rock or Hard Soils~~	~~Well graded gravel (GW) or well graded sand (SW) both with less than 5% passing 75 µm 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 the corrected Standard Penetration Test value.~~
~~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.~~
~~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 compressibility (MI-MH or CI-CH);~~ ~~Silt with Clay of Intermediate compressibility (MI-CI);~~ ~~Silt with Clay of High compressibility (MH-CH).~~
~~Type D: Unstable, collapsible, liquefiable soils~~	~~Requires site specific study and special treatment according to site condition (*see* 6.3.5.3)~~

~~6.3.5.3~~ 6.3.6.3 –

In soil deposits consisting of submerged loose, cohesionless soils, e.g., sands and silty sands and marine clay ~~soils falling under classification SP~~ have potential to liquefy during earthquake loading.~~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~~.

Liquefaction potential need not be investigated when the following conditions are met by the field values of $(N'_1)_{60}$ below design water table:

% passing 75 μm IS sieve	$(N_1')_{60}$
≤ 5	>30
> 5 and ≤ 15	>25
> 15 and ≤ 35	>21

In case the above mentioned condition is not satisfied, a simplified method for assessment of liquefaction potential as given in Annex G should be adopted.The liquefiable sites should be avoided preferably for locating new structures and should be avoided for locating structures of important projects. Otherwise, earthquake~~-~~related settlements need to be estimated and appropriate methods shall be adopted ~~of~~ for compaction or stabilization to achieve the minimum desired $(N_1')_{60}$-values as indicated ~~in Note 4 of Table 1~~ above. Alternatively, deep pile foundations may be adopted and ~~anchored~~ founded at depths well below the underlying soil layers, which are likely to liquefy or undergo excessive settlements.

For liquefaction mitigation, consider ground improvement by compacting the soil to increase its penetration resistance $(N_1')_{60}$ within the range given above for which liquefaction potential is not to be investigated. Alternatively, suitable drainage can be provided to minimize the likelihood of pore-water pressure generation due to ground shaking.

Provision of pile foundations needs due caution since they would have to transfer forces despite the loss of soil support in the liquefiable layer(s).

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.~~

6.3.6.4 –Simplified pile design loads through liquefiable layer

Liquefaction can often lead to lateral spreading of ground in case of large sloping ground near water bodies and rivers, especially in case of water front and free face situations. A site with average sloping ground steeper than 5˚ must be given due consideration to this phenomenon. In such conditions, it is generally expected to consider ground improvement for mitigation of liquefaction potential to avoid large differential settlements and damage to the structure.

In case, the structure is to be provided with piles without liquefaction mitigation, the loads due to lateral spreading are to be considered for checking the lateral capacity of pile, while the seismic inertial force is ignored for this analysis. The lateral force on pile is found by considering passive earth pressure acting in non-liquefiable layers and 30% of overburden pressure acting as horizontal force in liquefiable layers.

6.3.6.5 –

Vertical capacity of pile can be obtained considering 20% of shear strength in liquefiable layers.

6.4 Design Acceleration Spectrum

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.~~

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

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

S_a/g = 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 applicable for all structures, if not specified otherwise ~~as given~~ in other 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:

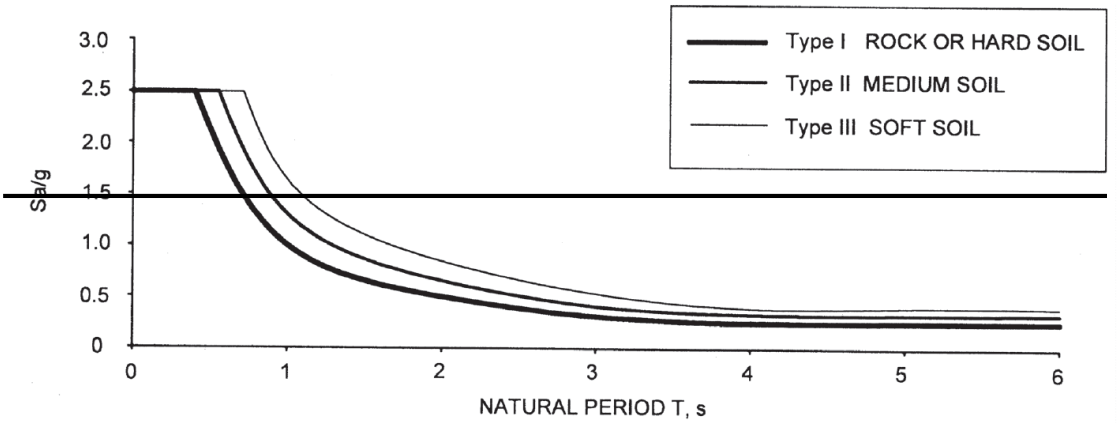
~~a) For use in equivalent static method [see Fig. 2(a)]:~~

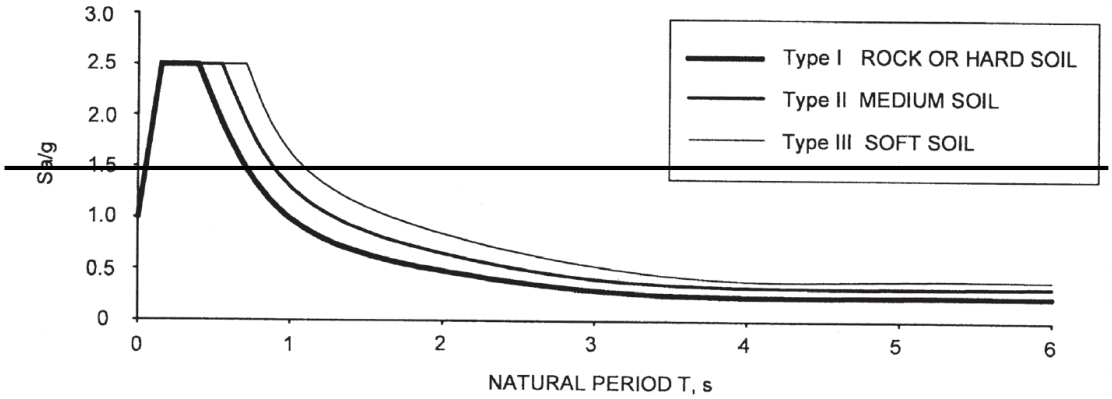
$$\frac{S_a}{g} = \begin{cases} \text{For rocky or hard soil sites} & \begin{cases} 2.5 & 0 < T < 0.40 \text{ s} \\ \dfrac{1}{T} & 0.40 \text{ s} < T < 4.00 \text{ s} \\ 0.25 & T > 4.00 \text{ s} \end{cases} \\ \text{For medium stiff soil sites} & \begin{cases} 2.5 & 0 < T < 0.55 \text{ s} \\ \dfrac{1.36}{T}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

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

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

$$\frac{S_a}{g} = \left\{ \begin{array}{ll} \text{For rock or hard soil sites} & \left\{ \begin{array}{ll} 1+15T & 0 < T < 0.10s \\ 2.5 & 0.10 < T < 0.40s \\ \dfrac{1.00}{T} & 0.40 < T < 4s \\ \dfrac{4.00}{T^2} & T > 4s \end{array} \right\} \\ \text{For medium stiff soil sites} & \left\{ \begin{array}{ll} 1+15T & 0 < T < 0.10s \\ 2.5 & 0.10 < T < 0.55s \\ \dfrac{1.36}{T} & 0.55 < T < 4s \\ \dfrac{5.44}{T^2} & T > 4s \end{array} \right\} \\ \text{For soft soil sites} & \left\{ \begin{array}{ll} 1+15T & 0 < T < 0.10s \\ 2.5 & 0.10 < T < 0.67s \\ \dfrac{1.67}{T} & 0.67 < T < 4s \\ \dfrac{6.68}{T^2} & T > 4s \end{array} \right\} \end{array} \right\}$$





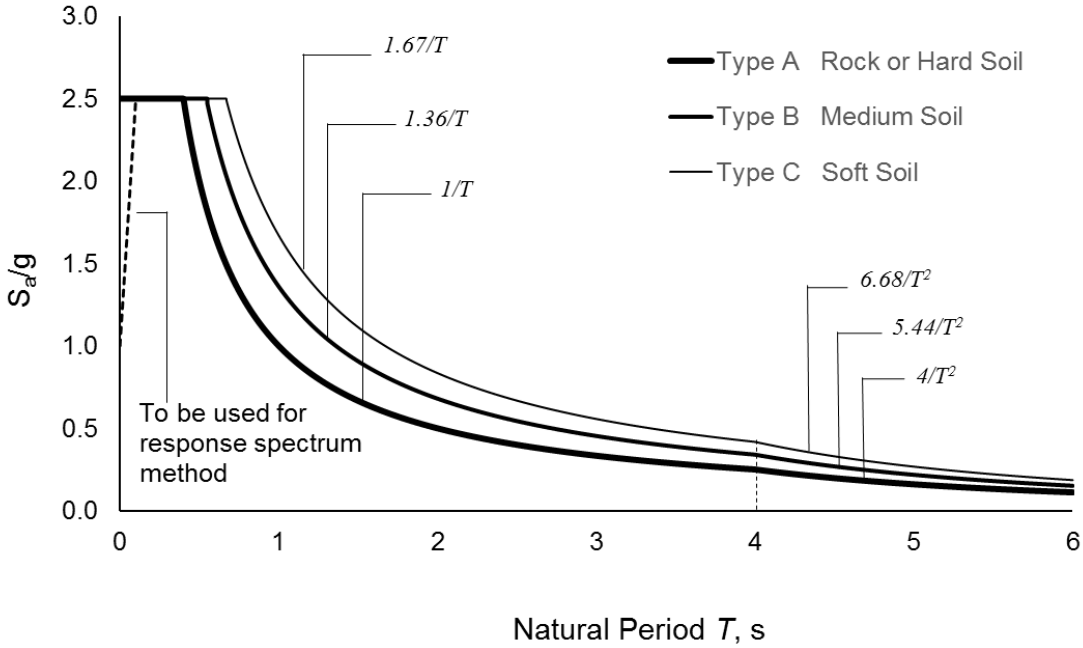


Figure 2 – Design acceleration coefficient (S_a/g) <u>for horizontal acceleration</u> (corresponding to 5 percent damping)

6.4.2.1

Figure 2 shows the 5 percent spectra for different soil sites and Table 4 gives the multiplying factors for obtaining spectral values for various other damping.

Table 4 - Multiplying factor for obtaining (S_a/g) values for other damping[1)]

(Clause 6.4.2.1)

Damping (%)	Factors
0	3.20
2	1.40
5	1.00
7	0.90
10	0.80
15	0.70
20	0.60
25	0.55
30	0.50

[1)] The multiplying factor for different damping values is not to be applied to the point at zero period.

~~6.4.2.1~~

6.4.2.2

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~~2, as:

a) Soil type ~~I~~A — Rock or hard and dense soils;

b) Soil type ~~II~~B — Medium dense or stiff soils; and

c) Soil type ~~III~~C — Soft and loose soils.

~~In Table 1, 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.~~

~~Table4ClassificationofTypesofSoilsforDeterminingtheSpectrumtobeUsedtoEstimateDesignEarthquakeForce~~

~~(Clause6.4.2.1)~~

~~SlNo.~~ ~~(1)~~	~~Soil Type~~ ~~(2)~~	~~Remarks~~ ~~(3)~~
~~i)~~	~~I~~ ~~Rockor Hardsoils~~	~~a) Wellgradedgravel(GW)orwellgradedsand(SW)bothwithlessthan5percentpassing75 msieve (Fines)~~ ~~b) Well graded gravel-sand mixtures with or without fines (GW-SW)~~ ~~c) Poorlygradedsand(SP)orclayeysand(SC),allhavingNabove30~~ ~~d) StifftohardclayshavingNabove30,whereNisstandardpenetrationtestvalue~~
~~ii)~~	~~II~~ ~~Mediumor Stiffsoils~~	~~a) Poorlygradedsandsorpoorlygradedsandswithgravel(SP)withlittleornofineshavingNbetween10and30~~ ~~b) Stifftomediumstifffine-grainedsoils,likesiltsoflowcompressibility(ML)orclaysoflow compressibility(CL)havingNbetween10and30~~
~~iii)~~	~~III~~ ~~Soft Soils~~	~~AllsoftsoilsotherthanSPwithN<10.Thevariouspossiblesoilsare:~~ ~~Softsoils a)~~ ~~Siltsofintermediatecompressibility(MI);~~ ~~b) Siltsofhighcompressibility(MH);~~ ~~c) Claysofintermediatecompressibility(CI);~~ ~~d) Claysofhighcompressibility(CH);~~ ~~e) Siltsandclaysofintermediatetohighcompressibility(MI-MHorCI-CH);~~ ~~f)) Siltwithclayofintermediatecompressibility(MI-CI);and~~

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~~ response history method, and

3) ~~Time~~ Response history method.

In this standard, Equivalent Static Method, Response Spectrum Method and ~~Time~~ Response History Method are 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~~ frames: 70 percent of I_{gross} of columns, and 35 percent of I_{gross} of beams;

b) For RC and masonry walls: 50 percent of I_{gross};

c) For RC slabs: 25 percent of I_{gross}; and

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

However, no reduction is required for axial and torsional stiffnesses.

6.4.6 –

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

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

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

where, S_a/g is spectral acceleration spectrum for vertical motion for all soil types as a function of vertical period of vibration, T_v (Figure 3):

$$\frac{S_a}{g} = \begin{cases} 1+\frac{100}{3}T_v & 0 < T_v < 0.05s \\ \frac{8}{3} & 0.05 < T_v < 0.15s \\ \frac{0.4}{T_v} & T_v > 0.15s \end{cases}$$

~~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**.~~For vertical excitation, R values should be taken based on expected energy dissipation based on rational analyses. In the absence of such information, R shall be taken as 1.0 for essentially elastic response.

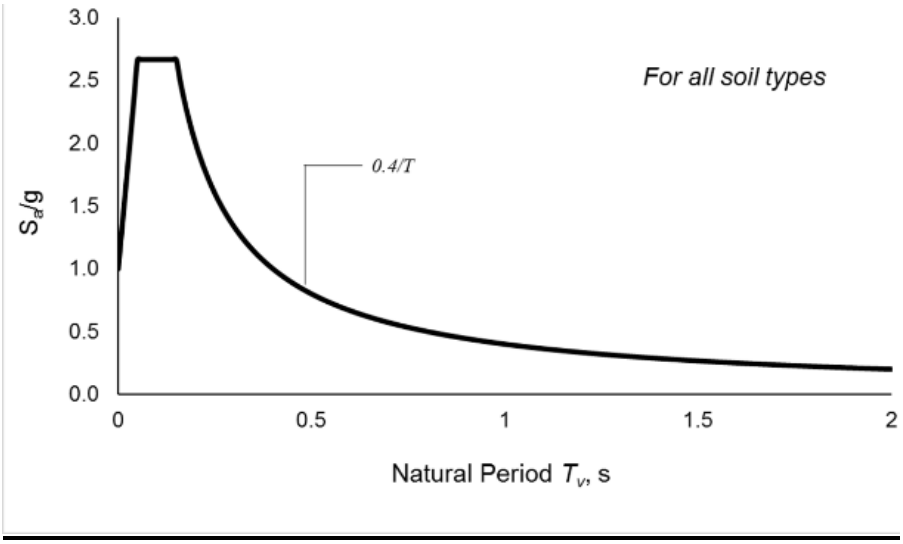


Figure 3 – Design acceleration coefficient (S_a/g) for vertical acceleration (corresponding to 5% damping)

7 Building

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

a) Complete load path and robust structural configuration,

b) At least a minimum elastic lateral stiffness,

c) At least a minimum lateral strength, and

d) Adequate ductility.

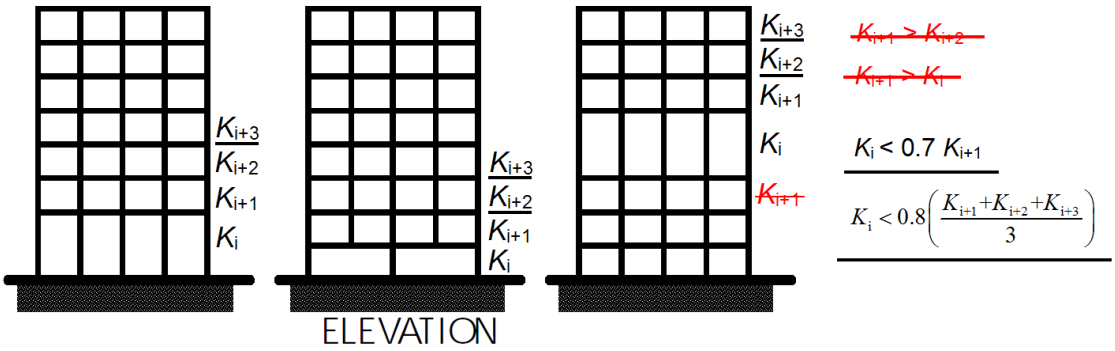
7.1 Regular and Irregular Configuration

Table 6 – Definition of irregular buildings – Vertical irregularities (Figure 4~~5~~)

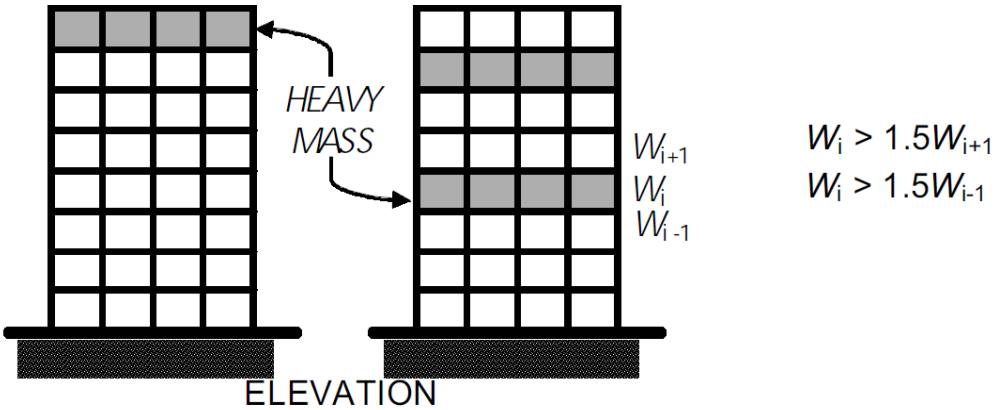
(*Clause* 7.1)

i) *Stiffness Irregularity (Soft Storey)*# ~~A soft storey is a storey whose lateral stiffness is less than that of the storey above.~~ A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above. Stiffness irregularities shall not to be considered 1. for one storey buildings in any Seismic zone and 2 storey buildings in Seismic Zone III, IV and V. 2. when no storey shall have a drift more than 130 % of the drift of the storey above. Torsional effects need not be taken into account while calculating drifts. Drift ratios of top two storeys need not be considered for this purpose. ~~*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 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.*~~ *Dynamic analysis methods of section 7.7 shall be be used for buildings ~~-~~and vertical elements of soft storey shall be designed for load effects as per special load combinations in 6.3.5.*
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 adjacent floors ~~below~~(Exception: A roof that is lighter than the floor below). Mass irregularities shall not be considered for buildings upto two storey. ~~*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)*~~ *Buildings with mass irregularities shall be dealt with in the same way as buildings with stiffness irregularities of Table 6.*
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~~150 percent of adjacent 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)*~~ *Buildings with vertical geometric irregularity shall be dealt with in the same manner as buildings with out of plane offsets for plan irregularities of Table 5.*
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 and cause overturning moment demands on supporting elements. ~~*In buildings with in-plane discontinuity and located in Seismic Zone II, the lateral drift of the building under 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.*~~ *Buildings with In-plane discontinuity in lateral force resisting vertical elements shall be dealt*

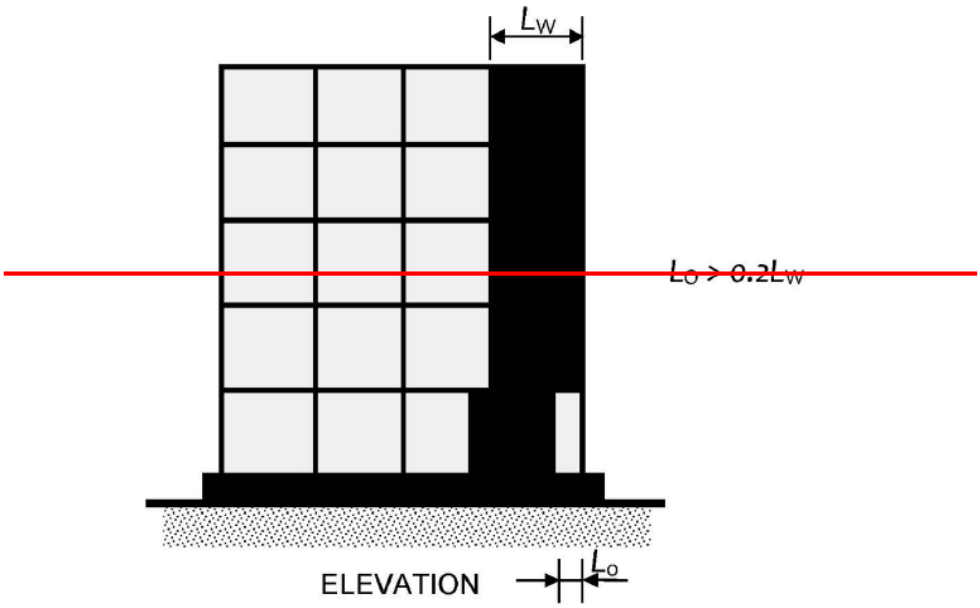
with in the same manner as buildings with out of plane offsets for plan irregularities in Table 5.
v) *Strength Irregularity (Weak Storey)*+ A weak storey is a storey whose lateral strength is less than 70 percent of that ~~of~~ in the storey above. ~~*In such a case, buildings in Seimic Zones III, IV and V shall be designed such that safety of the building is not jeopardized; also, provisions of 7.10 shall be followed*~~ *Except for building upto two storeys, weak storeys shall not be permitted in zone III, IV and V.*
~~**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 a part of or supporting the primary lateral load resisting system.*~~
~~**vii) Irregular Modes of Oscillation in Two Principal Plan Diirections**~~ ~~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~~ ~~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 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 contribute less than 65 percent mass participation factor in each principal plan direction, and~~ ~~*2) the* fundamental lateral natural periods of the building in the two principal directions are away from each other by at least 10 percent of the larger value.~~

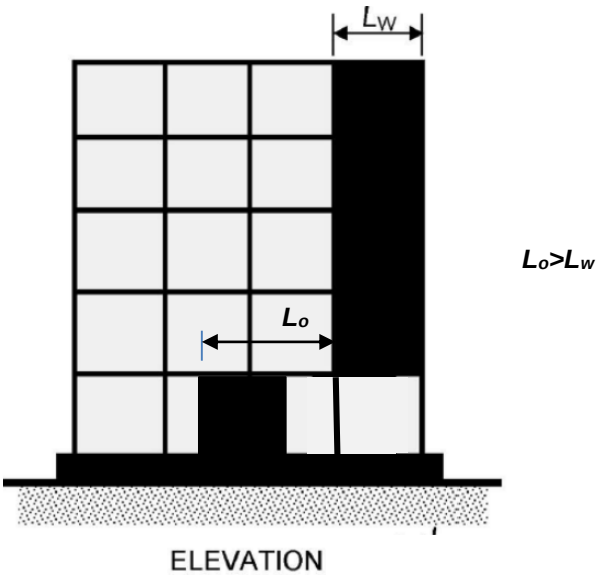


45A STIFFNESS IRREGULARITY

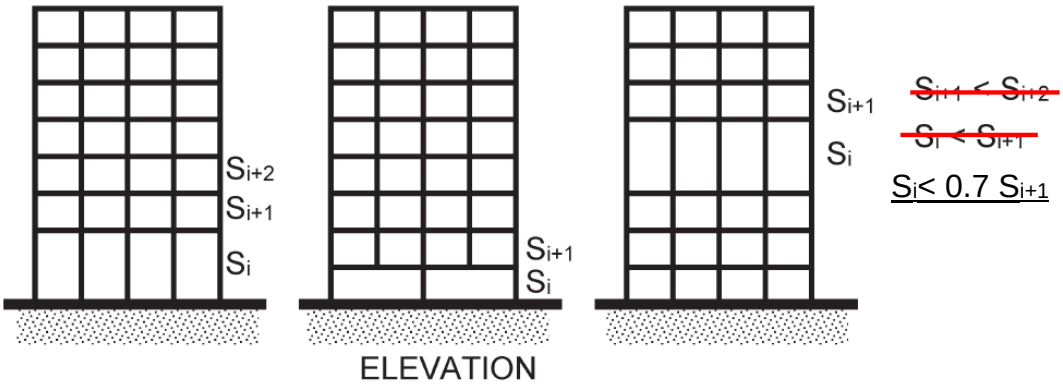


45B MASS IRREGULARITY





45D IN-PLANE DISCONTINUITY IN VERTICAL ELEMENTS RESISTING LATERAL FORCE



45E STRENGTH IRREGULARITY (WEAK STOREY)

FIGURE 5 – DEFINITIONS OF IRREGULAR BUILDINGS – VERTICAL IRREGULARITIES

7.2 Lateral Force

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~~ by:

$$(V_B)_{\min} = \frac{\rho IW}{100}$$

where ρ is defined in Table 7.

Table 7 Minimum Design Earthquake Horizontal Lateral Force for Building

Sl. No. (1)	Seismic Zone (2)	ρ ~~percent~~ (3)
i)	II	~~0.7~~0.8
ii)	III	~~1.1~~1.3
iii)	IV	~~1.6~~2.0
iv)	V	~~2.4~~3.0

7.2.6 Response Reduction Factor

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 lateral load resisting elements provided within the buildings. ~~and NOT for just the lateral load resisting elements, which are built in isolation.~~

Table 9 – Response Reduction Factor, *R*, for Building Systems

Sl No.	***Lateral load resisting system***	***R***
i) Moment frame systems		
	a) RC buildings with Ordinary moment resisting frame (OMRF) (*see* Note 1a)	3.0
	b) RC buildings with Intermediate moment resisting frame (IMRF) (*see* Note 1b)	4.0
	~~b~~c)RC buildings with special moment resisting frame (SMRF)	5.0
	~~c~~d)Steel buildings with ordinary moment resisting frame (OMRF) (*see* Note 1a)	3.0
	~~d~~e)Steel buildings with special moment resisting frame (SMRF) ~~(*see* Note 1a)~~	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 wall buildings	
	1) Unreinforced masonry (designed as per IS 1905) without horizontal RC seismic bands (see Note 1a)	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[see SP 7 (PART 6) SECTION 4]	3.0
	b) Buildings with ordinary RC structural walls (see Note 1a)	3.0
	c) Buildings with intermediate RC structural walls (ISW) (*see* Note 1b)	3.5
	~~c~~ d) Buildings with ~~ductile~~ RC special structural walls (SSW)	4.0
iv) Dual Systems		
	~~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 special shear~~ structural ~~walls and RC OMRFs (see Note 1)~~	~~4.0~~
	~~d) Buildings with ductile RC special shear~~ structural ~~walls and RC SMRFs (see Note 1)~~	~~5.0~~
	Buildings with moment frame systems (i) with structural walls systems (iii) (*see* Note 3)	$0.5(R^{i}+R^{iii})$
v) Flat Slabs (Two-Way Slabs Without Beams)~~ Structural Wall systems~~ (see Note ~~4~~ 1c)		2.5
	~~RC buildings 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~~	~~3.0~~

	~~walls and the perimeter RC SMRFs (see Note 1)~~	

Notes

1) Limitations on use of lateral force resisting systems-

   a) 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.
   b) This system with relaxed ductile detailing requirements shall not be used in Zone IV and V.
   c) Flat slabs (two-way slab without beams) as a lateral force resisting system are permitted in Seismic Zones II and III for building heights less than 20 m.

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 lateral loads, or

   b) Frames are designed to carry 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.~~

7.2.8 Combination of Structural Systems

Different lateral force resisting systems as given in Table 9 can be incorporated in the same structure subject to following requirements:

a. Combination along different directions

Any combination of building frame systems, shear wall system or dual systems can be used along each of the two orthogonal axis of the structure and respective *R* value shall apply to each system.

b. Vertical Combination

For the vertical combination the following requirements shall apply:

1. In cases where the lower structural system has a smaller *R*, the design of the upper and the lower systems shall be carried out using their respective *R* values. Forces transferred from the upper system to the lower system shall be increased by the ratio of the larger *R* to the smaller *R* values.
2. When the upper system has a smaller *R*, the entire structure (both upper and lower systems) shall be designed using the smaller *R* value of the upper system.

The following two-stage equivalent static analysis procedure may be used for structures having flexible upper portion above a rigid lower portion:

a. The stiffness of the lower portion shall be atleast 10 times the stiffness of the upper portion.
b. The upper flexible portion shall be designed as a separate structure using the appropriate value of *R*.
c. The lower portion shall be designed as a separate structure using an appropriate value of *R*.
d. Reactions of the upper portion shall be increased by the ratio of *R* values of the values of the two portions and the factored reactions shall be applied at the top of the lower portion.

7.3 Design Imposed Loads for Earthquake Force Calculation

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 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.

Diaphragms shall be designed for shear and bending stresses induced due to design lateral forces. Special attention are required at diaphragm discontinuities, such as, reentrant corners, openings, etc, where the design shall ensure that diaphragm and collector or chord forces, if any, are within tensile and shear capacity of the diaphragm.

7.5.1 ~~7.6.4~~Diaphragm Flexibility

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~~ 1.5 times the average displacement of the entire diaphragm (*see* Figure **6**).

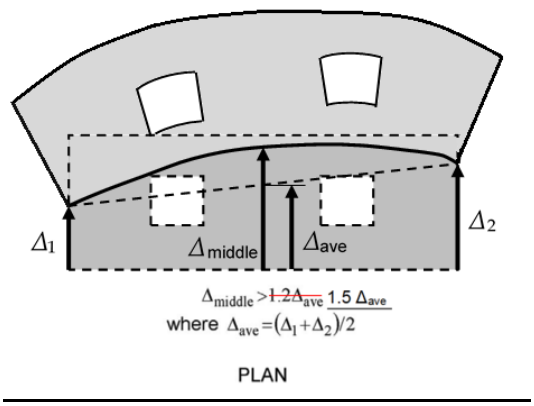


Fig. 6 Definition of Flexible Floor Diaphragm

Usually, reinforced concrete monolithic slab-beam floors or those consisting of prefabricated/precast elements with reasonable reinforced screed concrete as topping, and of plan aspect ratio less than 3, without plan irregularities can be considered to be providing rigid diaphragms action.

7.5.2 Diaphragm Design Forces

Diaphragm design forces for roof and floor diaphragms shall be calculated from structural analysis but should not be less than that given as follows:

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px}$$

where,

F_{px} = design diaphragm force at level x

F_i =seismic design force at level i

w_i = seismic weight at level i

w_{px} = seismic weight at level x

The diaphragm force F_{px} shall not be less than $0.4ZI\ w_{px}$, but not more than $0.8ZI\ w_{px}$.

In case of plan offset of vertical resiting force elements, the lateral force above the diaphragm is to be transferred through the diaphragm to the vertical elements below the diaphragm. This lateral force shall be added to the F_{px} for checking the adequacy of the diaphragm.

7.5 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 15m in Seismic Zone *II*.~~

This method shall be applicable for the following structures.

a) All structures regular or irregular in seismic zone II.
b) Regular structures less than 50 m in height.
c) Regular structure exceeding 50 m but period less than 1.5 s.
d) Irregular structures not more than 5 storeys not exceeding 20 m in height.

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):

$T_a = 0.075\ h^{0.75}$ for RC frame buildings

~~$= 0.08\ h^{0.75}$ for RC-steel Composite MRF buildings~~

$= 0.085\ h^{0.75}$ for steel frame building

where

h= Height (in m) of building (*see* Fig. ~~6~~ 7). 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 and masonry structural walls:

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

$$T_a = \frac{0.00058h}{\sqrt{C_w}}$$

~~where A_w is total effective area (m²) of walls in the first storey of the building given by~~

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

$$C_W = \frac{100}{A_B}\sum_{i=1}^{N_w}\frac{A_{wi}}{\left[1 + 0.83\left(\frac{h}{L_{wi}}\right)^2\right]}$$

where

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

A_B= area of base of structure, in m²;

A_{wi}= effective cross-sectional area of structural wall *i* in ~~first storey~~ of building, in m²;

C_w= effective structural wall area factor;

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

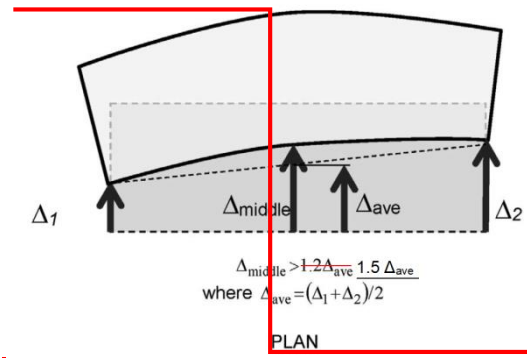
~~d = base dimension of the building at the plinth level along the considered direction of earthquake shaking, in m; and~~

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

~~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 1.5 times the average displacement of the entire diaphragm (*see* Figure 6).~~



~~**Fig. 6** Definition of Flexible Floor Diaphragm~~

~~Usually, reinforced concrete monolithic slab-beam floors or those consisting of prefabricated / precast elements with reasonable reinforced screed concrete (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 diaphragms action.~~

7.6 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 the various lateral load resisting elements) for ~~all~~ the following buildings~~, other than regular buildings lower than 15 m in Seismic Zone *II*.~~ :

a) *Regular buildings* - Those having height greater than 50m and T_a greater than 1.5s in Zones III and IV and V. Modeling as per **7.7.5.4** can be used.

b) *Irregular buildings with plan irregularities of Type (i)a, (ii), (iii), (iv) or (v) of Table 5 or vertical irregularities of Type (iv) or (v) of Table 6* - All buildings higher than 20 m in Zone V, and those having T_a greater than 1.5s in all Zones.

c) *Irregular buildings with plan irregularity of Type (i)b of Table 5 or vertical irregularities of Type (i), (ii) or (iii) of Table 6* – All buildings higher than 20 m in Zones III, IV and V.

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. Buildings with plan irregularities cannot be modeled for dynamic analysis by the method given in **7.7.5.4.**

7.7.3 –

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

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

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

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

7.7.4 – ~~Time~~ Response 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.~~

Response history method shall consist of analysis of linear mathematical model of structures to determine its response to a set of ground motion acceleration histories compatible with the design acceleration spectrum for the site specified by this standard (**6.4.2**) or by a site-specific study (**6.4.7**).

a. The target design acceleration spectrum shall be obtained by multiplying the 5% damped elastic design acceleration spectrum (6.4.2, Fig. 2) with *ZI/2* for the seismic zone the structure is located in. This is not required for the site-specific elastic spectrum.

b. For 3-D analysis, pair of horizontal ground motion time history components shall be selected from not less than 3 recorded events having magnitudes, source mechanism, duration consistent with the magnitude and source characteristics of the design level earthquakes at the site. Synthetic ground motions may be used if appropriate recorded ground motions are not available.

c. The selected motions shall be scaled such that average of SRSS of 5% damped spectrum does not fall below the target elastic design acceleration spectrum in the period range from 0.2*T* to 1.5*T*, where *T* is the fundamental natural period of the structure.

d. For 2-D analysis, major component of horizontal ground motion of selected earthquakes shall be used to perform response history analysis. The selected motions shall be scaled such that average of all 5% damped spectrum does not fall below the target elastic design acceleration spectrum in the period range from 0.2*T* to 1.5*T*, where *T* is the fundamental natural period of the structure.

e. All response parameters shall be multiplied by *1/R*. The maximum response of the three ground motions shall be taken for design and combined with other loads as per this standard. If seven or more pairs of ground motions are use then average results can be used.

f. For nonlinear response history analysis, the structural model should include nonlinear (hysteretic) properties of constituent members, such as yielding, cyclic strength/stiffness degradation, hysteretic pinching etc. The results of the

analysis as response parameter for design shall not be divided by the Response Reduction Factor, *R*.

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 vibration

Undamped free vibration analysis of the entire building shall be performed as per established methods of structural dynamics using the 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 $\cancel{\left[k \in \left(1, N_m\right)\right]}$ that need to be considered as per **7.7.5.2.**

7.7 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 (**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:

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

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

where

e_{si} = Static eccentricity at floor *i*

= the 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~~ Dynamic Analysis Method.

7.9 RC Frame Buildings with Unreinforced Masonry Infill Walls

7.9.2.1 –

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

$$E_m = 550\, f_m$$

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

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

$$\underline{f_m = 0.63 f_b^{0.49} f_{mo}^{0.32}}$$

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 modelled 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~~**8**) shall be taken as

$$\underline{w_{ds} = 0.25 L_{ds}}$$

$$\cancel{w_{ds} = 0.17 \alpha_h^{-0.4} L_{ds}}$$

where

L_{ds} is length of diagonal strut

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

~~Where E_m and E_f are the moduli of elasticity of the materials of the URM infill and RC, 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~~

c) Thickness of the equivalent diagonal strut shall be taken as thickness t of original URM infill wall, provided h/t < ~~12~~<u>30</u> and l/t < ~~12~~<u>30</u>, 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.

d) <u>The advantages of strength contributed by the infill shall not to be considered when the height of the building is more than 12m.</u>

e) <u>All the RC frames shall be designed to support the vertical gravity loads,</u>

including the weight of masonry infill walls, without any assistance from the masonry infill walls. Also, the frame acting alone shall be capable of resisting at least 50 percent of the design seismic forces.

7.10 ~~RC~~ Frame Buildings with Open Storeys

7.10.1 –

~~RC moment 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 well as irregularities of out-of-plane offsets, in-plane discontinuity and possibly torsional irregularity due to unequal distribution of the infills as per Tables 5 and 6. In such buildings, suitable measures shall be adopted, which should increase both stiffness and strength of vertical members (columns) of the open story to the level required by 2.5 times of the seismic load combinations. ~~in the open storey and 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 in strength 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~~

~~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;~~

~~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}, in percent, 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 the 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

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. This storey drift shall include both translational and torsional deflections. Building with load bearing masonry walls as lateral load resisting element shall be subjected to lower limit of 0.0015 times the storey height.

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 during earthquake shaking. The deflection of the building will be more than elastic drift calculated using 7.11.1 and can be estimated as $R\Delta$. Under this drift ($R\Delta$), induced bending moments and shear forces can exceed the elastic capacity and they should be provided adequate ductility as per IS 13920.~~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

In order to avoid damage due to pounding between two ~~Two~~ adjacent buildings, or two adjacent units of the same building ~~with separation joint between them~~, shall be separated by a distance equal to the square root of sum of squares of ~~the amount *R* times the sum of~~ inelastic storey displacements, R_1 times Δ_1 and R_2 times Δ_2 calculated as per **7.11.1.** That is,

$$\Delta_{sep} = \sqrt{\left(R_1\Delta_1\right)^2 + \left(R_2\Delta_2\right)^2}$$

where R_1 and Δ_1 correspond to building 1, and R_2 and Δ_2 to building 2.

~~of the two buildings or two units of the same building, to avoid pounding as the two buildings or two units of the same buildings oscillate towards each other~~.

~~When the floor levels of the adjacent units of a building or buildings are at the same level, the separation distance shall be calculated as $(R_1\Delta_1 + R_1\Delta_1)$ / 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 R.C.C. footings without tie beams, or unreinforced strip foundations, shall not be adopted in buildings rested on soft soils (with corrected ~~*N*~~ $(N_1')_{60} < 10$) in any seismic zone. The 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*.

The seismic forces (overturning moments and shear forces) for foundation design of vertical elements of seismic force resisting system shall not be less than 125% of the design resistance of structural elements supported by the foundation.

In buildings located in seismic Zones *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. 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 the larger of the column or pile cap loads. Here A_h is as per 6.4.2.

Piles shall be designed and constructed to withstand maximum curvature imposed (structural response) by earthquake ground shaking. Design of anchorage of piles into 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 tower, tanks, parapets, smoke stacks/chimneys) and other vertical cantilever projections attached to buildings and projecting above the roof, but not a part of structural system of the building, shall be designed and checked for stability for 2.5 *ZI* times the seismic weight of the structure ~~five times the design horizontal seismic coefficient Ah specified in 6.4.2 for that building~~. In the analysis of the building, the weight of these projecting elements shall be lumped with the roof weight.

7.12.2.2 – Horizontal Projections

All horizontal projections of building (like cantilever structural members at the porch level or higher) or attached to buildings (like brackets, cornices and balconies) shall be designed for 0.9 *ZI* times the seismic weight of the structure ~~five times the design vertical coefficient A_v specified in 6.4.6 for that building.~~

7.12.5– Temporary Structures

Temporary structures such as scaffolding, shelters, tents, temporary excavations and other facilities during the construction of structures are meant for a limited-time use. For such structures, the A_h in section 6.4.2 shall be reduced by 50%.

7.12.6– Parking Structure

Parking structures shall be designed for the same force as building structures. However, a larger drift equal to 1.5 times the storey drift limit of section 7.11.1 can be permitted.

7.13 Nonstructural Elements

7.13.1–General

7.13.1.1-

This section establishes minimum design criteria for the nonstructural components of architectural, mechanical, and electrical systems permanently installed in buildings, including supporting structures and attachments.

7.13.1.2–

This section is not applicable where a nonstructural component directly modifies the strength or stiffness of the building structural elements, or its mass affects the building loads. In such a case, its characteristics should be considered in the structural analysis of the building.

7.13.1.3–

For nonstructural elements of great importance or of a particular dangerous nature, the seismic analysis should be based on the floor response spectra derived from the response of the main structural system. Specialist literature may be referred to for the methods of determining floor response spectrum for various floors/elevations.

7.13.1.4–

Particular care should be taken to identify masonry infill that could reduce the effective length of adjoining columns.

7.13.1.5–

In general, if the component weight exceeds 20% of the total dead weight of the floor, or exceeds 10% of the total weight of the structure provisions in this section should not be used.

7.13.2–

Depending on response sensitivity, nonstructural elements can be classified as deformation sensitive, acceleration sensitive, or both deformation and acceleration sensitive. Table 11 classifies nonstructural elements according to their response sensitivity.

7.13.2.1–

Acceleration sensitive nonstructural elements should be designed according to the force

provisions contained in clause 7.13.3.

7.13.2.2–

Deformation sensitive nonstructural elements should be designed according to the provisions contained in clause 7.13.4.

7.13.2.3–

Some components may be both acceleration and deformation sensitive, but generally one or the other of these characteristics is dominant (Table 11). They must be analyzed for both forms of response, that is, as per provisions ~~1.3 and 1.4~~.of 7.13.3 and 7.13.4

7.13.3– Design Seismic Force

7.13.3.1–

Design seismic force F_p on a nonstructural element shall be calculated as

$$F_p = \frac{Z}{2}\left(1+\frac{x}{h}\right)\frac{a_p}{R_p} I_p W_p \geq 0.10 W_p$$

where

Z= Zone factor given in Table ~~2~~3,

x = Height of point of attachment of the nonstructural element above top of the foundation of the building,

h = Height of the building,

a_p=Component amplification factor given in Table 12 and 13,

R_p = Component response modification factor given in Table 12 and 13,

I_p = Importance factor of the nonstructural element given in Table 14, and

W_p = Weight of the nonstructural element.

Table 11: Response Sensitivity of Nonstructural Components (clause 7.13.2)

Component		Sensitivity		Component		Sensitivity	
		Acc	**Def**			**Acc**	**Def**
A. Architectural				**B. Mechanical Component**			
1.	**Exterior Skin**			**1.**	**Mechanical Equipment**		
	Adhered Veneer	S	P		Boilers and Furnaces	P	
	Anchored Veneer	S	P		General Manufacturing and Process Machinery	P	
	Glass Blocks	S	P		HVAC Equipment, Vibration Isolated	P	
	Prefabricated Panels	S	P		HVAC Equipment. Non-vibration Isolated	P	
	Glazing Systems	S	P				
2.	**Partitions**				HVAC Equipment, Mounted In-line with Ductwork	P	
	Heavy	S	P				
	Light	S	P	**2.**	**Storage Vessels and Water Heaters**		
3.	**Interior Veneers**				Structurally Supported Vessels	P	
	Stone, Including Marble	S	P		Flat Bottom Vessels	P	
	Ceramic Tile	S	P				
4.	**Ceilings**			**3.**	**Pressure Piping**	P	S
	a. Directly Applied to Structure	P		**4.**	**Fire Suppression Piping**	P	S
	b. Dropped, Furred, Gypsum Board	P		**5.**	**Fluid Piping, not Fire Suppression**		
	c. Suspended Lath and Plaster	S	P		Hazardous Materials	P	S
	d. Suspended Integrated Ceiling	S	P		Non-hazardous Materials	P	S
5.	**Parapets and Appendages**	P		**6.**	**Ductwork**	P	S
6.	**Canopies and Marquees**	P					
7.	**Chimneys and Stacks**	P					

8.	**Stairs**	P	S		
Acc=Acceleration-Sensitive				P=Primary Response	
Def=Deformation Sensitive				S=Secondary Response	

Table 12: Coefficients for Architectural Components (clause 7.13.3)

Architectural Component or Element	a_p [a]	R_p
Interior Nonstructural Walls and Partitions		
Plain (unreinforced) masonry walls	1.0	1.5
All other walls and partitions	1.0	2.5
Cantilever Elements (Unbraced or braced to structural frame below its center of mass)	2.5	2.5
Parapets and cantilever interior nonstructural walls	2.5	2.5
Chimneys and stacks where laterally supported by structures.		
Cantilever elements (Braced to structural frame above its center of mass)		
Parapets	1.0	2.5
Chimneys and stacks	1.0	2.5
Exterior Nonstructural Walls	1.0	2.5
Exterior Nonstructural Wall Elements and Connections		
Wall Element	1.0	2.5
Body of wall panel connection	1.0	2.5
Fasteners of the connecting system	1.25	1.0
Veneer		
High deformability elements and attachments	1.0	2.5
Low deformability and attachments	1.0	1.5
Penthouses (except when framed by and extension of the building frame)	2.5	3.5
Ceilings		
All	1.0	2.5
Cabinets		
Storage cabinets and laboratory equipment	1.0	2.5
Access floors		
Special access floors	1.0	2.5
All other	1.0	1.5
Appendages and Ornamentations	2.5	2.5
Signs and Billboards	2.5	2.5
Other Rigid Components		
High deformability elements and attachments	1.0	3.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
Other flexible Components		
High deformability elements and attachments	2.5	3.5
Limited deformability elements and attachments	2.5	2.5
Low deformability elements and attachments	2.5	1.5

[a] A lower value for a_p is permitted provided a detailed dynamic analysis is performed which justifies a lower value. The value for a_p shall not be less than 1.0. The value of a_p =1.0 is for equipment generally regarded as rigid and rigidly attached. The value of a_p =2.5 is for flexible components and flexibly attached components.

Table 13: Coefficients for Mechanical and Electrical Components (clause 7.13.3)

Mechanical and Electrical Component or Element[b]	a_p[a]	R_p
General Mechanical		
Boilers and Furnaces	1.0	2.5
Pressure vessels on skirts and free-standing	2.5	2.5
Stacks	2.5	2.5
Cantilevered chimneys	2.5	2.5
Others	1.0	2.5
Manufacturing and Process Machinery		
General	1.0	2.5
Conveyors (non-personnel)	2.5	2.5
Piping Systems		
High deformability elements and attachments	1.0	2.5
Limited deformability elements and attachments	1.0	2.5
Low deformability elements and attachments	1.0	1.5
HVAC System Equipment		
Vibration isolated	2.5	2.5
Non-vibration isolated	1.0	2.5
Mounted in-line with ductwork	1.0	2.5
Other	1.0	2.5
Elevator Components	1.0	2.5
Escalator Components	1.0	2.5
Trussed Towers (free-standing or guyed)	2.5	2.5
General Electrical		
Distributed systems (bus ducts, conduit, cable tray)	2.5	5.0
Equipment	1.0	1.5
Lighting Fixtures	1.0	1.5

[a] A lower value for a_p is permitted provided a detailed dynamic analysis is performed which justifies a lower value. The value for a_p shall not be less than 1.0. The value of a_p =1.0 is for equipment generally regarded as rigid and rigidly attached. The value of a_p =2.5 is for flexible components or flexibly attached components.

Table 14: Importance Factor (I_p) of Nonstructural Elements (Clause 7.13.3)

Description of nonstructural element	I_p
Component containing hazardous contents	1.5
Life safety component required to function after an earthquake (e.g., fire protection sprinklers system)	1.5
Storage racks in structures open to the public	1.5
All other components	1.0

7.13.3.2–

For vertical nonstructural elements F_p will be the horizontal force, and for horizontal nonstructural elements F_p will be the vertical force.

7.13.3.3–

For a component mounted on a vibration isolation system, the design force shall be taken as $2F_p$.

7.13.3.4– Connections

Connections and attachments or anchorage of the nonstructural element should be designed for twice the design seismic force required for that nonstructural element. Connection and attachment shall be bolted, welded, or otherwise positively fastened without consideration of frictional resistance produced by the effect of gravity. Connections to ornaments, veneers, appendages, and exterior panels including anchor bolts shall be corrosion resisting, ductile, and have adequate anchorages.

7.13.4– Seismic Relative Displacement

Seismic relative displacement (D_p), that a nonstructural element must be designed to accommodate shall be determined as per clause 7.13.4.1, 7.13.4.2 and 7.13.4.3.

7.13.4.1–

For two connection points on the same structure A, one at a height h_x, and other at a height h_y, seismic relative displacement D_p shall be determined as

$$D_p = \delta_{xA} - \delta_{yA}$$

D_p is not required to be taken as greater than

$$R(h_x - h_y)\frac{\Delta_{aA}}{h_{sx}}$$

where,

$\delta_{xA} =$ Deflection at building level x of structure A due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as

per Table 9,

δ_{yA} = Deflection at building level *y* of structure A due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as per Table 9,

h_x = Height of level *x* to which upper connection point is attached,

h_y = Height of level *y* to which lower connection point is attached,

Δ_{aA} = Allowable storey drift for structure A calculated as per 7.11.1, and

h_{sx} = Storey height below level x.

7.13.4.2–

For two connection points on separate structures A and B, or separate structural systems, one at height h_x and the other at a height h_y, D_p shall be determined as

$$D_p = |\delta_{xA}| + |\delta_{yB}|$$

D_p is not required to be taken as greater than

$$R\left(h_x \frac{\Delta_{aA}}{h_{sx}} + h_y \frac{\Delta_{aB}}{h_{sx}} \right)$$

where,

δ_{yB} = Deflection at building level *y* of structure B due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as per Table 9,

Δ_{aB} = Allowable storey drift for structure B calculated as per 7.11.1.

7.13.4.3-

The effect of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

Annex F

Corrections in Standard Penetration Test (SPT) Values

$$(N_1)_{60} = N \times C_N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4$$

where,

N = measured (raw) SPT blow count

N_{60} = Normalized SPT blow count for 60% energy efficiency

$(N_1)_{60}$ = SPT blow count normalized for vertical effective stress of 1 atmosphere (i.e., about 100 kPa) and delivery of 60 % of theoretical hammer energy

η_1, η_2, η_3 and η_4 = correction factors (see Table F1)

C_N = correction for effective overburden pressure = $\sqrt{P_a/\sigma'_{v0}}$ subjected to $C_N \leq 1.7$ (avoiding unnecessary large values near ground surface), where, P_a = Atmospheric Pressure and σ'_{v0} = the effective vertical stress at the time of standard penetration testing.

For soils containing significant amount of fine sand and silt found below water table with raw SPT blow count greater than 15, in applications in a drained problem, a correction given below is applied to eliminate the effects of dilatancy on measured (raw) SPT blow count:

$(N_1')_{60}$ = 15+ 1/2 $((N_1)_{60}-15)$

$(N_1')_{60}$ = Dilatancy corrected SPT blow count normalized for vertical effective stress or 1 atmosphere and delivery of 60 % of theoretical hammer energy

Note that such a correction is not applicable in liquefaction assessments.

Table F1 - Correction factors for Standard Penetration Test (SPT)

Hammer energy correction, η_1	
System	η_1
Donut hammer and rope and pulley	0.75
Safety hammer and rope and pulley	1.00
Automatic (trip) hammer	1.33
Rod Length correction, η_2	
Rod length, m	η_2
> 10	1.00
6 to 10	0.95
4 to 6	0.85
0 to 4	0.75
Liner correction, η_3	
Presence or absence of liner, type of soil	η_3
Without liner , all soils	1.00
With liner, dense sand and clay	0.80
With liner, loose sand	0.90
Borehole diameter correction, η_4	
Borehole diameter, mm	η_4
60 to120	1.00
150	1.05
200	1.15
All diameters. if borehole is supported mechanically (with full casing or with hollow-stem augers)	1.00

Annex G

Simplified Procedure for Evaluation of Liquefaction Potential

Due to the difficulties in obtaining and testing undisturbed representative samples from most potentially liquefiable sites, in-situ testing is the approach preferred by most engineers 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[1]. The procedure may be used with either SPT blow count, CPT tip resistance or shear wave velocity measured within the deposit as discussed below:

Step 1: The subsurface data used to assess liquefaction susceptibility should include the location of the water table at the time of subsurface investigation (i.e., at the time of SPT, CPT or shear wave velocity measurement), either SPT blow count (*N*) (or tip resistance of a standard CPT cone q_c or the shear wave velocity), mean grain size D_{50}, unit weight, and fines content of the soil (percent by weight passing the IS Standard Sieve No. 75*μ*).

Step 2: Evaluate the total vertical stress, σ_{vd}, and the effective vertical stress, σ'_{vd} for all ~~potentially liquefiable~~ soil layers within the deposit for the design condition. The design water table for evaluation of liquefaction potential shall be assumed to be the highest water table that can exist at the site over several weeks in a year. It should be noted that the elevation of water table considered in design may differ from that at the time of subsurface investigation. Post construction ground level could also differ from that during the subsurface investigation due to site grading and permanent fill placement.

Step 3: The following equation can be used to evaluate the *stress reduction factor* r_d:

$$r_d = 1 - 0.00765z \qquad 0 < z \leq 9.15 \text{ m}$$

$$r_d = 1.174 - 0.0267z \qquad 9.15 \text{ m} < z \leq 23.0 \text{ m}$$

where z is the depth below the ground surface in meters.

Step 4: Calculate the *critical stress ratio, CSR* induced by the design earthquake for a soil layer at depth *z* as;

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

where ~~σ_{vd} and σ'_{vd} are the total and effective vertical stresses, respectively, at depth z,~~ $a_{\max}$ is the peak ground acceleration at the ground surface, and g is the acceleration due to gravity and r_d is the 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~~ magnitudes other than 7.5, vertical effective stresses (including possible stress increase due to the presence of existing or proposed structures) exceeding 100 kPa ~~high overburden stress level and high initial static shear stress~~ and static shear stress due to ground slope and/or presence of structures using:

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

[1] Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Chtristian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., Stokoe II, K.H. 2001. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. J. of Geotech. and Geoenv. Engrg., ASCE. 127(10): 817-833.

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 given by the following equation:

$$\cancel{MSF = 10^{2.24} / M_w^{2.56}}$$

$$MSF = (M_w/7.5)^{-3.3}$$

This factor is required when the magnitude is different than 7.5. If earthquake magnitude M_w for the site is not available, it can be taken according to the table below.

Table G1: Earthquake magnitude M_w

Earthquake Zone	M_w
Zone – II	6.0
Zone – III	6.5
Zone – IV	7.0
Zone – V	7.5

The correction for high overburden stresses K_σ is required ~~when overburden pressure is high~~ for ~~(~~depth > 15 m~~)~~ and can be found using the following equation.

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

Where σ'_{vd} effective overburden pressure for water table at its design elevation 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 stress 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 to be 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 for a hammer with an efficiency of 60 percent. Specifications of the "standardized" equipment corresponding to an efficiency of are given in Table 11. If equipment used is of non-standard type, N_{60} shall be obtained using measured value (N):~~

$$\cancel{N_{60} = NC_{60}}$$

~~where~~

$$\cancel{C_{60} = C_{HT} C_{HW} C_{SS} C_{RL} C_{BD}}$$

~~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 deilevered 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:~~

$$\cancel{(N_1)_{60} = C_N N_{60}}$$

~~where~~

$$\cancel{(C_N) = \sqrt{\frac{P_a}{\sigma'_{vo}}} \leq 1.7}$$

The cyclic resistance ratio $CRR_{7.5}$ is estimated from Fig.~~8~~G1, using $(N_1)_{60}$ value.

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

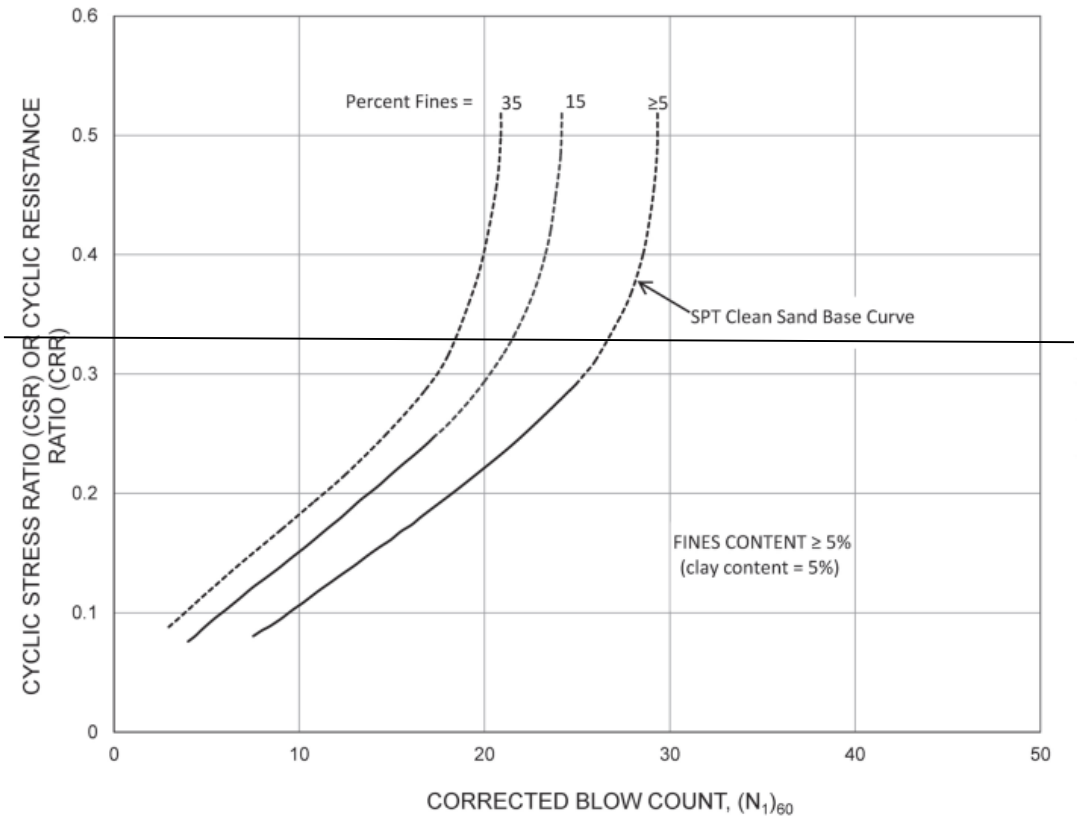
$$(N_1)_{60CS} = \alpha + \beta (N_1)_{60}$$

where

$$\alpha = \begin{cases} 0 & \beta = 1 & \text{for } FC \leq 5 \text{ percent} \\ e^{\left[1.76 - \left(\frac{190}{FC^2}\right)\right]} & \beta = 0.99 + \frac{FC^{1.5}}{1000} & \text{for 5 percent} < FC < 35 \text{ percent} \\ 5 & \beta = 1.2 & \text{for F}C \geq 35 \text{ percent} \end{cases}$$

Again, Fig. ~~8~~G1 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~~G1:

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



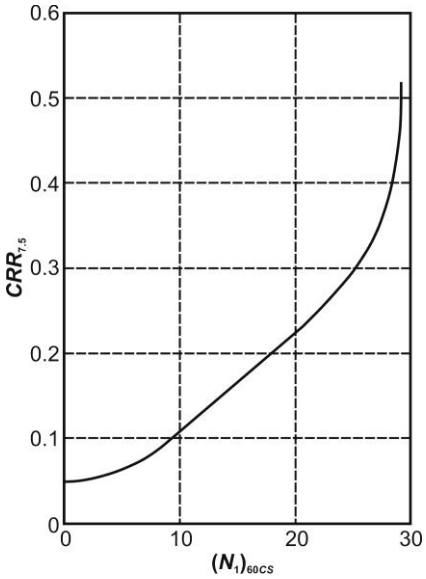


Fig.8G1: Relation Between $CRR_{7.5}$ and $(N_1)_{60CS}$ ~~for sand for M_w 7.5 EARTHQUAKES~~

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

$$C_Q = \left(\frac{P_a}{\sigma'_{vd}}\right)^n \text{ subject to } C_Q \le 1.7$$

where n is 0.5 and 1 for sand and clay, respectively

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

$$\left(q_{C1N}\right)_{CS} = k_c q_{C1N}$$

where

k_c = Correction factor to account for grain characteristics

$$= \begin{cases} 1.0 & \left(\text{for } I_c \le 1.64\right) \\ -0.403 I_c^{\,4} + 5.581 I_c^{\,3} - 21.63 I_c^{\,2} + 33.75 I_c - 17.88 & \left(\text{for } I_c > 1.64\right) \end{cases}$$

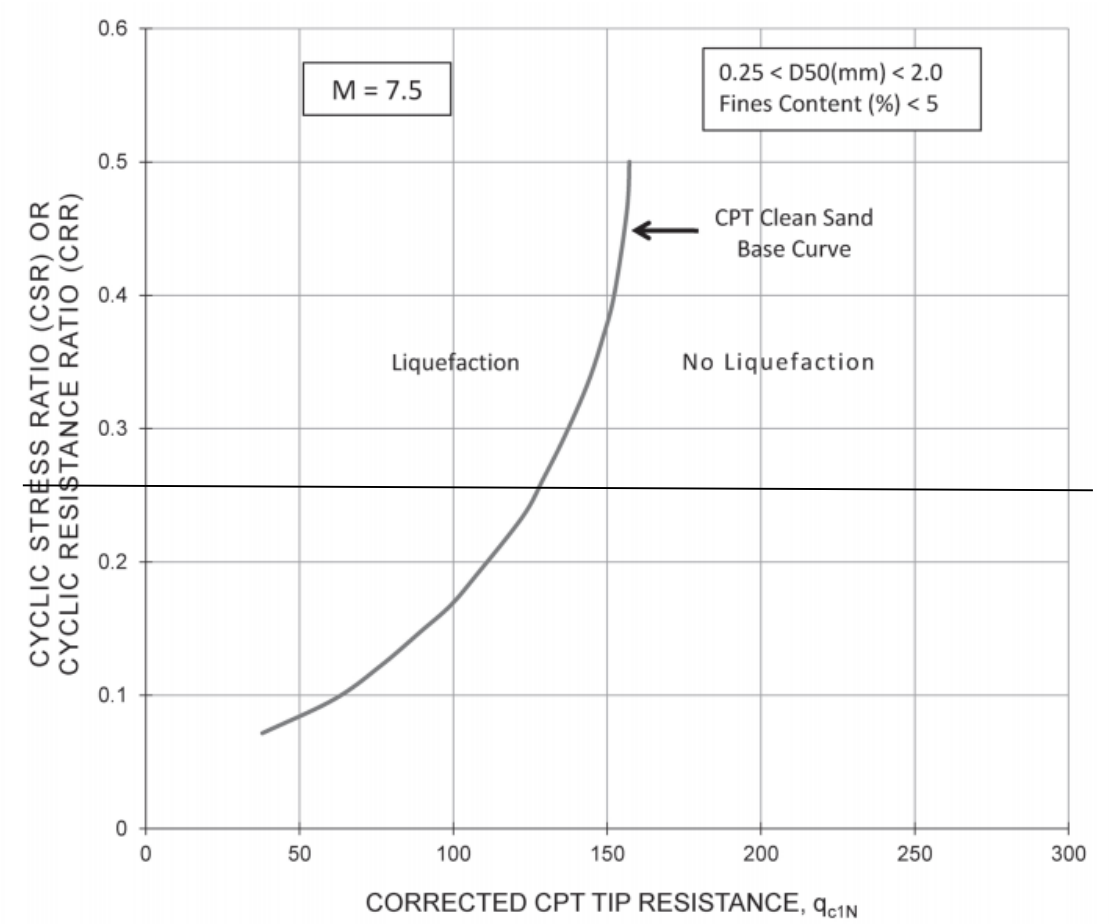
$$I_c = \sqrt{(3.47 - logQ)^2 + (1.22 + logF)^2}$$

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

$F = 100\left(\frac{f_s}{q_c - \sigma_{vd}}\right)$ percent, and where f_s is the measured sleeve friction.

Using $\left(q_{C1N}\right)_{CS}$ find the value of $CRR_{7.5}$ using Fig. ~~9~~G2. Alternatively, the $CRR_{7.5}$ can be found from the following equations:

$$CRR_{7.5} = \begin{cases} 0.833\left(\dfrac{\left(q_{C1N}\right)_{CS}}{1000}\right) + 0.05, & 0 < \left(q_{C1N}\right)_{CS} < 50 \\ 93\left(\dfrac{\left(q_{C1N}\right)_{CS}}{1000}\right)^3 + 0.08, & 50 \le \left(q_{C1N}\right)_{CS} < 160 \end{cases}$$



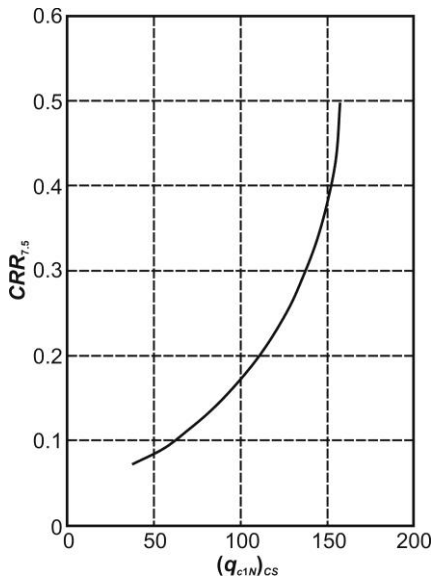


Fig.9 G2 : Relation between $CRR_{7.5}$ and $(q_{C1N})_{cs}$ ~~for M_w 7.5 EARTHQUAKES~~

6(c)*Using shear wave velocity:*

Apply correction for overburden stress to shear wave velocity V_s for ~~clean~~ sand ~~using~~ to obtain

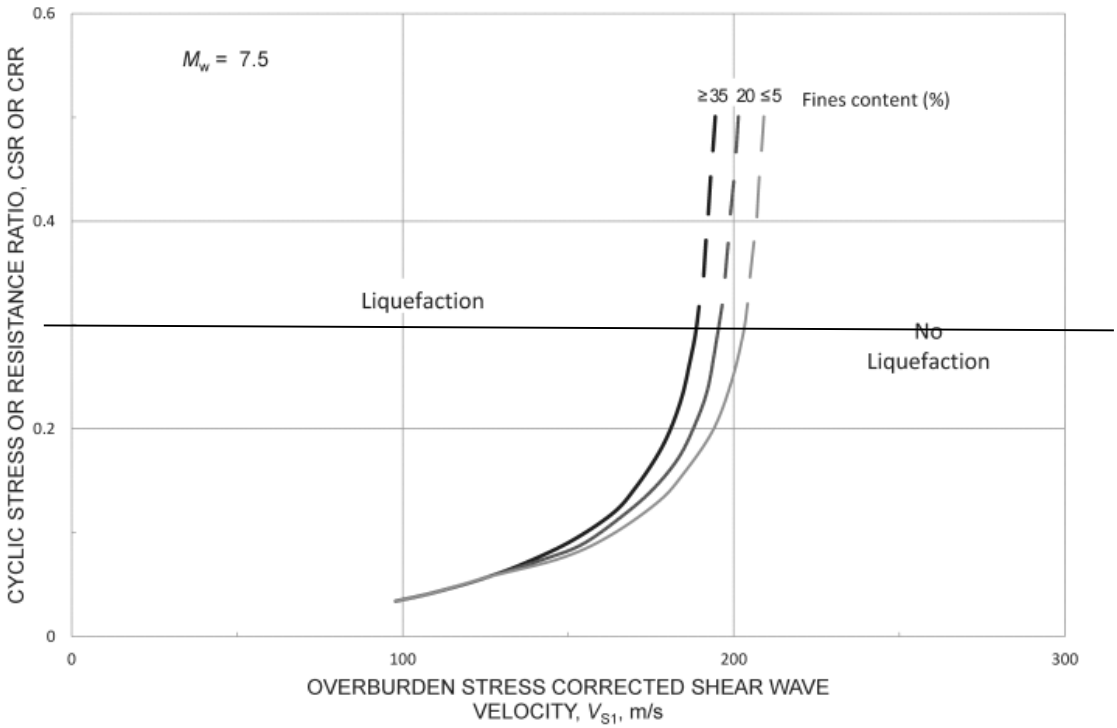
$$V_{s1} = (P_a/\sigma'_{vo})^{0.25} V_s \quad \text{(subject to } V_{s1} \leq 1.3V_s\text{)}$$

where V_{s1} is overburden stress corrected shear wave velocity and σ'_{vo} is the effective vertical

stress at the time of shear wave velocity measurement. Using V_{s1} find the value of $CRR_{7.5}$ using Fig. ~~10~~G3. Alternatively, the $CRR_{7.5}$ can be found using the 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 value of V_{s1} for liquefaction occurrence, a and b are curve fitting parameters. The values of a and b in Fig.~~10~~ G3 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.



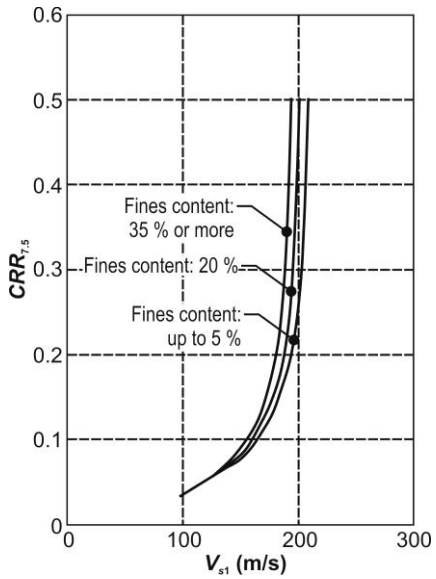


Fig. ~~10~~G3: Relation between $CRR_{7.5}$ and V_{S1} ~~for M_w 7.5 EARTHQUAKES~~

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 $FS \geq 1.2$~~.

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

~~**Table 11: Recommended "Standardized' SPT Equipment (*see* IS 2131)**~~

~~[*Clause* F-1, *Step*: 6(a)]~~

~~**Sl No. (1)**~~	~~**Element**~~	~~**Standard Specification**~~
~~i)~~	~~Sampler~~	~~Standard split-spoon sampler with: (a) Outside diameter, O.D. = 51 mm, and Inside Diameter, I.D. = 35 mm~~ ~~(constant – i.e., 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; (b) drop = 762 mm (delivers 60of theoretical free fall energy)~~
~~iv)~~	~~Rope~~	~~Two wraps of rope around the pulley~~
~~v)~~	~~Borehole~~	~~100 to 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 to 460 mm of penetration into the ground~~

~~Table 12: Correction Factors for Non-Standard SPT Procedures and Equipment~~

~~[*Clause* F-1, *Step*: 6(a)]~~

~~Correction for~~	~~Correction Factor~~
~~Nonstandard Hammer Type~~ ~~(*DH*= doughnut hammer; *ER* = energy ratio)~~	~~C_{HT} =0.75 for *DH* with rope and pulley~~ ~~C_{HT} =1.33 for *DH* with trip/auto and *ER* = 80~~
~~Nonstandard Hammer Weight or Height of fall~~ ~~(*H* = height of fall in mm; *W* = hammer weight in kg)~~	$C_{HW} = \frac{H.W}{635 \times 762}$
~~Nonstandard Sampler Setup (standard samples with room for liners, but used without liners~~	~~C_{SS} =1.10 for loose sand~~ ~~C_{SS} =1.20 for dense sand~~
~~Nonstandard Sampler Setup (standard samples with room for liners, but liners are used)~~	~~C_{SS} =0.90 for loose sand~~ ~~C_{SS} =0.80 for dense sand~~
~~Short Rod Length~~	~~C_{RL} =0.75 for rod length 0-3 m~~
~~Nonstandard Borehole Diameter~~	~~C_{BD} =1.05 for 150 mm borehole diameter~~ ~~C_{BD} =1.15 for 200 mm borehole diameter~~

~~Notes : *N* = Uncorrected SPT blow count.~~

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

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

~~C_N = Correction factor for overburden pressure~~

~~$(N_1)_{60} = C_N N_{60} = C_N C_{60} N$~~