
By

Durgesh C Rai¹
Sudhir K Jain²

with assistance from

Parul Srivastava¹
Ankul Kumar¹

¹) Indian Institute of Technology Kanpur
²) Indian Institute of Technology Gandhinagar

November 2019
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.

Comments and feedback may please be forwarded to:
Prof. Sudhir K Jain
email: skjain@iitg.ac.in, skjain.iitgn@gmail.com
Acknowledgements

- We gratefully acknowledge the World Bank for sponsoring this project on Improving Seismic Resilience of Built Environment in India. Special thanks are due to Keiko Sakoda, Thomas Moullier, Anup Karanth and Deepak Singh at the World Bank.

- We are especially grateful to our colleagues, Prof. Mahesh Tandon (Tandon Consulting, New Delhi), Amit Prashant (IIT Gandhinagar) and Debasish Roy (IIT Kharagpur) for discussion and drafting of provisions related to liquefaction and other geotechnical aspects.

- The authors are also grateful to the World Bank’s reviewer, Prof. Andreas J. Kappos, United Kingdom for providing comprehensive set of recommendations that contributed to enhancing the quality of this document.

- The authors acknowledge input and feedback received from internal reviewers Nilesh Shah, Hemal Mistry, and Narayan Kochak and other Indian colleagues, including Anal Shah, Alok Bhowmick, Vipul Ahuja, A K Jain, etc.

- We sincerely appreciate the participation of following colleagues in the roundtable workshop held at IIT Gandhinagar on April 8, 2019 to review and discuss seismic design codes, IS 1893 (Part 1) and IS 13920:

  - Ahuja, Vipul Consulting Engineer, New Delhi
  - Bhowmick, Alok Consulting Engr., New Delhi
  - Brzev, Svetlana IIT Gandhinagar
  - Goswami, Rupen IIT Madras
  - Ingle, R. K. VNIT Nagpur
  - Jain, Sudhir K. IIT Gandhinagar
  - Jaiswal, O. R. VNIT Nagpur
  - Karanth, Anup The World Bank, Delhi
  - Kaushik, Hemant B. IIT Guwahati
  - Khandelwal, Praveen NTPC Delhi
  - Kochak, Narayan Consulting Engineer, Pune
  - Kumar, Arun S Bureau of Indian Standards, Delhi
  - Kumar, Hemant Consulting Engineer, Delhi
  - Kumar, Manish IIT Gandhinagar
  - Murty, C. V. R. IIT Madras
  - Mistry, Hemal Consulting Engineer, Surat
  - Pathak, Jayanta Assam Engineering College, Guwahati
  - Perez-Gavilan, Juan Jose Universidad Nacional Autónoma de México
  - Rai, Durgesh C. IIT Kanpur
  - Shah, Anal Consulting Engineer, Ahmedabad
  - Shah, Bhavin Consulting Engineer, Ahmedabad
  - Shah, Nilesh Consulting Engineer, Surat
  - Sharma, Rajeev Consulting Engineer, Delhi
  - Sheth, Alpa Consulting Engineer, Mumbai
  - Singh, Deepak The World Bank, Delhi
  - Singh, Yogendra IIT Roorkee
  - Singhal, Vaibhav IIT Patna
  - Tandon, Mahesh Consulting Engineer, Delhi

- Another discussion meeting on geotechnical aspects of seismic design and evaluation of liquefaction potential was held on April 17, 2019 at IIT Gandhinagar which was attended by the following colleagues:

  - Jain, Sudhir K. IIT Gandhinagar
  - Kumar, Manish IIT Gandhinagar
  - Prashant, Amit IIT Gandhinagar
  - Rao, G. V. former Professor, IIT Delhi
  - Roy, Debasish IIT Kharagpur
  - Tandon, Mahesh Consulting Engineer, Delhi
  - Venkataraman, M. Vice President - Indian Chapter of International Geosynthetics Society

- 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.
Table of Contents

FOREWORD ............................................................................................................................................. 6

1 – SCOPE ................................................................................................................................................ 12
  1.1 .............................................................................................................................................................. 12
  1.2 .............................................................................................................................................................. 12
  1.3 .............................................................................................................................................................. 13

2 REFERENCES ......................................................................................................................................... 14

3 – TERMINOLOGY .................................................................................................................................. 17
  3.1 CLOUSSY-SPACED MODES ............................................................................................................... 17
  3.2 CRITICAL DAMPING ........................................................................................................................ 17
  3.3 DAMPING ............................................................................................................................................. 17
  3.4 DESIGN ACCELERATION SPECTRUM ............................................................................................. 17
  3.5 DESIGN BASIS EARTHQUAKE ......................................................................................................... 18
  3.6 DESIGN HORIZONTAL ACCELERATION COEFFICIENT ($A_n$) .................................................. 18
  3.7 DESIGN HORIZONTAL FORCE ...................................................................................................... 18
  3.8 DUCTILITY .......................................................................................................................................... 18
  3.9 EPICENTRE ......................................................................................................................................... 18
  3.10 FLOOR RESPONSE SPECTRUM ..................................................................................................... 19
  3.11 GEOTECHNICAL FIELD TEST PARAMETERS .............................................................................. 19
  3.12 IMPORTANCE FACTOR (I) .............................................................................................................. 19
  3.13 INTENSITY OF EARTHQUAKE ....................................................................................................... 19
  3.14 LIQUEFACTION .............................................................................................................................. 20
  3.15 LITHOLOGICAL FEATURES .......................................................................................................... 20
  3.16 MAXIMUM CONSIDERED EARTHQUAKE (MCE) ........................................................................ 20
  3.17 MODAL MASS ($M_k$) IN MODE (K) OF A STRUCTURE ................................................................... 22
  3.18 MODAL PARTICIPATION FACTOR ($P_k$) IN MODE (K) OF A STRUCTURE .................................. 22
  3.19 MODES OF OSCILLATION ................................................................................................................ 22
  3.20 MODE SHAPE COEFFICIENT ($\Phi_{mk}$) .......................................................................................... 23
  3.21 NATURAL PERIOD ($T_k$) IN MODE (K) OF OSCILLATION ............................................................. 23
  3.22 NORMAL MODE OF OSCILLATION ............................................................................................... 23
  3.23 PEAK GROUND ACCELERATION ................................................................................................... 24
  3.24 RESPONSE REDUCTION FACTOR (R) ............................................................................................. 25
  3.25 RESPONSE SPECTRUM ................................................................................................................... 25
  3.26 RESPONSE ACCELERATION COEFFICIENT OF A STRUCTURE ($S_{a}/G$) ................................. 27
  3.27 SEISMIC MASS OF A FLOOR ............................................................................................................ 27
  3.28 SEISMIC MASS OF A STRUCTURE ................................................................................................ 28
  3.29 SEISMIC WEIGHT OF A FLOOR ($W$) ............................................................................................ 28
  3.30 SEISMIC WEIGHT OF A STRUCTURE ($W$) .................................................................................... 28
  3.31 SEISMIC ZONE FACTOR ($Z$) ......................................................................................................... 28
  3.32 RESPONSE HISTORY ANALYSIS ............................................................................................... 28

4 SPECIAL TERMINOLOGY FOR BUILDINGS ......................................................................................... 29
  4.1 .............................................................................................................................................................. 29
  4.2 BASE .................................................................................................................................................. 29
  4.3 BASE DIMENSION (D) ..................................................................................................................... 31
  4.4 CENTRE OF MASS (CM) ................................................................................................................ 32
  4.5 CENTRE OF RESISTANCE (CR) ....................................................................................................... 32
  4.6 ECCENTRICITY .................................................................................................................................. 33
  4.7 DESIGN SEISMIC BASE SHEAR ($V_{bh}$) ...................................................................................... 33
  4.8 DIAPHRAGM ..................................................................................................................................... 33
  4.9 HEIGHT OF FLOOR ($H_i$) ................................................................................................................. 34
  4.10 HEIGHT OF BUILDING ($H$) .............................................................................................................. 34
  4.11 HORIZONTAL BRACING SYSTEM .................................................................................................. 34
  4.12 JOINTS .............................................................................................................................................. 34
  4.13 LATERAL FORCE RESISTING SYSTEM .......................................................................................... 34
  4.14 MOMENT-RESISTING FRAME ........................................................................................................ 35
7.1 REGULAR AND IRREGULAR CONFIGURATION .......................................................... 77
7.2 LATERAL FORCE ..................................................................................................... 92
7.3 DESIGN IMPOSED LOADS FOR EARTHQUAKE FORCE CALCULATION ............... 99
7.4 SEISMIC WEIGHT .................................................................................................. 101
7.5 DIAPHRAGM .......................................................................................................... 101
7.6 EQUIVALENT STATIC METHOD ............................................................................ 105
7.7 DYNAMIC ANALYSIS METHOD .......................................................................... 112
7.8 TORSION ............................................................................................................... 119
7.9 RC FRAME BUILDINGS WITH UNREINFORCED MASONRY INFILL WALLS ......... 121
7.10 BUILDINGS WITH OPEN STOREYS .................................................................... 125
7.11 DEFORMATION ..................................................................................................... 129
7.12 MISCELLANEOUS ................................................................................................ 133
7.13 NONSTRUCTURAL ELEMENTS ............................................................................ 135

ANNEX A - MAP OF INDIA SHOWING EPICENTRES .................................................. 143
ANNEX B - MAP OF INDIA SHOWING PRINCIPAL TECTONIC FEATURE ............... 144
ANNEX C - MAP OF INDIA SHOWING PRINCIPAL TECTONIC FEATURE ............... 145
ANNEX D - MSK 64 INTENSITY SCALE .................................................................... 146
ANNEX E - ZONE FACTOR, Z FOR SOME CITIES ..................................................... 149
ANNEX F - CORRECTIONS IN STANDARD PENETRATION TEST (SPT) VALUES ......... 151
ANNEX G - SIMPLIFIED PROCEDURE FOR EVALUATION OF LIQUEFACTION POTENTIAL .......................................................... 152
REFERENCES FOR COMMENTARY ......................................................................... 161
Foreword

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

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

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

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

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

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

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

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

- **Part 1**: General provisions and buildings
- **Part 2**: Liquid retaining tanks—Elevated and ground supported
- **Part 3**: Bridges and retaining walls
- **Part 4**: Industrial structures, including stack-like structures
- **Part 5**: Dams and embankments *(to be formulated)*

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

In the 2016 sixth revision, the following changes have been included:

- **a)** Design spectra are defined for natural period up to 6 s;
- **b)** Same design response spectra are specified for all buildings, irrespective of the material of construction;
- **c)** Bases of various load combinations to be considered have been made consistent for earthquake effects, with those specified in the other codes;
- **d)** Temporary structures are brought under the purview of this standard;
- **e)** Importance factor provisions have been modified to introduce intermediate importance category of buildings, to acknowledge the density of occupancy of buildings;
- **f)** A provision is introduced to ensure that all buildings are designed for at least a
CODE

- minimum lateral force;
- Buildings with flat slabs are brought under the purview of this standard;
- Additional clarity is brought in on how to handle different types of irregularity of structural system;
- Effect of masonry infill walls has been included in analysis and design of frame buildings;
- Method is introduced for arriving at the approximate natural period of buildings with basements, step back buildings and buildings on hill slopes;
- Provisions on torsion have been simplified; and
- Simplified method is introduced for liquefaction potential analysis.

COMMENTARY

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;

- Improved relation for calculating approximate fundamental period of buildings with structural walls;
- Minimum design base shear is enhanced in view of new findings;
- Forces for strength design of diaphragm components is included;
- 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-
CODE

normative content can also be moved to commentary when it becomes the integral part of the code.

d) A more detailed treatment of SSI 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 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.

COMMENTARY

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


<table>
<thead>
<tr>
<th>CODE</th>
</tr>
</thead>
<tbody>
<tr>
<td>2012</td>
</tr>
</tbody>
</table>

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

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

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

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value observed or calculated, expressing the result of a test or analysis, shall be rounded off in accordance with IS 2: 1960 'Rules for rounding off numerical values (revised)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.
# Proposed Modifications & Commentary IS:1893 (Part 1)

## CODE

### 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.

### 1.4

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

## COMMENTARY

### C1.1

Structures designed with this standard in general should be able to resist moderate intensity of ground shaking without structural damage and with some possibility of damages to non-structural components. They resist the ground motions having high intensity of shaking in expected design level of earthquake without collapse, but possibly expecting structural as well as non-structural damages.

It is expected that for even a major design level earthquake, the damages be limited to a repairable level. However, these provisions do not guarantee that significant structural damages will not occur under the maximum level of earthquake ground motion.

Provisions of this code are not applicable for large dams, nuclear installations and other hazardous industries, unless specified otherwise by the project authority.

Please refer to clause 7.12.6.

### C1.2

The latest versions of the codes mentioned are as follows:


Please refer to clause 7.12.5.
CODE

1.5

1.3

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

COMMENTARY


- IS 13920 : 2016 – Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces – Code of Practice


- IS 15988-2013: Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings - Guidelines

It should be noted that IS 1893 Part I is primarily about specifying general provisions for design earthquake force and those related to buildings. Similarly, other parts of the code deal with specific nature of those structures, such as bridges, industrial structures, etc. as described in the Foreword of the code. In addition, there are industry-specific standards, e.g., Atomic Energy Regulatory Board for nuclear power plants, Central Water Commission for dams, Indian Road Congress for highway bridges and Research and Design Standards Organization for railway bridges, etc.
2 References

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

<table>
<thead>
<tr>
<th>IS No.</th>
<th>Title</th>
</tr>
</thead>
<tbody>
<tr>
<td>800: 2007</td>
<td>Code of practice for general construction in steel (Second revision)</td>
</tr>
<tr>
<td>875</td>
<td>Code of practice for design loads (other than earthquake) for buildings and structures:</td>
</tr>
<tr>
<td>(Part 1): 1987</td>
<td>Dead loads – Unit weights of building material and stored materials (second revision)</td>
</tr>
<tr>
<td>(Part 2): 1987</td>
<td>Imposed loads (second revision)</td>
</tr>
<tr>
<td>Snow loads (second revision)</td>
<td>Wind loads (third revision)</td>
</tr>
<tr>
<td>(Part 3): 2015</td>
<td>Snow loads (second revision)</td>
</tr>
<tr>
<td>(Part 4): 1987</td>
<td>Special loads and load combinations (second revision)</td>
</tr>
<tr>
<td>1343: 2012</td>
<td>Classification and identification of soils for general engineering purposes (first revision)</td>
</tr>
<tr>
<td>1498: 1970</td>
<td>Method of load test on soils (second revision)</td>
</tr>
<tr>
<td>1888: 1982</td>
<td>Criteria for earthquake resistant design of structures:</td>
</tr>
<tr>
<td>1893</td>
<td>Liquid retaining tanks</td>
</tr>
<tr>
<td>(Part 2): 2014</td>
<td>Bridges and retaining walls</td>
</tr>
<tr>
<td>(Part 3): 2014</td>
<td>Bridges and retaining walls</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>------</td>
<td>------------</td>
</tr>
<tr>
<td>(Part 4) : 2015</td>
<td>Industrial structures including stack-like structures <em>(first revision)</em></td>
</tr>
<tr>
<td>2131: 1981</td>
<td>Method of standard penetration test for soils <em>(first revision)</em></td>
</tr>
<tr>
<td>2809:1972</td>
<td>Glossary of terms and symbols relating to soil engineering <em>(first revision)</em></td>
</tr>
<tr>
<td>2810: 1979</td>
<td>Glossary of terms relating to soil dynamics <em>(first revision)</em></td>
</tr>
<tr>
<td>2974</td>
<td>Code of practice for design and construction of machine foundations:</td>
</tr>
<tr>
<td>(Part 1) : 1982</td>
<td>Foundation for reciprocating type machines</td>
</tr>
<tr>
<td>(Part 2) : 1980</td>
<td>Foundations for impact type machines (Hammer foundations)</td>
</tr>
<tr>
<td>(Part 3) : 1992</td>
<td>Foundations for rotary type machines (Medium and high frequency)</td>
</tr>
<tr>
<td>(Part 4) : 1979</td>
<td>Foundations for rotary type machines of low frequency</td>
</tr>
<tr>
<td>(Part 5) : 1987</td>
<td>Foundations for impact machines other than hammer (Forging and stamping press, pig breaker, drop Crushe and jolter)</td>
</tr>
<tr>
<td>4326:2013</td>
<td>Earthquake resistant design and construction of buildings - Code of practice <em>(third revision)</em></td>
</tr>
<tr>
<td>6403: 1981</td>
<td>Code of practice for determination of bearing capacity of shallow foundations <em>(first revision)</em></td>
</tr>
<tr>
<td>13827:1993</td>
<td>Improving earthquake resistance of earthen buildings – Guidelines</td>
</tr>
<tr>
<td>13828:1993</td>
<td>Improving earthquake resistance of low strength masonry buildings – Guidelines</td>
</tr>
<tr>
<td>13920:2016</td>
<td>Ductile design and detailing of reinforced concrete structures subjected to seismic forces - Code of practice <em>(first revision)</em></td>
</tr>
<tr>
<td>3935:1993</td>
<td>Repair and seismic strengthening of buildings – Guidelines</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>---------------------</td>
<td>------------------------------------------------------------</td>
</tr>
<tr>
<td>15988:2013</td>
<td>Seismic evaluation and strengthening of existing reinforced concrete building — Guidelines</td>
</tr>
<tr>
<td>8009:</td>
<td>Code of Practice for Calculation of Settlement of Foundations</td>
</tr>
<tr>
<td>Part I: 1976</td>
<td>Shallow Foundations subjected to Symmetrical Static Vertical Loads</td>
</tr>
<tr>
<td>Part II: 1980</td>
<td>Deep Foundations subjected to Symmetrical Static Vertical Loading</td>
</tr>
</tbody>
</table>
3 – Terminology

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

3.1 Closely-Spaced Modes

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

3.2 Critical Damping

The damping beyond which the free vibration motion will not be oscillatory.

3.3 Damping

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

C3.3– Damping

When a system is allowed to vibrate freely (with no external forces acting on the system), amplitude of vibrations decays with time due to loss of energy in a number of ways. This type of vibration is called a damped vibration. Damped vibration of real life systems is a complex phenomenon. However, mathematically it is convenient to assume an equivalent damping force of magnitude proportional to velocity and direction opposite to the movement of the system. Such simplified damping is termed as viscous damping. In real life problems, even if the damping is of other kind, an equivalent viscous damping is convenient for calculations.

3.4 Design Acceleration Spectrum

Design acceleration spectrum refers to an average smoothened graph of maximum acceleration as a function of natural frequency

C3.4– Design Acceleration Spectrum

Use of two different terms used in earlier edition of the code, namely ‘Response Spectrum’ and ‘Design Acceleration Spectrum’, for essentially
<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>or natural period of oscillation for a specified damping ratio for the expected earthquake excitations at the base of a single degree of freedom system.</td>
<td>the same thing often confused the users. Therefore, the term ‘Design Acceleration Spectrum’ has now been used to indicate the graph of response spectrum with natural period that is used in design.</td>
</tr>
</tbody>
</table>

### 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.

**C3.5 Design Basis Earthquake**

See the commentary of clause 3.16.

### 3.6 Design Horizontal Acceleration Coefficient ($A_h$)

It is a horizontal acceleration coefficient that shall be used for design of structures.

**C3.6– Design Horizontal Acceleration Coefficient ($A_h$)**

This is the factor, which on multiplying by seismic weight of structure gives design horizontal seismic force on the structure.

### 3.7 Design Horizontal Force

It is the horizontal seismic force prescribed by this standard that shall be used to design a structure.

### 3.8 Ductility

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

**C3.8– Ductility**

Ductility is a very important property, especially when the structure is subjected to seismic loads. Ductile structures have been found to perform much better in comparison to brittle structures. High ductility allows a structure to undergo large deformations before it collapses.

### 3.9 Epicentre

It is the geographical point on the surface of

**C3.9– Epicentre**

Distance from epicenter to any point of interest is
3.9

3.10 Floor Response Spectrum

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

3.11 Geotechnical Field Test Parameters

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

a) $N$-value: Value of Standard Penetration Test at a depth, to which corrections are applied as per Annexure - 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 Importance Factor ($I$)

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

3.13 Intensity of Earthquake

It is the measure of the strength of ground shaking manifested at a place during the earthquake vertically above the point of origin of the earthquake.

called epicentral distance.

For more information on seismological terms one may refer http://www.iitk.ac.in/nicee/EQTips/EQTip03.pdf

C3.10– Floor Response Spectra

Like ground response spectrum, the floor response spectrum can be determined for acceleration, velocity and displacement. Floor response spectrum is used for seismic qualification of important equipment and auxiliary systems mounted on a floor of a structure.

C3.13– Intensity of Earthquake

Intensity is a qualitative measure of the actual manifestation of earthquake shaking at a location during an earthquake. The intensity at a place is
CODE

earthquake, and is indicated by a roman capital numeral on the MSK scale of seismic intensity (see Annex D).

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).

3.13

3.15 Lithological Features

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

3.16 Maximum Considered Earthquake (MCE)

The most severe earthquake considered by this standard.

COMMENTARY

evaluated considering three features of shaking – perception by people, performance of buildings, and changes to natural surroundings. It is denoted in a roman capital numeral. There are many intensity scales. Two commonly used ones are the Modified Mercalli Intensity (MMI) Scale and the MSK Scale (Annexure D of this code). Both scales are quite similar and range from I (least perceptive) to XII (most severe).

C3.14– Liquefaction

During earthquake shaking, contractive soils, e.g., loose saturated sand tends to densify, leading to build up of pore water pressure. Rapid and repeated cycling of loading and unloading during ground shaking does not allow the pore water pressure to dissipate during the shaking. Resulting increase in pore water pressure leads to a decrease in effective stress and a significant loss of soil shear strength. This can lead to catastrophic failure of structures supported on or through such deposits. In a more realistic scenario, dissipation of earthquake-related excess pore water pressure results in large permanent ground deformations that affect the functionality of structures constructed above. Similar deformations have also been found to affect structures at sites underlain by soils containing cohesive fines, e.g., marine and sensitive clayey deposits, in past earthquakes particularly if the liquid limit is less than 47 % and plasticity index less than 20 %.
proposed modifications & commentary is:1893 (part 1)

province. it is generally an upper bound of the expected magnitude on a fault or in a tectonic province, irrespective of the return period of the earthquake which may range from say 100 years to 10,000 years. it is usually evaluated on the basis of geological evidence.

other terms used in the literature that are somewhat similar to mce are ‘maximum possible earthquake’, ‘maximum expectable earthquake’, ‘maximum probable earthquake’ and ‘maximum considered earthquake’.

maximum considered earthquake (mce) is defined in the international building code 2000 (usa) corresponding to an earthquake having a 2% probability of being exceeded in 50 years, i.e., 2,500 year return period. in the uniform building code 1997 (usa) maximum considered earthquake is defined as an earthquake having 10% probability of being exceeded in 100 years, i.e., 1,000 year return period. for a given area, mce based on 2,500 year return period will be larger than the mce based on 1,000 year return period.

in is1893, however, mce motion does not correspond to any specific probability of occurrence or return period and is somewhat similar to the maximum credible earthquake used in other international codes.

design basis earthquake is the earthquake motion for which the structure is to be designed, in general, considering inherent conservatism in the design process. in the ubc 1997 and ibc 2000, it corresponds to an earthquake having 10% probability of being exceeded in 50 years, i.e., 475 year return period.

the ratio of peak ground acceleration for a 2500 year return period versus that for a 500 year return period will vary for different seismic regions. for example, atc 18-1997 shows this ratio to be 1.06 for san francisco, 1.28 for san diego, 1.43 for seattle, and 2.33 for boston. however, for simplicity, the building codes tend to take constant value for this ratio. for example, ibc 2000 assumes design basis earthquake as two-thirds of maximum considered earthquake.

in is 1893, the zone map is not probabilistic and the acceleration values for mce and dbe do not correspond to any specific probability of occurrence (or return period). as an empirical approach, design basis earthquake motion has been assumed as one half of maximum considered earthquake and this is reflected by factor 2 in the
3.14

3.17 Modal Mass \((M_k)\) in Mode \((k)\) of a Structure

It is a part of the total seismic mass of the structure that is effective in natural mode \(k\) of oscillation during horizontal or vertical ground motion.

C3.17– Modal Mass \((M_k)\)

Mass of the structure that is effective in one particular natural mode of vibration is termed as modal mass for that mode. For simple lumped mass systems, the modal mass can be obtained using the equation in clause 7.7.5.4. Sum of modal masses of all the modes is equal to the total mass of structure. Generally, only first few modes are considered for seismic analysis.

3.15

3.18 Modal Participation Factor \((P_k)\) in Mode \((k)\) of a Structure

The amount by which natural mode \(k\) contributes to overall oscillation of the structure during horizontal or vertical earthquake ground motion. Since the amplitudes of mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for defining mode shapes.

C3.18– Modal Participation Factor \((P_k)\)

It is a term used in dynamic analysis (clause 7.7.5.4).

3.16

3.19 Modes of Oscillation

See 3.22

denominator of equation for \(A_h\) (clause 6.4.2).

The use of the word “reasonably” in the definition of Design Basis Earthquake in 2002 edition of the code is vague. Given that the code does not propose different design earthquakes for structures with different design lives, definition of design basis earthquake has been modified in clause 3.5.
3.17

3.20 Mode Shape Coefficient 
\((\Phi_{ik})\)

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

3.18

3.21 Natural Period \((T_k)\) in Mode \((k)\) of oscillation

The time taken (in second) by the structure to complete one cycle of oscillation in its natural mode \(k\) of oscillation.

C3.21—Natural Period \((T)\)

Consider a single degree of freedom (SDOF) system. When some initial disturbance (displacement and/or velocity) is given to this SDOF system, it starts vibrating and soon settles into a harmonic motion, where the mass swings back and forth. This vibration is called free vibration. The time required to complete one oscillation of free vibration is called natural period of the SDOF system.

A multi-degree of freedom system with masses at different locations can undergo free vibration oscillations in different normal mode shapes of deformation. In each of these normal modes of vibration, the structure takes a definite amount of time to complete one cycle of motion; this time taken to complete one cycle of motion is called natural period of motion of that normal mode of vibration.

3.19

3.22 Normal Mode of Oscillation

The mode of oscillation in which there are special undamped free oscillations in which all points on the structure oscillate harmonically at the same frequency (or period), such that all
CODE

these points reach their individual maximum responses simultaneously.

3.20

3.23 Peak Ground Acceleration

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

COMMENTARY

C3.23 – Peak Ground Acceleration (PGA)

Figure C1– Horizontal ground motion at El Centro during Imperial Valley earthquake

Figure C1 shows a typical ground motion record wherein ground acceleration is shown on vertical axis and time on horizontal axis. The largest value of ground acceleration is termed as peak ground acceleration. Usually, ground motion is recorded in two mutually perpendicular horizontal directions and the vertical direction. Hence, PGA value can be different in different directions. Vertical PGA value is generally taken as a fraction of the horizontal PGA.

The term Zero Period Acceleration (ZPA) indicates the maximum acceleration experienced by a rigid structure (zero natural period, i.e., $T=0$; in practice $T$ is 0.03 s or less). An infinitely rigid structure has zero natural period and does not deform, which means that (a) there is no relative motion between its mass and its base, and (b) the mass has same acceleration as of the ground. Therefore, zero period acceleration is the same as the peak ground acceleration.

The value of peak ground acceleration can, therefore, be read from the acceleration spectrum as shown in Figure C2. See C3.25 for Response Spectrum.
Figure C2 shows the average shape of acceleration response spectrum for 5% damping. It can be noted that the ordinate at 0.1 to 0.3 seconds \( \approx 2.5 \) times the peak ground acceleration.

The 2002 edition of the code used this relationship to define effective peak ground acceleration (EPGA). The term effective peak ground acceleration has been defined in numerous different ways in the literature. However, for the purpose of this code, it is not important to differentiate between EPGA and PGA. Similarly, the use of term Zero Period Acceleration (ZPA) in the 2002 edition of the code caused some confusion, while all of these three terms essentially mean the same thing. Therefore, the terms ZPA and EPGA have been dropped from the code.

3.21

3.24 Response reduction Factor \((R)\)

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

3.22

3.25 Response Spectrum

It is the representation of maximum responses of a spectrum of idealized single degree freedom systems of different natural periods

C3.24—Response Reduction Factor \((R)\)

See commentary of clause 6.4.2.

C3.25—Response Spectrum

During ground shaking it is possible to plot a graph between ground acceleration and time. The instrument used for this purpose is known as the...
but having the same damping, under the action of the same earthquake ground motion at their bases. The response referred to here can be maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

CODE

but having the same damping, under the action of the same earthquake ground motion at their bases. The response referred to here can be maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

COMMENTARY

‘accelerograph’ and the record thus obtained is called the ‘accelerogram’. Using a computer, one can calculate the response of single degree of freedom (SDOF) system with time, which is known as the time history of response (Figure C3). Response may mean any response quantity of interest to us, for example, displacement or acceleration at a point, or bending moment at a location in a member.

Figure C3 – (a) Ground motion time history (accelerogram), (b) time history of deformation (relative displacement of mass with respect to base) response, and (c) response spectrum developed. (From Chopra, 2001).

The maximum response can be read from the time history of the response. By repeating the same exercise for systems having different natural periods, one can draw a graph of maximum response versus natural period for a given value of damping. Such a graph of maximum response versus natural period for a given accelerogram is called the response spectrum. A response spectrum can be used to obtain the maximum

Displacement (in inches)
**CODE**

**COMMENTARY**

response of any SDOF system for that given accelerogram and given value of damping.

Unless otherwise mentioned, response spectrum is based on a linear elastic system. As stated earlier, response may mean any response quantity of interest to us, like:

1. Absolute acceleration of the mass, the response spectrum of which is termed as *acceleration response spectrum*. For the purpose of this document, response spectrum implies acceleration response spectrum.

2. Relative velocity of the mass with respect to base, the response spectrum of which is termed as *velocity response spectrum*.

3. Relative displacement of the mass with respect to base, the response spectrum of which is termed as *displacement response spectrum*.

### 3.23

**3.26 Response Acceleration Coefficient of a Structure \((S_a/g)\)**

It is a factor denoting the normalized design acceleration spectrum value to be considered for the design of structures subjected to earthquake ground shaking; this value depends on the natural period of oscillation of the structure and damping to be considered in the design of the structure.

### 3.24

**3.27 Seismic Mass of a Floor**

It is the seismic weight of the floor divided by acceleration due to gravity.

**C3.27 – Seismic Mass**

It is the seismic weight divided by the acceleration due to gravity, i.e., it is in units of mass (kg) rather than in units of weight (N or kN). While working on problems related to dynamics one needs to be careful between mass and weight. Mass times gravity is weight, i.e., the weight of 1 kg mass is equal to 9.81 N.
CODE

3.28 Seismic Mass of a Structure
It is the seismic weight of a structure above base divided by acceleration due to gravity.

3.26

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

3.27

3.30 Seismic Weight of a Structure \((W)\)
It is the sum of seismic weights of all floors above base.

3.28

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

C3.31 – Zone Factor \((Z)\)
The values of zone factor for different seismic zones in India are given in clause 6.4.2. These have been arrived at empirically using engineering judgment.

3.29

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

C3.32 Response History Analysis
Dynamic analysis can be performed either as response history analysis, or as response spectrum analysis. Response history analysis is a more sophisticated method and is rarely used for the design of ordinary structures. Response history analysis can be performed by modal superposition method or by using direct integration of equations of motion. However, only the latter method is applicable for nonlinear system. A number of text books are available that cover dynamic analysis.
4 Special Terminology for Buildings

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

4.2 Base

   It is the level at which inertia forces generated in the building are considered to be transferred to the ground through the foundation. For buildings with basements, it is considered at the bottommost basement level. For buildings resting on,
   a) pile foundations, it is considered to be at the top of pile cap;
   b) raft, it is considered to be at the top of raft; and
   c) footings, it is considered to be at the top of the footing.

   For buildings with combined types of foundation, the base is considered as the bottommost 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.

C4.2 Base

   The earlier definitions considered the floor at the grade or ground as the base for the structure as most buildings had their first floor at the grade. However, for buildings having basement storeys of sufficient stiffness, the basement portion has a little effect on the period of oscillation on the total building which is dominated by the longer period of the upper tower portion of the structure with the first floor acting as the base.

   The first floor base was used for computing the approximate fundamental period, $T_a$ for building from which the design spectral accelerations $S_a/g$ and base shear, $V_b$ was determined. Generally, for buildings with basement storeys, the base shear so determined was transferred to the soil by basement storeys of sufficient stiffness.

   Ground/ floor will still be the base for most of the buildings. With this new definition, it is important to consider the mechanism through which the earthquake forces are transmitted from the soil into the building and base is considered as the plane at which it is accomplished. As a result, the buildings with basement storeys, the base can be taken as a rigid basement floor slab or the top of the footing. If the foundation/retaining walls all around the building are stiff, the base will be at a floor above the grade. In case the basement walls are not stiff enough due to presence of openings, etc., the base will be taken at the top of the footings. If the building with basement is located in close proximity of similar buildings with basement storeys, it may be proper to consider the seismic base below the grade at the top of footings.

   Similarly, when vertical elements of buildings support the surrounding soils, the base will be considered at the grade, if the soil is stiff and competent, that is, capable of transmitting seismic
CODE

COMMENTARY

forces. In case, the surrounding soils are poor and loose, the base will be considered at the top of footings. In cases, where the site is sloped and the vertical elements resist the lateral soil pressure, the base should be taken on the side where the grade is low. Whenever in doubt, the base can be conservatively located at the lower elevation.

For set-back buildings, usually for building on a sloping site, the base can be taken as the highest contact point with the site provided very stiff structural elements are used below this point. (SEAOC 1990).

In summary, several factors affect the location of base for seismic design, such as the presence of basement storeys, stiffness of basement/foundation wall, competence of surrounding soils, manner in which lateral soil pressure is resisted, proximity of adjacent buildings, slope of the grade, etc. Some examples for the location of the seismic base is shown in Figure C4.
4.3 Base Dimension ($d$)
It is the dimension (in metre) of the base of the building along a direction of shaking.

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

4.5.1. For Single-storey buildings:
It is the point on the roof of a building through which when the resultant internal resistance acts, the building undergoes,
a) pure translation in the horizontal direction; and
b) no twist about vertical axis passing through the CR.

4.5.2. For Multi-storey buildings:
It is the set of points on the horizontal floors of a multi-storey building through which, when the resultant incremental internal resistances across those floors act, all floors of the building undergo,
a) pure translation in the horizontal direction; and
b) no twist about vertical axis passing through the CR.

C 4.5 Centre of Resistance

If a single storey building undergoes pure translation in the horizontal direction (that is, no rotation or twist or torsion about vertical axis), the point through which the resultant of the restoring forces acts is referred as the center of resistance (or rigidity).

However, in multi-storey buildings either of the following two definitions may be used for the centre of resistance (CR).

a) All-floor definition: Center of the rigidities are the set of points located one on each floor, through which application of lateral load profile would cause no rotation in any floor, (Figure C5). As per this definition, location of CR is dependent on building stiffness properties as well as on the applied lateral load profile.

b) Single-floor definition: Center of rigidity of a floor is defined as the point on the floor such that application of lateral load passing through that point does not cause any rotation of that particular floor, while the other floors may rotate (Figure C5). This definition is independent of applied lateral load.

The two definitions may give somewhat different values of design eccentricity but the difference is not very substantial. However, all floor definition is preferred option.
4.6 Eccentricity

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

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

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

4.8 Diaphragm
It is a horizontal, or nearly horizontal system, (for example, reinforced concrete floors and horizontal bracing systems), which transmits lateral forces to vertical elements connected to it.
4.9 Height of Floor ($h_i$)

It is the difference in vertical elevations (in metre) of the base of the building and top of floor $i$ of the building.

4.10 Height of Building ($h$)

It is the height of building (in metre) from its base to top of roof level.

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

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

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

C4.10 Height of Building ($h$)

This definition of the height of the building is for use in determining empirically the approximate fundamental period, $T_a$ of the entire building or its portion which will undergo oscillatory motion for horizontal components of ground motion.

The analysis and design of building structures with basement storeys should be dealt with in the same manner as the combination of different framing systems in the same direction.

Height of building shall be calculated

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

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

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

4.11 Horizontal Bracing System

It is a horizontal truss system that serves the same function as a diaphragm.

4.12 Joints

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

4.13 Lateral Force Resisting System
CODE

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

4.14 Moment-Resisting Frame

It is an assembly of beams and columns that resist induced and externally applied forces primarily by flexure.

4.14.1 Ordinary Moment-Resisting Frame (OMRF)

It is a moment-resisting frame designed and detailed as per IS 456 or IS 800, but not meeting special detailing requirements for ductile behaviour as per IS 13920 or IS 800, respectively.

4.14.2 Special Moment-Resisting Frame (SMRF)

It is a moment-resisting frame designed and detailed as per IS 456 or IS 800, and meeting special detailing requirements for ductile behaviour as per IS 13920 or IS 800, respectively.

4.14.3 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.

COMMENTARY

C4.14 – Moment-Resisting Frame

Ductile structures perform much better during earthquakes, and hence, ductile structures are designed for lower seismic forces than non-ductile structures. This can also be inferred by comparing the R values in Table 9.

In many countries, intermediate level of ductility requirements are specified for moderate seismic zones. It seems appropriate to develop somewhat lower ductility requirements for moment resisting frames of zone III than those in zones IV and V in India; these frames have been termed as Intermediate Moment Resisting Frame.
4.16—Core Structural Walls, Perimeter Columns, Outriggers and Belt Truss System

It is a structural system comprising of a core of structural walls and perimeter columns, resisting the vertical and lateral loads, with

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

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

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

4.17

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.

C4.16

Irrespective of building plan geometry, only the configuration of the lateral force resisting elements in the plan shall be considered for determining the principal orthogonal plan axes. Refer Figure C6 in which structural walls of trapezoidal building plan are majorly oriented along orthogonal axes. Generally, an inclination of 10 to 15 degrees for vertical elements can be ignored.
**CODE**

**COMMENTARY**

![Figure C6. Principal orthogonal plan axes in a building](image)

4.18

4.17 P-Δ Effect

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

4.19

4.18 RC Structural Wall

It is a wall designed to resist lateral forces acting in its own plane.

4.19.1

4.18.1 – Ordinary RC Structural Wall

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

4.19.2

4.18.2 – Special RC Structural Wall

It is a RC structural wall designed and detailed as per IS 13920, and meeting special detailing requirements for ductile behaviour as per IS 13920.

4.18.3 – Intermediate Structural Wall

C4.18 – RC Structural Wall

In recent literature, such walls are more commonly being called as Structural Walls.
CODE

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

4.19 Storey

It is the space between two adjacent floors.

4.20.1

4.19.1– Soft Storey

It 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. 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.

4.21

4.20 Storey Drift

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

4.22

4.21 Storey Shear ($V_i$)

It is the sum of design lateral forces at all levels above the storey $i$ under consideration.

COMMENTARY

C4.19 – Storey

There is a clear distinction between stiffness and strength. Stiffness is the force needed to cause a unit displacement and is given by the slope of the force-displacement relationship, whereas, strength is the maximum force that a system can take. Soft storey refers to stiffness and weak storey refers to strength. Usually, a soft storey may also be a weak storey.
### CODE

#### 4.23

#### 4.22 Storey Lateral Shear Strength \((S_i)\)

It is the total lateral strength of all lateral force resisting elements in the storey considered in a principal plan direction of the building.

### COMMENTARY

#### C4.22

Storey strength of moment resisting frames can be computed on the basis of column mechanism or beam mechanism.

If the flexural capacities of the columns are known, the story strength may be obtained as shown in Figure C7(a):

\[ V_s = \frac{2}{h} \sum_{i=1}^{n_c} M_{uc,i} \]

where

- \(V_s\) = Storey shear strength
- \(h\) = story height
- \(n_c\) = number of columns in the storey
- \(M_{uc}\) = flexural capacity of the column hinges at the top and bottom of the columns

Storey strength may also be computed on the basis of beam strengths. The mechanism for computing the story strength is as shown in Figure C7(b) and the computed capacity is
4.24

4.23 Storey Lateral Translational Stiffness \((K_i)\)

It is the total lateral translational stiffness of all lateral force resisting elements in the storey considered in a principal plan direction of the building.

C4.23

In order to avoid concentration of inelastic deformation in the soft storey and possible collapse, vertical elements of sufficient strength and stiffness should be provided to keep them nearly elastic. Storey stiffness may be calculated by the following method (Figure C8):

\[
V_u = \frac{1}{h} \sum_{i=1}^{n_b} \left[ M_{sh,i}^h + M_{sh,i}^s \right]
\]

where

- \(V_u\) = Storey shear strength
- \(n_b\) = Number of bays
- \(M_{sh,i}^h\) = Hogging moment flexural capacity at one end of the beam
- \(M_{sh,i}^s\) = Sagging moment flexural capacity at the other end of the beam
4.24 RC Structural Wall Plan Density ($\rho_{sw}$)

It is the ratio of the cross-sectional area at the plinth level of RC structural walls resisting the lateral load and the plinth of the building, expressed as a percentage.
## 5 Symbols

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

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$A_b$</td>
<td>Area of base of the structure (in $m^2$)</td>
</tr>
<tr>
<td>$A_o$</td>
<td>Design horizontal earthquake acceleration coefficient</td>
</tr>
<tr>
<td>$A_k$</td>
<td>Design horizontal earthquake acceleration spectrum value for mode $k$ of oscillation</td>
</tr>
<tr>
<td>$a_p$</td>
<td>Component amplification factor</td>
</tr>
<tr>
<td>$A_{wi}$</td>
<td>Effective cross-sectional area of structural wall $i$ in the considered direction of lateral forces (in $m^2$)</td>
</tr>
<tr>
<td>$b_i$</td>
<td>Plan dimension of floor $i$ of the building, perpendicular to the direction of earthquake shaking</td>
</tr>
<tr>
<td>$C$</td>
<td>Index for the closely-spaced modes</td>
</tr>
<tr>
<td>$C_w$</td>
<td>Effective structural wall area factor for calculation of natural period</td>
</tr>
<tr>
<td>$d$</td>
<td>Base dimension (in metre) of the building in the direction in which the earthquake shaking is considered.</td>
</tr>
<tr>
<td>$DL$</td>
<td>Response quantity due to dead load</td>
</tr>
<tr>
<td>$D_p$</td>
<td>Seismic relative distance</td>
</tr>
<tr>
<td>$e_{di}$</td>
<td>Design eccentricity to be used at floor $i$ calculated as per 7.8.2</td>
</tr>
<tr>
<td>$e_{si}$</td>
<td>Static eccentricity at floor $i$ defined as the distance between centre of mass and centre of resistance</td>
</tr>
<tr>
<td>$EL_x$</td>
<td>Response quantity due to earthquake load for horizontal shaking along $X$-direction</td>
</tr>
<tr>
<td>$EL_y$</td>
<td>Response quantity due to earthquake load for horizontal shaking along $Y$-direction</td>
</tr>
<tr>
<td>$EL_z$</td>
<td>Response quantity due to earthquake load for vertical shaking along $Z$-direction</td>
</tr>
<tr>
<td>$F_i$</td>
<td>Design lateral forces at the floor $i$ due to all modes considered</td>
</tr>
<tr>
<td>$F_e$</td>
<td>Design seismic force on a nonstructural element</td>
</tr>
<tr>
<td>$F_{roof}$</td>
<td>Design lateral forces at the roof due to all modes considered</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>$h$</td>
<td>Height (in metre) of structure</td>
</tr>
<tr>
<td>$h_i$</td>
<td>Height measured from the base of the building to floor $i$</td>
</tr>
<tr>
<td>$h_{sx}$</td>
<td>Storey height below level $x$</td>
</tr>
<tr>
<td>CODE</td>
<td>COMMENTARY</td>
</tr>
<tr>
<td>-----------------------------</td>
<td>---------------------------------------------------------------------------</td>
</tr>
<tr>
<td>$h_x$</td>
<td>Height of level x to which upper connection point is attached</td>
</tr>
<tr>
<td>$h_y$</td>
<td>Height of level y to which lower connection point is attached</td>
</tr>
<tr>
<td>$I$</td>
<td>Importance factor</td>
</tr>
<tr>
<td>$I_e$</td>
<td>Importance factor of the nonstructural element</td>
</tr>
<tr>
<td>$K_x$</td>
<td>Lateral translational stiffness of storey $i$</td>
</tr>
<tr>
<td>$L$</td>
<td>Dimension of a building in a considered direction</td>
</tr>
<tr>
<td>$L_{wi}$</td>
<td>Length of structural wall $i$ in the considered direction of lateral forces (in metre)</td>
</tr>
<tr>
<td>$M_k$</td>
<td>Modal mass of mode $k$</td>
</tr>
<tr>
<td>$n$</td>
<td>Number of storeys or floors</td>
</tr>
<tr>
<td>$N$</td>
<td>measured (raw) SPT blow count</td>
</tr>
<tr>
<td>$N_{60}$</td>
<td>Normalized SPT blow count for 60% energy efficiency</td>
</tr>
<tr>
<td>$(N_{160})$</td>
<td>SPT blow count normalized for vertical effective stress of 1 atmosphere (i.e., about 100 kPa) and delivery of 60% of theoretical hammer energy</td>
</tr>
<tr>
<td>$(N_{160}')$</td>
<td>Dilatancy corrected SPT blow count normalized for vertical effective stress of 1 atmosphere and delivery of 60% of theoretical hammer energy</td>
</tr>
<tr>
<td>$N_w$</td>
<td>Number of walls in the considered direction of earthquake shaking</td>
</tr>
<tr>
<td>$Q_{ik}$</td>
<td>Design lateral force at floor $i$ in mode $k$</td>
</tr>
<tr>
<td>$R$</td>
<td>Response reduction factor</td>
</tr>
<tr>
<td>$R_p$</td>
<td>Component response modification factor</td>
</tr>
<tr>
<td>$S_{a/g}$</td>
<td>Design/Response acceleration coefficient for rock or soil sites as given by Fig. 2 and 6.4.2 based on appropriate natural periods</td>
</tr>
<tr>
<td>$S_i$</td>
<td>Lateral shear strength of storey $i$</td>
</tr>
<tr>
<td>$T$</td>
<td>Undamped natural period of oscillation of the structure (in second)</td>
</tr>
<tr>
<td>$T_a$</td>
<td>Approximate fundamental period (in seconds)</td>
</tr>
<tr>
<td>$T_k$</td>
<td>Undamped natural period of mode $k$ of oscillation (in second)</td>
</tr>
<tr>
<td>$T_{v}$</td>
<td>Undamped natural period of vertical oscillation of the structure (in second)</td>
</tr>
<tr>
<td>$T_{1}$</td>
<td>Fundamental natural period of oscillation (in second)</td>
</tr>
<tr>
<td>$V_B$</td>
<td>Design seismic base shear Design base shear calculated using the approximate</td>
</tr>
</tbody>
</table>
fundamental period $T_a$

$V_B^*$: Design base shear calculated using the approximate fundamental period $T_a$ dynamic analysis method.

$V_i$: Peak storey shear force in storey $i$ due to all modes considered

$V_{ik}$: Shear force in storey $i$ in mode $k$

$V_{roof}$: Peak storey shear force in the top storey due to all modes considered

$W$: Seismic weight of the building

$W_i$: Seismic weight of floor $i$

$W_e$: Weight of the nonstructural element

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

$Z$: Seismic Zone factor

$\Delta_{Aa}$: Allowable storey drift for structure $A$

$\Delta_{Bb}$: Allowable storey drift for structure $B$

$\delta_{xA}$: Deflection at building level $x$ of structure $A$ due to design seismic load

$\delta_{yA}$: Deflection at building level $y$ of structure $A$ due to design seismic load

$\delta_{yB}$: Deflection at building level $y$ of structure $B$ due to design seismic load

$\Phi_{ik}$: Mode shape coefficient at floor $i$ in mode $k$

$\lambda$: Peak response (for example, member forces, displacements, storey forces, storey shears or base reactions) due to all modes considered

$\lambda_k$: Absolute value of maximum response in mode $k$

$\lambda_c$: Absolute value of maximum response in mode $c$, where mode $c$ is a closely-spaced mode.

$\lambda^*$: Peak response due to the closely-spaced modes only

$\rho_{ij}$: Coefficient used in Complete Quadratic Combination (CQC) method while combining responses of modes $i$ and $j$. 
6 General Principles and Design Criteria

C6. – General Principles and Design Criteria

6.1 General Principles

C6.1 – General Principles

6.1.1 – Ground Motion

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

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

6.1.2 –

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

C6.1.2 –

Ground shaking can affect the safety of structure in a number of ways, like:

1. Shaking induces inertia force
2. Sandy soil may liquefy
3. Sliding failure of founding strata may take place
4. Fire or flood may be caused as secondary effect of the earthquake

The code primarily addresses the first issue, that is, inertia force induced by ground shaking. The engineer needs to be cautious about other effects.
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.

C6.1.3 –

Large earthquakes have much lower frequency of occurrence than the smaller earthquakes. Usually, a structure may have a design life of 50-100 years. In such a case, it may be uneconomical to design a building so that it remains undamaged during a large earthquake that may take place, say, once in 500 years. A reference to Earthquake Tip 8 may be useful:
http://www.iitk.ac.in/nicee/EQTips/EQTips08.pdf

Conventionally, seismic design philosophy is stated as:
1. Minor and frequent earthquake should not cause damage
2. Moderate earthquakes should not cause significant structural damage but could have some non-structural damage and
3. Major and infrequent earthquake should not cause collapse

Hence, the structures are designed for much smaller forces than actual seismic loads during strong ground shaking on the basis of a number of factors. These factors are discussed in detail in commentary of clause 6.4.2.

Clause 6.1.3 implies that DBE relates to the “moderate shaking” and MCE relates to the “strong shaking”. While the definition of Zone Factor (Z) in clause 6.4.2 clearly states that the DBE is assumed to be 50% of MCE. We can see that these are at variance with the definitions of DBE and MCE given in clauses 3.6 and 3.19 of 2002 edition of the code.

Last paragraph of clause 6.1.3 has special relevance to moderate seismic regions. The design seismic force provided in the code is a reduced force considering the overstrength, redundancy and ductility. Hence, even if design force due to actions other than seismic actions (say, wind force) exceed those due to design seismic force, one needs to comply with the seismic requirements on design, detailing and construction.
6.1.4 –
Members and connections of reinforced and prestressed concrete members shall be designed (as per IS 456 and IS 1343) to ensure that premature failure does not occur due to shear or bond. Some provisions for appropriate ductile detailing of RC members are given in IS 13920. Members and their connections of steel structures should be so proportioned that high ductility is obtained in the structure, avoiding premature failure due to elastic or inelastic buckling of any type. Some provisions for appropriate ductile detailing of steel members are given in IS 800.

C6.1.4
The earthquake resistant structures should generally be ductile. IS13920: 2016 deals with the ductile detailing requirements for reinforced concrete structures. Ductile detailing provisions for steel structures are not explicitly covered in the Indian codes (IS 800). Hence, reference has been made to SP6 (Part 6), but it really relates to plastic design. Thus, it is advisable to refer to codes of other countries and literature along with IS 800 for ductile detailing of steel structures.

As of now, ductile detailing provisions for precast structures and for prestressed concrete structure are not provided in the Indian codes and specialist literature or codes of other countries may be referred to. Connections in precast structures in high seismic regions require special attention. Poor performance of precast structures has been largely attributed to poor performance of connections.

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.

C6.1.5 – Soil Structure Interaction
If there is no structure, motion of the ground surface is termed as free field ground motion. In normal practice, the free field motion is applied to the structure base assuming that the base is fixed. But this is valid only for structures on rock sites. It may not be an appropriate assumption for soft soil sites. Presence of a structure modifies the free field motion since the soil and the structure interact, and the foundation of the structure experiences a motion different from the free field ground motion. Soil structure interaction (SSI) accounts for this difference between the two motions. The soil structure interaction generally decreases lateral seismic forces on the structure, and increases lateral displacements and secondary forces associated with P-delta effect. For ordinary buildings, the soil structure interaction is usually ignored. One may refer to NEHRP (FEMA 450) provisions for seismic regulations or ASCE 7-16, for a simple procedure to account for soil-structure interaction in buildings.

SSI is not to be confused with site effects. Site effects refer to the fact that free field motion at a site due to a given earthquake depends on the properties and geological features of the subsurface soils also.
6.1.6 –
Equipment and other systems, which are supported at various floor levels of the structure, will be subjected to different motions at their support points. In such cases, it may be necessary to obtain floor response spectra for design of equipment supports. For details, reference may be made to IS 1893 (Part 4).

C6.1.6
See clause 3.10

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 \( 5\% \), 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

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

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

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

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

C6.2 – Assumptions

The note mentioned after assumption (a), has been necessitated in view of experiences such as that in Mexico city (1985).

The earthquake occurred 400 kms from the Mexico City. A great variation in damages was seen in the Mexico City. Some parts experienced very strong shaking whereas some other parts of the city hardly felt any motion. The peak ground acceleration at soft soils in the lake zone was about 5 times higher than that at the rock sites though the epicentral distance was same at both the locations. Extremely soft soils in lake zone amplified weak long-period waves. The natural period of soft clay layers happened to be close to the dominant period of incident seismic waves and it created a resonance-like conditions. Buildings between 7 and 18 storeys suffered extensive damage since the natural period of such buildings was close to the period of seismic waves.

The probability of occurrence of strong earthquake shaking is low. So is the case with strong winds. Therefore, the possibility of strong ground shaking and strong wind occurring simultaneously is very low. Thus, it is commonly assumed that earthquakes and winds of very high intensity do not occur simultaneously. Similarly, it is assumed that strong earthquake shaking and maximum flood or sea waves will not occur at the same time.

It is difficult to precisely specify the modulus of elasticity of materials such as concrete, masonry and soil because its value depends on factors such as stress level, loading condition (static versus dynamic), material strength and age of material.

For such materials, there tends to be large variation in the value of $E$. For instance, IS 456:1978 recommended $E_c = 5700\sqrt{f_{ck}}$, whereas IS 456:2000 has modified the value to $5000\sqrt{f_{ck}}$; both under static condition. Further, the actual concrete strength will be different from the specified value.
Proposed Modifications & Commentary IS:1893 (Part 1)

6.3 Load Combinations and Increase in Permissible Stresses.

6.3.1 – Load Combinations

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

6.3.1.1

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

C6.3.1.1

The design forces prescribed by this standard may be lower than the actual seismic loads which requires higher energy dissipation resulting from inelastic behaviour of the structure. Therefore, it is essential that the lateral force resisting system shall conform to the special design and ductile detailing requirement which shall ensure their satisfactory performance. Thus, in cases even when the wind loads govern the strength and drift design, the provisions related to ductile detailing shall be followed.

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

C6.3.2 – Design Horizontal Earthquake Loading

C6.3.2.1 –

It is expected that the peak ground acceleration does not occur simultaneously in two perpendicular horizontal directions. Consider a building in which horizontal load is resisted by frames and or walls oriented in two orthogonal directions, say X and Y (see Figure C9). This clause requires that design ground motion be considered to act separately in X direction and in Y direction, i.e., the design motion in the X direction is assumed to not act simultaneously.
shall be determined as per 6.3.2.2.

with that in the Y direction. If at a given instant, motion is in any direction other than X or Y, one can resolve it into X and Y components, and the building will still be safe if it is designed for X and Y motions separately.

The load $EL$ in clause 6.3.1 implies earthquake load in $+X$, $-X$, $+Y$ and $-Y$ directions. A building with frames and or walls oriented in two orthogonal directions needs to be designed for the following 13 load combinations:

1. $1.5 \ (DL + IL)$
2. $1.2 \ (DL + IL + ELx)$
3. $1.2 \ (DL + IL - ELx)$
4. $1.2 \ (DL + IL + ELy)$
5. $1.2 \ (DL + IL - ELy)$
6. $1.5 \ (DL + ELx)$
7. $1.5 \ (DL - ELx)$
8. $1.5 \ (DL + ELy)$
9. $1.5 \ (DL - ELy)$
10. $0.9DL + 1.5 \ ELx$
11. $0.9DL - 1.5 \ ELx$
12. $0.9DL + 1.5 \ ELy$
13. $0.9DL - 1.5 \ ELy$

where

$ELx =$ Response quantity due to design earthquake load in $X$ direction and,

$ELy =$ Response quantity due to design

Figure C9 – Earthquake loading in both directions
6.3.2.2 –

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

(a) \(\pm EL_x \pm 0.3 EL_y\), and 
(b) \(\pm 0.3 EL_x \pm EL_y\),

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

1) \[1.2 [DL + IL \pm (EL_x \pm 0.3 EL_y)]\] and \[1.2 [DL + IL \pm (EL_y \pm 0.3 EL_x)]\],
2) \[1.5 [DL \pm (EL_x \pm 0.3 EL_y)]\] and \[1.5 [DL \pm (EL_y \pm 0.3 EL_x)]\], and
3) \[0.9 DL \pm 1.5 (EL_x \pm 0.3 EL_y)\] and \[0.9 DL \pm 1.5 (EL_y \pm 0.3 EL_x)\].

C6.3.2.2 –

In structures with non-orthogonal lateral load resisting system, the lateral load resisting elements may be oriented in a number of directions. Designing for \(X\) and \(Y\) direction loads acting separately may be unconservative for elements not oriented along \(X\) and \(Y\) directions.

A lateral load-resisting element (frame or wall) offers maximum resistance when the load is in the direction of the element. But in structures with non-orthogonal lateral load resisting systems, it may be tedious to apply lateral loads in each of the directions in which the elements are oriented. In such cases, the building may be designed for (Figure C10):

- 100% design earthquake load in \(X\) direction and 30% design earthquake load in \(Y\) direction, acting simultaneously
- 100% design earthquake load in \(Y\) direction and 30% design earthquake load in \(X\) direction, acting simultaneously

Similarly, when a building is torsionally unbalanced about both the orthogonal axes, it is advisable that the building be designed as per the “100% + 30% rule” described above.

The directions of earthquake forces are reversible. Hence, all combinations of directions are to be considered. Thus, \(EL\) in 6.3.1 now implies following eight possibilities:

\(+ (EL_x + 0.3 EL_y)\) \quad \(+ (0.3 EL_x + EL_y)\) \\
\(+ (EL_x - 0.3 EL_y)\) \quad \(+ (0.3 EL_x - EL_y)\) \\
\(-(EL_x + 0.3 EL_y)\) \quad \(-(0.3 EL_x + EL_y)\)
\(-(EL_x - 0.3 EL_y)\) \quad \(-(0.3 EL_x - EL_y)\)

Therefore, the total design load combinations will be 25.

Imposed (live) load in combinations mentioned in these clauses are as per clause 7.3.
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

C6.3.3.1 - Vertical components generally become important for near-filed earthquakes and for some long horizontal structural components in far-field earthquakes. In most cases, horizontal motions remain far more dominating than vertical motions.
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 0.32 may be used in additive or in counteracting manner to 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) \[1.2 \times [(1.0 + 0.3 ZI) DL + (0.3 ELX ± 0.3 ELY)]] \]

b) \[1.5 \times [(1.0 + 0.3 ZI) DL + (0.3 ELX ± 0.3 ELY)]] \]

c) \[0.9 \times [(1.0 – 0.3 ZI) DL ± 1.5 (0.3 ELX ± 0.3 ELY)]] \]

C6.3.3.3-
The alternative method assumes that the direct addition of response is not necessary because the simultaneous occurrence of maximum structural response of vertical and horizontal accelerations (direct and orthogonal) is unlikely. This load combination can be used to include the effect of vertical ground motion on an element basis as well as for the entire structure. The analysis for vertical motion for the element under consideration can be performed on a partial model of the element with appropriate substructure.

The value of 0.3ZI has been obtained by assuming the vertical motion to be 2/3 of the horizontal ground motion, 30% of the 100-30% combination rule (acting in conjunction with horizontal motion), design spectral acceleration \(S_a/g\) as \(8/3\) assuming the structure to be rigid for vertical oscillations and taking minimum value of 1.0 for \(R\) assuming nearly elastic response, that is,

\[0.3 \times \frac{2}{3} \times \frac{Z}{2} \times \frac{1}{1} \times \frac{8}{3} = 0.3ZI\]

This level of vertical earthquake effects still remains on the lower side — about 40% less than what is typically followed by US codes historically since recommended by ATC 3-06 (1978).

Imposed (live) load in combinations mentioned in these clauses are as per clause 7.3.

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

C6.3.4 – Combination for Two or Three Component Motion

C6.3.4.1 –
In complex structural systems (such as a nuclear reactor), one needs to consider earthquake motion in all three directions for a 3-D dynamic analysis.
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 $EL$ in the above referred load combinations shall be as given below:

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

6.3.4.2 –

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

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

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

C6.3.4.2 –

When using SRSS method, the signs of the stress resultants (e.g., axial force, shear force, bending moment) of members are lost. Thus, the engineer should carefully assign the sign to the response quantities for the most conservative result.
6.3.4.3 –
Procedure for combining shaking effects given by 6.3.4.1 and 6.3.4.2 apply to the same response quantity (say, bending moment in a column about its major axis, or storey shear in a frame) due to different components of the ground motion.

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

6.3.5 – 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 (EIₓ ± 0.3 EIᵧ)
1.2 [(1.0+ 0.3 ZI) DL + IL] ± 0.5R (EIᵧ ± 0.3 EIₓ)
1.2 [(1.0+ ZI) DL + IL] ± 0.5R (0.3 EIₓ ± 0.3 EIᵧ)

C6.3.5
This special load combination with amplified seismic loads is developed to ensure that the non-ductile or brittle component of a lateral force resisting system or part of a seismic load path have adequate strength. In other words, this factor is only for certain elements whose failure may result in loss of complete seismic lateral force resisting system or instability and collapse, and not for the entire structure. Elements designed with the amplified seismic loads may not necessarily remain elastic during design level earthquake. However, they are expected to suffer less damages and have low likelihood of collapse.

Imposed (live) load in combinations mentioned in these clauses are as per clause 7.3.

6.3.6.1 – Increase in Allowable Net Bearing Pressure on Soils in Design of Foundations

6.3.6.1.1
In the design of foundations, unfactored loads shall be combined in line with IS 2974, while assessing the bearing pressure in soils.

C6.3.6 –

C6.3.6.1 – Increase in Permissible Stress in Materials
Since maximum earthquake load is occurring only for a short duration and probability of such occurrence is very low, the code allows higher allowable stresses for load cases involving seismic loads when the working stress design method is adopted.

C6.3.6.2 – Increase in allowable pressure in
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 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.

Soils
Many modern codes, e.g., the International Building Code (IBC2000), classify the soil type as per weighted average in top 30m based on:

- Soil shear wave velocity, or
- Standard penetration resistance, or
- Soil un-drained shear strength

Table 1 defines different types of soils differently from the 2002 version. Some of the soil group symbols used in 2002 edition of the code were not consistent with the standard soil classification system.

Any of these three parameters mentioned in Table 2 can be used for the classification of soils. However, shear wave velocity based classification is preferred especially for rock sites.
### Table 1 - Percentage Increase in Net Allowable Bearing Pressure and Skin Friction of Soils
**(Clause 6.3.66.2)**

<table>
<thead>
<tr>
<th>Sl No.</th>
<th>Soil Type</th>
<th>Percentage Increase Allowable</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Type A: Rock, hard and dense soils</td>
<td>50</td>
</tr>
<tr>
<td>2</td>
<td>Type B: Medium dense or stiff soils</td>
<td>25</td>
</tr>
<tr>
<td>3</td>
<td>Type C: Soft and loose soils</td>
<td>0</td>
</tr>
</tbody>
</table>

**NOTES**
1. The net allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888-8009.
2. Only corrected values of $N$ shall be used.
3. 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 $N/N_1$ shall be as specified below:

<table>
<thead>
<tr>
<th>Seismic Zone Level</th>
<th>Depth below Ground (in meters)</th>
<th>(N/N_1) Values</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>II, III, IV and V</td>
<td>(\leq 5)</td>
<td>15</td>
<td>For values of depths between 5m and 10m, linear interpolation is recommended.</td>
</tr>
<tr>
<td></td>
<td>(\geq 10)</td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>II (for important Structures only)</td>
<td>(\leq 5)</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td></td>
<td>(\geq 10)</td>
<td>20</td>
<td></td>
</tr>
</tbody>
</table>

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 $\sigma_v$ using relation $N = C_N N_1$, where $CN = \left(\frac{P_a}{\sigma_v'}\right) + 1$, $P_a$ 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 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 6.3.56.2)

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Corrected SPT value $(N'_1)$</th>
<th>Tip resistance of CPT $q_c/P_a$</th>
<th>Shear wave velocity $V_s$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type – A</td>
<td>$&gt;30$</td>
<td>$&gt;100$</td>
<td>$&gt;360$</td>
</tr>
<tr>
<td>Type – B</td>
<td>15-30</td>
<td>50 – 100</td>
<td>180 – 360</td>
</tr>
<tr>
<td>Type – C</td>
<td>$&lt;15$</td>
<td>$&lt;50$</td>
<td>$&lt;180$</td>
</tr>
</tbody>
</table>

Note: The value of $(N'_1)_{soft}/P_a$ 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.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type A: Rock or Hard Soils</td>
<td>Well graded gravel (GW) or well graded sand (SW) both with less than 5% passing 75 μm sieve (Fines);</td>
</tr>
<tr>
<td></td>
<td>Well graded Gravel – Sand mixtures with or without fines (GW-SW);</td>
</tr>
<tr>
<td></td>
<td>Poorly graded Sand (SP) or clayey sand (SC), all having $N$ above 30;</td>
</tr>
<tr>
<td></td>
<td>Stiff to hard clays having $N$ above 30, where $N$ is the corrected Standard Penetration Test value.</td>
</tr>
<tr>
<td>Type B: Medium or Stiff Soils</td>
<td>Poorly graded sands or Poorly graded sands with gravel (SP) with little or no fines having $N$ between 10 and 30;</td>
</tr>
<tr>
<td></td>
<td>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.</td>
</tr>
<tr>
<td>Type C: Soft Soils</td>
<td>All soft soils other than SP with $N$&lt;10. The various possible soils are</td>
</tr>
<tr>
<td></td>
<td>- Silts of Intermediate compressibility (MI);</td>
</tr>
<tr>
<td></td>
<td>- Silts of High compressibility (MH);</td>
</tr>
<tr>
<td></td>
<td>- Clays of Intermediate compressibility (CI);</td>
</tr>
<tr>
<td></td>
<td>- Clays of High compressibility (CH);</td>
</tr>
<tr>
<td></td>
<td>- Silts and Clays of Intermediate to High compressibility (MI-MH or CI-CH);</td>
</tr>
<tr>
<td></td>
<td>- Silt with Clay of Intermediate compressibility (MI-CI);</td>
</tr>
<tr>
<td></td>
<td>- Silt with Clay of High compressibility (MH-CH).</td>
</tr>
<tr>
<td>Type D: Unstable, collapsible, liquefiable soils</td>
<td>Requires site specific study and special treatment according to site condition (...see 6.3.5.3)</td>
</tr>
</tbody>
</table>

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:

<table>
<thead>
<tr>
<th>% passing 75 μm IS sieve</th>
<th>((N'<em>1)</em>{60})</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\leq 5)</td>
<td>&gt;30</td>
</tr>
<tr>
<td>&gt;5 and (\leq 15)</td>
<td>&gt;25</td>
</tr>
<tr>
<td>&gt;15 and (\leq 35)</td>
<td>&gt;21</td>
</tr>
</tbody>
</table>

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 settlements in case of providing vertical drains should be accounted for in the overall design of the structure.

The criterion for undertaking investigation into liquefaction potential is based on recommendation of Caltrans Geotech Manual (2014).
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

C6.4 – Design Acceleration Spectrum

The term ‘Response Spectrum’ was introduced in clause 3.27. Seismic design force specified in terms of response spectrum is known as design spectrum.

Consider the acceleration response spectrum in Figure C12. In the region within the circle, a slight change in natural period leads to a large variation in maximum acceleration. Natural periods of civil engineering structures cannot be calculated precisely. Also, the peaks and valleys in the
CODE

response spectrum may not occur at the same values of natural periods for another earthquake ground motion even under similar site conditions. Thus, the design specifications used should not be very sensitive to a small change in natural period. Hence, design spectra are presented as smooth curves without local peaks or valleys observed in computed response spectra from individual ground motions.

Design spectrum is a design specification. It must take into consideration any issues that have bearing on seismic safety. Design spectrum must be accompanied by:

- Load factors or permissible stresses that must be used. Different choice of load factors will give different seismic safety to the structure.
- Damping to be used in design. Variation in the value of damping will affect the design force.
- Method of calculation of natural period. Depending on the modeling assumptions, one can get different values of natural period, and hence, different seismic force.
- Type of detailing for ductility. Design forces can be lowered if the structure has higher ductility.

![Figure C12 - Acceleration Response Spectrum](image)

Figure C12 – Acceleration Response Spectrum

6.4.1 –

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

C6.4.1 –

Seismic zone map in the first edition of the code (1962) was developed based on the epicentral distribution of past earthquakes and the isoseismals of such events. The enveloping lines marking areas that have sustained shaking of different intensity were then plotted to obtain a map that demarcated areas which have potential of ground shaking of intensity of: V (or less), VI, VII, VIII, IX, and X (and above). These seismic
zones were denoted as 0, I, II, III, IV, V and VI, respectively. The map was later revised in 1966 and in 1970 editions of the code considering the geological and geophysical data obtained from tectonic map and the aero-magnetic and gravity surveys. The 1966 version of the code also provided seven seismic zones. The Koyna earthquake of 1967 occurred within seismic zone I and triggered major revision of the map in the 1970 edition. It was decided to reduce the number of zones from seven to five by merging zones 0 into zone I, and zone VI with zone V. Thus, the five seismic zones of the 1970 edition corresponded to areas liable to shaking intensity of V (or less), VI, VII, VIII, and IX (and above), respectively.

The Latur earthquake of 1993 occurred in seismic zone I. A revision of the seismic zone map was undertaken and in 2002 edition of code seismic zone I was dropped by merging it with zone II; and, some parts of the peninsular India were brought into zone III. The post-earthquake reconstruction in Latur was undertaken corresponding to zone IV provisions of Indian codes: the area is classified in zone III as per current zone map.

6.4.2–

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

$$A_h = \left( \frac{Z}{2} \right) \left( \frac{S_z}{g} \right)$$

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, $Z$
CODE

(Clause 6.4.2)

<table>
<thead>
<tr>
<th>Seismic Zone Factor (1)</th>
<th>II (2)</th>
<th>III (3)</th>
<th>IV (4)</th>
<th>V (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Z</td>
<td>0.10</td>
<td>0.16</td>
<td>0.24</td>
<td>0.36</td>
</tr>
</tbody>
</table>

$R =$ 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)]:

<table>
<thead>
<tr>
<th>$S_{a/g}$</th>
<th>$T$</th>
</tr>
</thead>
<tbody>
<tr>
<td>For rocky or hard soil sites:</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td>$0 &lt; T &lt; 0.40$ s</td>
</tr>
<tr>
<td>1.25</td>
<td>$0.40 &lt; T &lt; 1.00$ s</td>
</tr>
<tr>
<td>2.0</td>
<td>$1.00 &lt; T &lt; 1.55$ s</td>
</tr>
<tr>
<td>0.75</td>
<td>$T &gt; 1.55$ s</td>
</tr>
<tr>
<td>For medium stiff soil sites:</td>
<td></td>
</tr>
<tr>
<td>1.24</td>
<td>$0.55 &lt; T &lt; 0.80$ s</td>
</tr>
<tr>
<td>1.34</td>
<td>$0.80 &lt; T &lt; 1.00$ s</td>
</tr>
<tr>
<td>1.67</td>
<td>$T &gt; 1.00$ s</td>
</tr>
<tr>
<td>For soft soil sites:</td>
<td></td>
</tr>
<tr>
<td>1.67</td>
<td>$0.67 &lt; T &lt; 0.80$ s</td>
</tr>
<tr>
<td>0.42</td>
<td>$T &gt; 0.80$ s</td>
</tr>
</tbody>
</table>

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

$S_{a/g} = S_{a/g}$

COMMENTARY

Importance Factor ($I$)

Seismic design philosophy assumes that a structure may undergo some damage during severe shaking. However, critical and important facilities must respond better in an earthquake than an ordinary structure. Moreover, high occupancy is likely to affect more persons, and therefore, requires a higher level of protection. Importance factor is meant to account for this by increasing the design force level for critical important and high occupancy structures. Importance factor facilitates the design of high consequence facilities with a reduced risk of damage in design level earthquake.

Response Reduction Factor ($R$)

The structure is allowed to be damaged in case of severe shaking. Hence, structure is designed for seismic force much less than what is expected under strong shaking if the structure were to remain linearly elastic. 1984 edition of the code just provided the required design force. It gave no direct indication that the real force may be much larger. 2002 edition onwards, the code provides for realistic force for elastic structure at Design Basis Earthquake (DBE) level and then divides that force by Response Reduction Factor ($R$). This gives the designer a realistic picture of the design philosophy. A building is expected to undergo controlled damage in case of shaking corresponding to the DBE level such that life safety of occupants can be ensured and the building can be restored. In the event of strong shaking at MCE level, it is expected that collapse of the structure is avoided, however, the building may not be repairable. This performance is achieved by detailing the structure for ductility and thereby dissipating seismic energy input to the structure. The response reduction factor, $R$ which primarily accounts for reduction in force demand due to ductility and energy dissipation can be applied by a seismic code to elastic response corresponding to either DBE or MCE level. In this code, $R$ is taken to correspond with DBE level response, and the DBE is taken as half of MCE ($Z/2$).

Consider a building in zone V. $Z=0.36g$ gives a realistic indication of ground acceleration for MCE and $0.18g$ for DBE. For $T=0.3$ sec, $S_{a/g}=2.5$ (Figure 2, IS 1893: 2016), which implies that if the building remains elastic, it may experience a maximum horizontal force equal to 90% of its weight ($0.36\times2.5=0.90$) under MCE level and 45% of its weight ($0.18\times2.5=0.45$) under DBE
Proposed Modifications & Commentary IS:1893 (Part 1)

CODE

COMMENTARY

level. If we use $R$ factor of 5 and importance factor of 1, then as per clause 6.4.2 the building is to be designed for 0.09 times its weight. Thus, the building is designed for only one tenth of the maximum elastic force of MCE level, and hence, adequate ductility and quality control for good post yield behaviour should be provided.

Over strength, redundancy and ductility together contribute to the fact that an earthquake resistant structure can be designed for force much lower than that imposed by the strong shaking (Figure C13).

a) Overstrength – The factors that account for the yielding of a structure at loads higher than the design load are:

1. Partial Safety Factors
   - Partial safety factor on seismic loads
   - Partial safety factor on gravity loads
   - Partial safety factor on materials

2. Material Properties
   - Member size or reinforcement larger than required
   - Strain hardening in materials
   - Improved strength due to confinement of concrete.
   - Higher material strength under cyclic loads

3. Strength contribution of non-structural elements

4. Additional strength due to special ductile detailing.

b) Redundancy – Yielding at one location in the structure does not imply yielding of the structure as a whole. Load redistribution in redundant structures provides additional safety margin. Sometimes, the additional margin due to redundancy is considered within the “overstrength” term.

c) Ductility – Yielding of the structure results in:

1. Higher energy dissipation due to hysteresis.
2. Softer (flexible) structure with increased natural period resulting in lower seismic force.

Higher ductility indicates that the structure can withstand stronger shaking without collapse.

According to the equivalent displacement principle, the inelastic displacement will be
CODE

about \( R \) times the elastic displacement
\( (\Delta_{\text{max}} = R\Delta_{el}) \)

**Figure C13 – Concept of Response Reduction Factor**

**Response Acceleration Coefficient \((S/\gamma)\)**

For very stiff structures (i.e., natural period for first mode < 0.1 s), ductility is not helpful in reducing the design force. Further, structures falling on the rising arm of the spectra (i.e., those with \( T < 0.1 \) s for which peak ground acceleration linearly increases to pseudo spectral acceleration) will crack once they suffer violent shaking, and their fundamental period will increase leading to higher response. If structures are designed for the rising arm coefficient, they will sustain more lateral force once they crack, than the design force. Hence, codes tend to disallow the use of the rising part of the acceleration spectrum for very short period structures, especially when equivalent static method is used for the analysis which is based on the fundamental mode of oscillation. The second paragraph of clause 6.4.2 in 2002 edition of the code attempted to ensure a minimum design force (i.e., \( A_h \) not less than \( Z/2 \), regardless of the value of \( I/R \)) for very stiff structures with fundamental natural period, \( T \) being less than 0.1 s. However, there are difficulties with this restriction and hence, to address this issue, the graphs and the equations...
CODE

COMMENTARY

giving the values for response acceleration coefficient \((S_a/g)\) have been modified in the 2016 revision of the code such that the rising part of \(S_a/g\) plot between zero and 0.1 s is not to be used for the fundamental modes of oscillation considered for the equivalent static method of analysis. However, it can be used for response spectrum analysis which typically considers many natural modes of oscillations. The two separate set of \(S_a/g\) curves included in the 2016 for the equivalent static method (no rising part for \(T<0.1s\)) and for the response spectrum analysis (with rising part for \(T<0.1s\)) are proposed to be combined in one \(S_a/g\) plot as shown in Figure 2.

Spectral acceleration in the constant-velocity portion of the design response spectrum (Newmark-Hall type) is inversely proportional to the period of the structure. In the constant-displacement region, it is inversely proportional to the square of the natural period. The transition from the constant-velocity to the constant-displacement region depends on the nonlinearity of soils and varies from 3 s to 6 s depending on characteristics of the ground motion. Using the flat spectrum over the displacement sensitive region for period greater about 4 s as done in the 2016 version, overestimates the displacement response and can lead to overly conservative design of long-period structures sensitive to displacements. Moreover, this will create anomalies in generating spectrum compatible synthetic ground motion for use in response history analysis. In the current proposal, a period of 4 s is chosen as the transition from velocity to displacement sensitive region of the spectrum. The response spectrum has appropriately been modified to account for the realistic seismic displacements experienced by structures.

Soil Effects

Recorded earthquake motions show that response spectrum shape varies with the soil profile at the site (Figure C14).

This variation in ground motion characteristics for different sites is accounted for by providing different shapes of response spectrum for each of the sites (Figure 2). The soil types A, B and C have been defined in Table 2 of the 2016 code.
Figure C14 – Recorded earthquake motions for different types of soil sites (From Geotechnical Earthquake Engineering by Kramer, 1996)
Figure 1 – Seismic Zones of India
Figure 2 – Design acceleration coefficient ($S_a/g$) for horizontal acceleration
**CODE**

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.

**COMMENTARY**

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

(Clause 6.4.2.1)

<table>
<thead>
<tr>
<th>Damping (%)</th>
<th>Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>3.20</td>
</tr>
<tr>
<td>2</td>
<td>1.40</td>
</tr>
<tr>
<td>5</td>
<td>1.00</td>
</tr>
<tr>
<td>7</td>
<td>0.90</td>
</tr>
<tr>
<td>10</td>
<td>0.80</td>
</tr>
<tr>
<td>15</td>
<td>0.70</td>
</tr>
<tr>
<td>20</td>
<td>0.60</td>
</tr>
<tr>
<td>25</td>
<td>0.55</td>
</tr>
<tr>
<td>30</td>
<td>0.50</td>
</tr>
</tbody>
</table>

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

C6.4.2.1 Damping Factors

Irrespective of the level of damping, a very stiff structure (whose T is close to zero) will not undergo any deformation relative to its base when shaken at its base. Thus, all spectra with different values of damping will start only from the PGA value. This is explained through Figure C15 for the example case of Type B stiff soil site and 10% damping.

Figure C15 – Scaling for acceleration spectrum for damping other than 5%

The response spectrum value at zero period is equal to peak ground acceleration (see commentary of clause C3.23) regardless of damping. The design acceleration spectrum given in Figure 2 is for damping value of 5 percent of critical damping. Ordinates for other values of damping can be obtained by multiplying the value for 5 percent damping with the factors given in Table 4. Note that the acceleration spectrum ordinate at zero period equals peak ground acceleration regardless of the damping value. Hence, the multiplication should be done for T ≥ 0.1sec only. For T = 0, multiplication factor will be 1, and values for 0<T<0.1sec should be interpolated accordingly.

Instead of using interpolation from tabular values, the following expression for the multiplying factor, which compares well with table 4, can be used for damping more than 5%:

\[ \frac{10}{\sqrt{5 + \zeta}} \]
For determining the correct spectrum to be used in the estimate of \( S_v / g \), 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 \( IA \) — Rock or hard and dense soils;

b) Soil type \( IB \) — Medium dense or stiff soils; and

c) Soil type \( IC \) — 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.

### Table 4

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Weilgraded gravel (GW) or wellgraded and SW without less than 5 per cent passing 75 mm sieve (Fines)</td>
</tr>
<tr>
<td>2</td>
<td>Well-graded gravel sand mixtures with or without fines (GW-SW)</td>
</tr>
<tr>
<td>3</td>
<td>Poorly graded and (SP) or clayey and (SC) all having N above 30</td>
</tr>
<tr>
<td>4</td>
<td>Stiff or hard clay having N above 30 where N standard penetration test value</td>
</tr>
<tr>
<td>5</td>
<td>Poorly graded sand or poorly graded clays and gravel (SP) with fines or fines having N between 10 and 30</td>
</tr>
<tr>
<td>6</td>
<td>Stiff medium stiff fine-grained soils like silts or low compressibility (ML) or clay or flow compressibility (CL) having N between 10 and 30</td>
</tr>
<tr>
<td>7</td>
<td>All soft soils other than SP with N &lt; 10. The various possible soils are: Soft soils — a) Silts of intermediate compressibility (ML); b) Silts of high compressibility (MH); c) Clay of intermediate compressibility (ML); d) Clay of high compressibility (CH); e) Silts and clays of intermediate to high compressibility (ML-MH or CL-CH); f) Clay with clay of intermediate to high compressibility (ML-CL); and</td>
</tr>
</tbody>
</table>
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 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 $I_{\text{gross}}$ of columns, and 35 percent of $I_{\text{gross}}$ of beams;
b) For RC and masonry walls: 50 percent of $I_{\text{gross}}$;
c) For RC slabs: 25 percent of $I_{\text{gross}}$; and
d) In steel structures: $I_{\text{gross}}$ of both beams and columns.

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

C6.4.3.1
Structural members are designed to resist earthquake loads undergoing inelastic deformations. Stiffness of RC members for linear seismic analysis using the equivalent static method, response spectrum method or response history method should be based on the stiffness of cracked sections corresponding to the first yield of the longitudinal reinforcement. The cracked or effective stiffness depends on the amount of longitudinal reinforcement, stress level due to axial loads, extent of cracking etc.

In lieu of the tedious rational analysis, the effective moment of inertia of section can be taken as the suggested percentage of the gross moment of inertia. The same effective section properties can be used for dynamic properties, design actions (forces) and drift checks. For steel moment frames, shear deformation of panel zone shall also be included.

It should be further noted that uncracked section properties shall not be used with the 5% damped design response spectrum. It has been observed
6.4.4 –
Where a number of modes are to be considered in response spectrum method, $A_h$ as defined in 6.4.2 for each mode $k$ shall be determined using the natural period $T_k$ of oscillation of that mode.

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

C6.4.5
When seismic waves hit the ground surface, these are reflected back into the ground. The reflection mechanics is such that the amplitude of vibration at the free surface is much higher (almost double) than that under the ground. This clause allows the design spectrum to be one half in case the structure is at a depth of 30m or below. Linear interpolation of design spectrum is suggested resorted to for structures with depths less than 30m. The words ‘underground structures and foundations’ have been mentioned in this clause because this clause is also applicable for calculation of seismic inertia force on foundation under the ground.

One should bear in mind that in the case of a bridge or any above-ground structure with foundation going deeper than 30m, this clause can be used only to reduce the seismic inertia force due to mass of foundation under the ground and not for the calculation of inertia force of the superstructure. The reduced $A_h$ shall be applicable for the structure or portion of the structure which is underground.

6.4.6 –
The design seismic acceleration spectral value $A_v$ of vertical motions shall be taken as:

C6.4.6 –
Usually the vertical motion is weaker than the horizontal motion. On an average, peak vertical acceleration is one-half to two-thirds of the peak horizontal acceleration.

Recent studies show that the characteristics of the acceleration response spectrum for vertical components of ground motions are significantly different from those of horizontal motions. Therefore, it is a case of oversimplification when the same acceleration response spectrum is specified for both horizontal and vertical ground motions.
Vertical ground motion is dominated by high frequency spectral content and faster attenuation (Bozorgnia and Campbell 2004, NEHRP 2009). In design codes such as EC 8 and ASCE 7-16, the acceleration response spectrum for vertical motion response is kept similar to that of the horizontal ground motion with three distinct regions: linear amplification region from peak ground value to pseudospectral acceleration, constant spectral acceleration region and the region where it decays inversely with the vertical period of vibration. Studies also indicate that vertical acceleration spectrum is not significantly affected by the soil types. Similar to EC8 and ASCE 7-16, a separate spectral acceleration spectrum as a function of vertical period of vibration is proposed for vertical motions.

\[ R \text{ values given in the Table 9 are for the horizontal component of the ground motion, for which the structure is expected to undergo inelastic deformations and dissipate seismic energy. This permits the use of elastic design forces reduced by the response reduction factor } R, \text{ depending on the degree of the ductility and energy dissipation potential of the system. However, structures are typically very stiff for vertical forces and respond elastically to the vertical component of the ground motion. Therefore, a value of } R = 1.0 \text{ should be used. However, in cases where the structure is flexible and can undergo inelastic deformations, a value larger than 1.0 can be used when substantiated by rational analysis. Following are some of the useful references on vertical seismic ground motion.} \]


\[ A_v = \left( \frac{\frac{2}{3} \times \frac{Z}{2}}{\frac{1}{R}} \right) \]

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):

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

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.
6.4.7 –

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

C6.4.7 –

Seismic design codes are generally meant for ordinary structures. For important projects, such as nuclear power plants, dams, and major bridges, site-specific seismic design criteria are used in design. Development of site specific design criteria takes into account geology, seismicity, geotechnical conditions and nature of the project. Site-specific criteria are developed by experts and usually reviewed by independent peers. Following are some of the useful references on site-specific design criteria.


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.

A complete lateral force resisting system must form a continuous load path to transfer inertial forces from the masses to the foundation for seismic effects in any horizontal direction. A complete load path with adequate strength and stiffness is the most essential requirement for all buildings. A general load path is as follows: The inertial forces originating are delivered through connections to horizontal diaphragms which distribute these forces to vertical lateral force resisting element which in turn transfers the forces to the foundation and the foundation transfers it to the supporting soil.

Any discontinuity in the load path is likely to render building unable to resist the resisting seismic forces regardless of strength of the structural elements. Therefore, the elements and also their interconnections (continuity) are important to achieve the intended seismic performance of the structure. Observations from past earthquake suggests that buildings with discontinuous load path have experienced significant damages and collapses.

More details on load path can be found here:
http://www.iitk.ac.in/nicee/EQTips/EQTip25.pdf

Adequate strength, stiffness, ductility detailing and a good construction quality is secondary if the building has a poor configuration. Though a building of an arch-shape, irregular geometry, weakness in storey or discontinuity in the lateral force resisting system may be designed to meet the requirements, the past experience tells us that such buildings do not perform as well as the buildings with no such irregularities.

Buildings with simple regular geometry and uniformly distributed mass and stiffness in plan and in elevation, suffer much less damage than buildings with irregular configurations. All efforts shall be made to eliminate irregularities by modifying architectural planning and structural configurations. A building shall be considered to be irregular for the purposes of this standard, even if any one of the conditions

Geometrically a building may appear to be regular and symmetrical, but may have irregularity due to uneven distribution of mass and stiffness.

Vertical irregularities can result into a different load distribution than those assumed in Equivalent Static Method. Plan irregularity can cause different torsional response, diaphragm deformation and local stress concentration. A
higher order 3-D dynamic analysis may be appropriate for some irregularities to assess the demand arising from them.

Various criteria to establish limits on irregularity is based on past experience or interpretation of damages in the past earthquakes. In many cases, an inelastic dynamic analysis of the complete structural and non-structural system will be required to assess the effect of these irregularities. Elastic dynamic analysis using 3-D models may help to understand the behaviour of highly irregular plan configurations, torsional effects, diaphragm deformability and their orthogonal effects.

<table>
<thead>
<tr>
<th>Table 5– Definition of Irregular Buildings – Plan irregularity (Figure 34) (Clause 7.1)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>ia) Torsion Irregularity</strong></td>
</tr>
<tr>
<td>Usually, a well-proportioned building does not twist about its vertical axis, when</td>
</tr>
<tr>
<td>a) the stiffness distribution of the vertical elements resisting lateral loads is balanced in plan according to the distribution of mass in plan (at each storey level); and</td>
</tr>
<tr>
<td>b) the floor slabs are stiff in their own plane (this happens when its plan aspect ratio is less than 3)</td>
</tr>
<tr>
<td>A building is said to be torsionally irregular, when the maximum storey drift (computed with design eccentricity) at one end of the structures transverse to an axis is more than 1.2 times the average of storey drifts at the two ends of the structure.</td>
</tr>
<tr>
<td>1) the maximum horizontal displacement of any floor in the direction of the lateral force at one end of the floor is more than 1.5 times its minimum average horizontal displacement at the far end of the same floor in that direction; and</td>
</tr>
<tr>
<td>2) the natural period corresponding to the fundamental torsional mode of oscillation is more than those of the first two translational modes of oscillation along each principal plan directions</td>
</tr>
<tr>
<td>In torsionally irregular buildings, when the ratio of maximum horizontal displacement at</td>
</tr>
</tbody>
</table>

Torsional irregularity relates to the excessive torsional flexibility or excessively reduced torsional stiffness of the system. It leads to the excessive displacement demands (possible yielding) on the vertical members located at the perimeter of the building in addition to being vulnerable to the torsional component of the ground motion.

Such irregularities may occur even when the lateral load resisting systems are symmetrically placed and may become more severe if the vertical seismic force resisting system are placed near the centre of the building.

For a simple one storey structure, the case of the torsional irregularity is shown with respect to placement of structural walls with respect to its centre with the lateral load applied at an accident eccentricity of 0.05 times the building width (Figure C16). For walls placed near the centre of the building with α close 0.1, the buildings tend to have large edge displacements compared to when walls are far away from the edge of the building with α close to 0.5. Torsional irregularities are more likely to occur for buildings which are rectangular in plan than for buildings that are square in plan.

The definition is restored to the 2002 edition because the increased deformation at the perimeter due to torsion, expressed as a function of the average displacement is indicative of the translational displacement undergone by the structure.
## CODE

| one end and the minimum horizontal displacement at the other end is |
|------------------|------------------|
| i) in the range 1.5 – 2.0, (a) the building configuration shall be revised to ensure that the natural period of the fundamental torsional mode of oscillation shall be smaller than those of the first two translational modes along each of the principal plan directions, and then (b) three dimensional dynamic analysis method shall be adopted; and |
| ii) more than 2.0, the building configuration shall be revised. |

1. For all structures, the design base shear $V_B$ (7.6.1) shall be increased, by reducing the response reduction factor $R$ to 0.75R in 6.4.2.
2. Buildings shall be designed for load combinations mentioned in 6.3.2.2, 6.3.3.3 or 6.3.4.1.

## COMMENTARY

NEHRP code also has another definition for torsionally irregular buildings: “Buildings having an eccentricity between the static center of mass and the static center of resistance in excess of 10 percent of the building dimension perpendicular to the direction of the seismic force should be classified as irregular”.

The US codes have addressed the additional demand on vertical elements by amplifying the effects of accidental eccentricity by a factor $A_x$ to represent the increased eccentricity caused by yielding of perimeter elements. This increased demand has provided effective control against excessive torsional yield in a given storey and avoided the same by ensuring the structure has adequate strength and stiffness to resist both design and accidental effects. This was first proposed in ATC 3-06 (1978) and has been part of the US codes (ASCE 07).

The provision for the amplification of accidental torsion was developed to encourage buildings with good torsional stiffness and to avoid larger deformations due to torsional flexibility (SEAOC 1990).

Eurocode EC-8 (2004) also accepts these torsionally flexible systems. However, it reduces the values of the behaviour factor ($q$) of the system recognizing the excessive displacement demand at the perimeter columns, which is disproportionate to those caused by transverse displacements. For such torsionally flexible systems, EC-8 specifies considerable reduction in the values of $q$ factor ranging from 0.66 to 0.75 indicating that design forces are increased.

The US practice of amplifying the accidental torsion is carried out at each storey. Moreover, the recent studies show that effect of this amplification needs to be calculated using equivalent static method, especially in cases of extreme torsional irregularity. Shifting the mass at each storey in a dynamic analysis is not able to capture the desired response. Moreover, the 100%-30% load combination rule can be applied to the building which mainly rely on the lateral resistance of vertical elements in orthogonal direction. (FEMA P-2012, 2018).

Since the torsional flexibility reduces the overall energy dissipation potential of the building, Eurocode EC8 is justified in reducing the value of behaviour factor and thereby enhancing the design base shear on the global basis.
In summary, both approaches are attempting to increase the design forces for vertical elements, the Eurocode approach is easier to use and is recommended in this standard. Therefore, alternative to the amplification of accidental torsion, the $R$ value is reduced to $0.75R$ for buildings with torsional irregularities.

**Figure C16- Effect of structural wall placement on maximum edge displacement due to torsional irregularity**

**ib) Extreme torsional Irregularity** *

To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structure transverse to an axis is more than 1.4 times the average of storey drifts at the two ends of the structure.

1. **Buildings having extreme torsional irregularity** with height more than 20 m shall not be permitted in zone V.
2. **For other buildings, the requirement of torsional irregularity shall be applicable.**

Extreme torsional flexibility is likely to result in much larger inelastic demands on elements at perimeter which can significantly reduce the overall energy dissipation potential of the building. Therefore, extreme torsional flexibility is restricted in high seismic zones. It should be noted that very strong and stiff buildings with large plan areas, even with significant irregularity will perform well [FEMA P-2012, 2018].
## Re-entrant Corners

A building is said to have a re-entrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15 percent of its overall plan dimension in that direction.

*For buildings up to a height of 50 m, the diaphragm elements shall be designed for 25% increase in diaphragm forces calculated as per relevant clauses. Alternatively, in buildings with re-entrant corners, three-dimensional dynamic analysis method shall be adopted.*

## Diaphragm Discontinuity (Floor Slabs having Excessive Cut-Outs or Openings)

Openings in slabs result in flexible diaphragm behaviour, and hence the lateral shear force is not shared by the frames and/or vertical members in proportion to their lateral translational stiffness. The problem is particularly accentuated when the opening is close to the edge of the slab. A building is said to have discontinuity in their in-plane stiffness, when floor slabs have cut-outs or openings of area more than 50 percent of the full area of the floor slab, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.

*In buildings with discontinuity in their in-plane stiffness, if the area of the geometric cut-out is,*

- a) less than or equal to 50 percent, the floor slab shall be taken as rigid or flexible depending on the location of and size of openings; and
- b) more than 50 percent, the floor slab shall be taken as flexible.

*Buildings more than 50 m in height shall be analyzed using 3-D dynamic analysis and the diaphragm design forces shall be increased.*

---

*Diaphragm discontinuity should not be confused with the flexibility of the diaphragm discussed in section 7.5.1.*

Significant differences between portions of the diaphragm due to differences in diaphragm configuration and/or presence of openings in the diaphragm can cause vertical distribution of seismic forces different from as it would have been with rigid floor diaphragms and create torsional forces not normally encountered in a regular building.

*Appropriate modelling of diaphragm in a 3-D model can be used for the analysis.*
### Proposed Modifications & Commentary IS:1893 (Part 1)

#### CODE

<table>
<thead>
<tr>
<th>iv) Out-of-Plane Offsets in Vertical Elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Out-of-plane offsets in vertical elements resisting lateral loads cause discontinuities and detours in the load path, which is known to be detrimental to the earthquake safety of the building. A building is said to have out-of-plane offset in vertical elements, when structural walls or frames are moved out of plane in any storey along the height of the building.</td>
</tr>
</tbody>
</table>

*In a building with out-of-plane offsets in vertical elements,*

- **a)** specialist literature shall be referred for design of such a building, if the building is located in Seismic Zone II: and
- **b)** the following two conditions shall be satisfied, if the building is located in Seismic Zones III, IV and V:
  1. Lateral drift shall be less than 0.2 percent in the storey having the offset and in the storeys below; and
  2. Specialist literature shall be referred for removing the irregularity arising due to out-of-plane offsets in vertical elements.

1. **Vertical and horizontal elements supporting discontinuous walls/frames shall be designed for load effects as per special load combinations in 6.3.5.**
2. **For buildings up to a height of 50 m, the diaphragm elements shall be designed for 25% increase in diaphragm forces calculated as per relevant clauses.**
3. **In addition to the above considerations, three-dimensional dynamic analysis method shall be used for buildings with height more than 50 m.**

#### COMMENTARY

An out-of-plane offset of the lateral load carrying vertical element (such as structural wall or column) imposes excessive demands on vertical elements and lateral loads on horizontal elements supporting such elements. This increase in the seismic load demands is due to the discontinuity in the load transfer path because of (in-plane and out-of-plane) offsets of the vertical elements in the building.

This is the most critical of plan discontinuity which significantly undermines the energy dissipation capacity of the structure and likely to cause excessive damage and even collapse. Therefore, it is important that horizontal as well as vertical elements in the load path transferring forces form discontinuous systems are designed for near elastic demands.

#### v) Non-parallel Lateral Force Systems

Buildings undergo complex earthquake behaviour and hence damage, when they do not have lateral force resisting systems oriented along two plan directions that are orthogonal to each other. A building is said to have non-parallel system when the vertically oriented structural systems

These systems are also known as non-orthogonal systems and both direct and orthogonal components of horizontal motion should be considered acting simultaneously as per the “100% + 30% rule.”
<table>
<thead>
<tr>
<th>CODE</th>
<th>COMMENTARY</th>
</tr>
</thead>
<tbody>
<tr>
<td>resisting lateral forces are not oriented along the two principal orthogonal axes in plan. Buildings with non-parallel lateral force resisting system shall be designed for load combinations mentioned in 6.3.2.2, 6.3.3.3 or 6.3.4.1.</td>
<td></td>
</tr>
</tbody>
</table>
CODE

COMMENTARY

34A TORSIONAL IRREGULARITY

34B RE-ENTRANT CORNERS
CODE

<table>
<thead>
<tr>
<th>OPENING LOCATED ANYWHERE IN THE SLAB</th>
<th>OPENING LOCATED ALONG ANY EDGE OF THE SLAB</th>
<th>VARIATION IN DIAPHRAGM CONFIGURATION</th>
</tr>
</thead>
</table>

**34C DIAPHRAGM DISCONTINUITY (FLOOR SLABS HAVING EXCESSIVE CUT-OUT AND OPENINGS)**

**34D OUT-OF-PLANE OFFSETS IN VERTICAL ELEMENTS**

**34E NON-PARALLEL LATERAL FORCE SYSTEM:**

(i) MOMENT FRAME BUILDING

(ii) MOMENT FRAME BUILDING WITH STRUCTURAL WALLS

**FIGURE 34 DEFINITIONS OF IRREGULAR BUILDINGS-PLAN IRREGULARITIES**
Table 6 – Definition of irregular buildings – Vertical irregularities (Figure 45)

(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 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 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.

Soft storey buildings are known for their poor performance during earthquakes. Typical examples for such irregularity are the buildings on stilts. In 2001 Bhuj earthquake, a majority of the multi-storey buildings that collapsed had soft ground storey.

Structures having stiffness, mass and geometric irregularities have the vertical distribution of lateral forces significantly different than that assumed in the seismic coefficient method. Therefore, the distribution shall be determined from the dynamic analysis.

Mass irregularity is induced by the presence of a heavy mass on a floor, for example, as in an intermediate service floor with water tanks and heavy equipment for air conditioning and/or back-up power generation.

The relaxation in case of roofs is warranted because the seismic weight of roof is usually much smaller than that of the typical floors. While checking the mass irregularity of such a building, the floor below the roof is likely to render the building irregular. This relaxation is not applicable particularly when large masses are added on the roof, for instance by the addition of
Proposed Modifications & Commentary IS:1893 (Part 1)

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% of adjacent storey below. 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. 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 Seismic Zones III, IV and V shall be designed such that safety of the building is not jeopardized; also, provisions of 7.10 shall be followed. Except for building up to two storeys, weak storeys shall not be permitted in zone III, IV and V.

NEHRP code and ASCE 7 also consider a building to be irregular even if a storey is 150 percent heavier than adjacent storeys.

Buildings with vertical offsets (e.g., set back buildings) fall in this category. There is also a possibility that a building may have no apparent offset, but its lateral load carrying elements may have irregularity. For instance, structural wall length may suddenly reduce. When building is such that a larger dimension is above the smaller dimension, it acts as an inverted pyramid and is particularly undesirable.

NEHRP code recommends a building to be irregular from vertical geometry considerations if the horizontal dimension of the lateral force resisting system in any storey is more than 130 percent of that in its adjacent storey.

The offset of 20% of the length of vertical element is too restrictive. It is restored to the 2002 definition with a caution that supporting elements will subjected to large demands due to overturning.
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.

This is a vertical irregularity in the force transfer by discontinuous columns. It should be treated as in-plane discontinuity in vertical elements which creates concentration of inelastic deformations at these locations and must be corrected, or strengthened, or its $R$ value should be decreased while calculation of design base shear to reflect its inelastic behaviour.

vii) Irregular Modes of Oscillation in Two Principal Plan Directions

Stiffnesses of beams, columns, braces and structural walls determine the lateral stiffness of a building in each principal plan direction. A building is said to have lateral storey irregularity in a principal plan direction if

- 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.

This is a case of torsional flexibility where even in a symmetric building, out of the first three eigen values, the lowest one may correspond to the torsional mode. It is understandable that when the first mode is torsional mode, the forces on the vertical elements near the perimeter will be amplified.

These are covered under torsional irregularity. It is undesirable that the first mode is the torsional mode as the seismic forces may be amplified as required in Table 5.
45A STIFFNESS IRREGULARITY

$K_i < 0.7 K_{ir+1}$

$K_i < 0.8 \left( \frac{K_{ir+1} + K_{ir+2} + K_{ir+3}}{3} \right)$

45B MASS IRREGULARITY

$W_i > 1.5 W_{i+1}$

$W_i > 1.5 W_{i-1}$
45\(^\circ\) C VERTICAL GEOMETRIC IRREGULARITY

- $A > 0.25L$
- $A > 0.125L$
- $A > 0.1L$
- $L_2 > 1.25L_1$
- $L_0 > 0.2L_w$
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

7.2.1 – Design Lateral Force
Buildings shall be designed for design lateral force \( V_B \) given by:

\[
V_B = A_h W
\]

where \( A_h \) shall be estimated as per 6.4.2, and \( W \) as per 7.4.

7.2.2 – Minimum Design Lateral Force
Buildings and portions thereof 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 \( \rho \) is defined in Table 7.

Table 7 Minimum Design Earthquake Horizontal Lateral Force for Building

<table>
<thead>
<tr>
<th>Sl. No.</th>
<th>Seismic Zone</th>
<th>( \rho ) percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>(1)</td>
<td>(2)</td>
<td>(3)</td>
</tr>
<tr>
<td>i)</td>
<td>II</td>
<td>0.708</td>
</tr>
<tr>
<td>ii)</td>
<td>III</td>
<td>1.113</td>
</tr>
<tr>
<td>iii)</td>
<td>IV</td>
<td>1.620</td>
</tr>
<tr>
<td>iv)</td>
<td>V</td>
<td>2.430</td>
</tr>
</tbody>
</table>

7.2.3 – Importance Factor (I)
In estimating design lateral force \( V_B \) of buildings as per 7.2.1, the importance factor \( I \) of buildings shall be taken as per Table 8.

Table 8 – Importance Factors, I

<table>
<thead>
<tr>
<th>Sl. No</th>
<th>Structure</th>
<th>( I )</th>
</tr>
</thead>
<tbody>
<tr>
<td>i)</td>
<td>Important services and community buildings or structure (for example, critical government buildings)</td>
<td>1.5</td>
</tr>
</tbody>
</table>

The minimum base shear (horizontal force) is based on satisfactory seismic performance of those buildings which had a lateral strength equal to about 3% weight of the structure. Such a level of lateral strength has been observed to provide adequate strength for long-period structures under far-field ground motions. Recent studies have also confirmed that such a level of minimum strength is necessary to develop the desirable yielding mechanism for energy dissipation at global level. [ATC 63, ASCE 7, FEMA 2009, FEMA P-2012, 2018]. Moreover, this minimum horizontal force is also related to structural integrity of the load path.
schools), signature buildings, monument buildings, lifeline and emergency buildings (for example hospital buildings, telephone exchange buildings. Television station buildings, radio station buildings, bus station buildings, metro rail buildings and metro rail station buildings), railways stations, airports, food storage buildings (such as warehouses, fuel station buildings, power station buildings, and fire station buildings), and large community hall buildings (for example, cinema halls, shopping malls, assembly halls and subway stations).

<table>
<thead>
<tr>
<th>ii)</th>
<th>Residential or commercial buildings [other than those listed in SI. No. (i)] with occupancy more than 200 persons</th>
<th>1.2</th>
</tr>
</thead>
<tbody>
<tr>
<td>ii)</td>
<td>All other buildings</td>
<td>1.0</td>
</tr>
</tbody>
</table>

NOTES:
1) Owners and design engineers of the buildings or structures may choose values of importance factor $I$ more than those mentioned above.

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

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

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

### 7.2.4 – Damping Ratio

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

### C7.2.4 –

It should be noted that the analysis model for RC structures must include effective cracked section properties as per 6.4.3.1 for the assumed 5% damping to be more reasonable.
7.2.5 – Design Acceleration Spectrum

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

C7.2.5 –

The code specifies same value of damping (5% of critical) for concrete, steel, or masonry buildings. It may be argued that steel as a material exhibits lower damping than masonry and therefore, different damping should be specified for three types of building materials. However, in the code, the damping has direct bearing on design seismic loads. Using a lower damping for steel buildings than for RC buildings will imply a higher value of seismic coefficient for steel buildings which cannot be justified in view of the relative performance of the RC and steel buildings in the past earthquakes. Moreover, partitions and other non-seismic members in steel building will still contribute the same amount of energy dissipation as in say RC building.

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.

C7.2.6 – Response Reduction Factor

The values of response reduction factor specified in Table 9 have been arrived at empirically based on engineering judgment.

Response reduction factors ($R$) were originally developed assuming that structures possess sufficient level of redundancy. Higher values of $R$ were justified by the large number of potential hinges that could form in such redundant systems, and the beneficial effects of progressive yield hinge formation. Buildings with special moment frames with relatively few bays supporting large floor and roof areas, and buildings with fewer walls results in lesser redundancy.

Reduced values of $R$ are proposed for less redundant structures and higher values are proposed for the structures with well distributed lateral-force resisting systems.

There are several issues that should be considered in quantifying redundancy. Conceptually, floor area, element/story shear ratios, element demand/capacity ratios, types of mechanisms which may form, individual characteristics of building systems and materials, building height, number of stories, irregularity, number of lines of resistance (load paths), and number of elements per line (load paths) are all important and will essentially influence the level of redundancy in systems and their reliability.
### Table 9 – Response Reduction Factor, $R$, for Building Systems

<table>
<thead>
<tr>
<th>SI No.</th>
<th>Lateral load resisting system</th>
<th>$R$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>i) Moment frame systems</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a)</td>
<td>RC buildings with Ordinary moment resisting frame (OMRF) <em>(see Note 1a)</em></td>
<td>3.0</td>
</tr>
<tr>
<td>b)</td>
<td>RC buildings with Intermediate moment resisting frame (IMRF) <em>(see Note 1b)</em></td>
<td>4.0</td>
</tr>
<tr>
<td>c)</td>
<td>RC buildings with special moment resisting frame (SMRF)</td>
<td>5.0</td>
</tr>
<tr>
<td>d)</td>
<td>Steel buildings with ordinary moment resisting frame (OMRF) <em>(see Note 1a)</em></td>
<td>3.0</td>
</tr>
<tr>
<td>e)</td>
<td>Steel buildings with special moment resisting frame (SMRF) <em>(see Note 1a)</em></td>
<td>5.0</td>
</tr>
<tr>
<td><strong>ii) Braced Frame Systems (see Note 2)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a)</td>
<td>Buildings with ordinary braced frame (OBF) having concentric braces</td>
<td>4.0</td>
</tr>
<tr>
<td>b)</td>
<td>Buildings with special braced frame (SBF) having concentric braces</td>
<td>4.5</td>
</tr>
<tr>
<td>c)</td>
<td>Buildings with special braced frame (SBF) having eccentric braces</td>
<td>5.0</td>
</tr>
<tr>
<td><strong>iii) Structural Wall systems (see Note 3)</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>a)</td>
<td>Load bearing masonry wall buildings</td>
<td></td>
</tr>
<tr>
<td>1)</td>
<td>Unreinforced masonry (designed as per IS 1905) without horizontal RC seismic bands <em>(see Note 1a)</em></td>
<td>1.5</td>
</tr>
<tr>
<td>2)</td>
<td>Unreinforced masonry (designed as per IS 1905) with horizontal RC seismic bands</td>
<td>2.0</td>
</tr>
<tr>
<td>3)</td>
<td>Unreinforced masonry (designed as per IS 1905) with horizontal RC seismic bands and vertical reinforcing bars at corners of rooms and jambs of openings <em>(with reinforcement as per IS 4326)</em></td>
<td>2.5</td>
</tr>
</tbody>
</table>
### Proposed Modifications & Commentary IS:1893 (Part 1)

#### 4) Reinforced masonry
[see SP 7 (PART 6) SECTION 4]

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td></td>
</tr>
</tbody>
</table>

#### 5) Confined masonry[see SP 7 (PART 6) SECTION 4]

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td></td>
</tr>
</tbody>
</table>

#### b) Buildings with ordinary RC structural walls (see Note 1a)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td></td>
</tr>
</tbody>
</table>

#### c) Buildings with intermediate RC structural walls (ISW) (see Note 1b)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5</td>
<td></td>
</tr>
</tbody>
</table>

#### c.d) Buildings with ductile RC special structural walls (SSW)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

### iv) Dual Systems

#### a) Buildings with Ordinary RC structural walls and RC OMRFs (see Note 1)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td></td>
</tr>
</tbody>
</table>

#### b) Buildings with Ordinary RC structural walls and RC SMRFs (see Note 1)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

#### c) Buildings with ductile RC special shear structural walls and RC OMRFs (see Note 1)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

#### d) Buildings with ductile RC special shear structural walls and RC SMRFs (see Note 1)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td></td>
</tr>
</tbody>
</table>

#### Buildings with moment frame systems (i) with structural walls systems (iii) (see Note 3)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5(R+R*)</td>
<td></td>
</tr>
</tbody>
</table>

The response reduction factor for dual systems shall be taken as the average value of the $R$ of its constituent lateral force resisting systems.

### v) Flat Slabs (Two-Way Slabs Without Beams)— Structural Wall systems (see Note 4.1c)

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>2.5</td>
<td></td>
</tr>
</tbody>
</table>

The lateral strength of flab slab relies on the transfer of moments between the slab and column which is generally not adequate due to low ductility of slab-column connections. Being relatively flexible system, large lateral deformations increase the potential for punching shear failure. The transfer of moments between the column and slab greatly reduces their punching shear resistance. So they should not be used in areas of high seismicity. However, they can be used as a secondary system for gravity loads along with a stiff lateral force resisting systems, such as braced frame or structural walls. In such cases, slab-column connections still have to be provided with adequate shear strength for the required deformation capacity in slab-column connection. Also integrity reinforcement at bottom of the slab extend beyond the slab-column interface which is required to prevent the dropping of slab panels and initiating
progressive collapse.

IS 13920 details provisions to design slab-column connections for the additional shear demand imposed by lateral drift of the structure.

An outrigger and belt truss system connecting the core ductile RC structural walls and the perimeter RC SMRFs is primarily a non-dual combination of two lateral load resisting framing systems, structural wall at the core and moment resisting frames at the perimeter to resist the seismic loads. And the outrigger beams/truss are used at several locations in elevation to control the lateral drift, typically in tall structures. The design of such a system will be governed by provisions for the combination of framing systems in a building.

Reference for seismic design of slab-column connections in a flat slab structure is given below:


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.

7.2.7 – Dual System

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

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

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

C 7.2.7

In dual systems, the moment frame shall resist at least 25% of total base shear so that it can serve as a secondary lateral force resisting system which will add to redundancy, ductility and energy dissipation capacity of the overall system. Both systems should act as a single unit and lateral load is shared in proportion to their relative lateral stiffness governed by force-deformation relation as per principles of mechanics. However, the moment frame should have sufficient strength to resist at least 25% of the total base shear.

Consider a combination of dual system consisting of
ordinary moment resisting frame (OMRF) with $R=3$ and special structural wall system (SSW) with $R=4$. Suppose the analysis indicates that the base shear $V_B$ for the dual system corresponding to $R$ value of 3.5, the SSW resists 85% of the $V_B$, while the remaining 15% is resisted by the OMRF. However, as per 7.2.7 for this system to be considered as a dual system, the OMRF will be designed for 25% of $V_B$, and the SSW will be proportioned for 85% of $V_B$. See Figure C17.

![Figure C17 – Design forces for dual systems](image)

If the above system is to be considered as non-dual system, then the structural wall (SSW) is the only lateral force resisting system which resists 100% of the $V_B$ determined with $R=4$.

### 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:

- **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.

- **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

### 7.2.8 Combination of Structural Systems

Different lateral force resisting systems can be combined in both plan as well as in vertical direction. For example, structural wall and moment resisting frames can be used along one of the principal axes and moment resisting frames along other principal axes as shown in Figure C18. Each system will be designed with respective $R$ values for its share of lateral loads in each direction.

The goal of the provisions For the vertical combination of framing systems is to prevent such mixed systems which may have concentration of large inelastic deformations in lower portions.
Proposed Modifications & Commentary IS:1893 (Part 1)

The 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:

- The stiffness of the lower portion shall be at least 10 times the stiffness of the upper portion.
- The upper flexible portion shall be designed as a separate structure using the appropriate value of \( R \).
- The lower portion shall be designed as a separate structure using an appropriate value of \( R \).
- Reactions of the upper portion shall be increased by the ratio of \( R \) values of the two portions and the factored reactions shall be applied at the top of the lower portion.

![Figure C18 – Combination of lateral load resisting systems (Moment Resisting Frame and Structural Wall)](image)

A simple two-stage analysis is proposed for a vertically mixed structural systems, in which the lower portion (say, basement storeys) are much stiffer (about 10 times) of the upper flexible portion (tower structure). The upper portion can be analyzed using equivalent static as well as dynamic method treating the bottom of the upper portion as the base while for the stiff lower portion the equivalent static method is a logical option for the dominance of the fundamental mode. The method is illustrated in Figure C19.

![Figure C19 – Analysis Methodology for Framing systems in vertical combination](image)

**7.3 Design Imposed Loads for Earthquake Force Calculation**

**7.3.1 –**

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

**C7.3 – Design Imposed Loads for Earthquake Force Calculation**

**C7.3.1 -**

This clause accounts for the fact that only a part of imposed loads used in design may be present at the time of earthquake shaking. Moreover, impact contribution of live load does not generate seismic load.
Table 10 – Percentage of Imposed Load to be Considered in Seismic Weight Calculation

(Clause 7.3.1)

<table>
<thead>
<tr>
<th>Imposed Uniformity Distributed Floor Loads (kN/m²)</th>
<th>Percentage of Imposed Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Up to and including 3.0</td>
<td>25</td>
</tr>
<tr>
<td>Above 3.0</td>
<td>50</td>
</tr>
</tbody>
</table>

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

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

a) Only that part of imposed load, which possesses mass, shall be considered; and

b) Lateral earthquake design force shall not be calculated on contribution of impact effects from imposed loads.

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

7.3.5 –
In regions of severe snow loads and sand storms exceeding intensity of 1.5 kN/m², 20
percent of uniform design snow load or sand load, respectively shall be included in the estimation of seismic weight. In case the minimum values of seismic weights corresponding to these load effects given in IS 875 are higher, the higher values shall be used.

7.3.6 –
In buildings that have interior partitions, the weight of these partitions on floors shall be included in the estimation of seismic weight; this value shall not be less than 0.5 kN/m². In case the minimum values of seismic weights corresponding to partitions given in parts of IS 875 are higher, the higher values shall be used. It shall be ensured that the weights of these partitions shall be considered only in estimating inertial effects of the building.

7.4 Seismic Weight

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

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

7.5 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.

C7.4 – Seismic Weight
It is the total dead weight of the structure plus that part of the imposed loads that may reasonably be expected to be attached to the structure at the time of earthquake shaking. It includes the weight of permanent and movable partitions, permanent equipment, and a part of live load as specified in 7.3.

C7.5– Diaphragm
Diaphragms serve a number of important roles in resisting lateral loads in addition to supporting gravity loads. Their in-plane stiffness determines how the lateral inertial forces developed in the plane of the diaphragm will be distributed among vertical elements of the seismic force resisting system. Further, the diaphragm plays a primary role in transferring these lateral inertial forces to the vertical elements of seismic force resisting elements. Diaphragms also provide lateral support to vertical elements and connect them to provide a three-dimensional load transfer path for seismic forces in the building. In cases of in-plane or out-of-plane discontinuity of vertical elements of seismic force resisting elements [Table 5(iv) and Table 6(iv)], diaphragms are required to transfer significantly...
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 to 1.5 times the average displacement of the entire diaphragm (see Figure 6).

![Fig. 6 Definition of Flexible Floor Diaphragm](image)

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.

C7.5.1 – Diaphragm Flexibility

Floor diaphragm plays an important role in seismic load distribution in a building. For horizontal loads, floor diaphragm (RC slab) acts as a deep beam with depth equal to building width, and width equal to slab thickness. Being a very deep beam, it does not deform in its own plane, and it forces the frames/walls to fulfill the deformation compatibility corresponding to in-plane deformation of floor. This is known as rigid floor diaphragm action.

In symmetrical building with symmetrical loading, the floor slabs undergo rigid body translation and different frames or walls share the seismic forces in proportion to their lateral stiffness.

When a building is not symmetrical, the floor undergoes rigid body translation and rotation.

In-plane rigidity of floors is sometimes misunderstood to mean that the beams are infinitely rigid and that the columns are not free to rotate at their ends. However, the rotation of columns is governed by the out-of-plane behaviour of slab and beam system (Figure C20).

When floor diaphragms do not exist, or when the diaphragm is extremely flexible as compared to the vertical elements, the loads can be distributed to the vertical elements in proportion to the tributary mass.

There are instances where the floor is not rigid. “Not rigid” does not mean it is completely flexible. Hence, buildings with flexible floors should be carefully analyzed considering in-plane floor flexibility. Clause 7.5.1 gives the criterion when the floor diaphragm is not to be treated as rigid (Figure C21). Alternatively, one can take the design force as an envelope of (that is, the higher of) the two extreme assumptions, mainly,

a) Rigid diaphragm action

b) No diaphragm action (Load distribution in proportion to tributary mass)

ASCE 7-16 specifies that a diaphragm shall be considered flexible if $\Delta_{\text{middle}} > 3 \Delta_{\text{ave}}$. The 2002 version of IS 1893 specified this limit to a conservative value of $\Delta_{\text{middle}} > 1.5 \Delta_{\text{ave}}$, considering the uncertainties in determining the diaphragm deflections. In 2016 edition the limit was further reduced to $\Delta_{\text{middle}} > 1.2 \Delta_{\text{ave}}$ which seems highly

large forces. Diaphragms are to be designed for moment and shear forces resulting from in-plane bending of the diaphragm, collector/drag forces as well as stress concentration around openings and re-entrant corners.
conservative. Therefore, it is proposed to bring back the 2002 definition.

![Diagram](image1)

**Figure C20**—(a) In plane floor deformation, (b) Out-of-plane floor deformation. (From Jain, 1995)

![Diagram](image2)

**Figure C21**—Definition of Flexible Floor

Diaphragm (From Jain, 1995)

### 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.

### C7.5.2– Diaphragm

Diaphragm design forces correspond to the peak response acceleration of floors and are obtained by multiplying it with tributary mass of the diaphragm.
as follows:

\[
F_{px} = \sum_{i=x}^{n} \frac{F_i}{w_{ix}} \sum_{i=x}^{n} w_i
\]

where,

- \(F_{px}\) = design diaphragm force at level \(x\)
- \(F_i\) = seismic design force at level \(i\)
- \(w_i\) = seismic weight at level \(i\)
- \(w_{ix}\) = seismic weight at level \(x\)

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

In case of plan offset of vertical resisting 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.

It should be noted that these forces may not necessarily be the same as design seismic forces for the vertical elements of the seismic force resisting systems, which are either obtained from Equivalent Static Method (7.6) or Dynamic Methods (7.7).

The diaphragm design requirements can be determined by analyzing one floor at a time for the corresponding diaphragm force.

As shown in Figure C22, the diaphragm in-plane moment due to lateral inertia forces is resisted by tension chord and compression chord while uniformly distributed shear is resisted along the depth of the diaphragm. Collector elements gather this shear and transmit to the vertical element. The collector element is subjected to axial tension and axial compression forces due to diaphragm shear being transferred through it.

Large diaphragm openings not only reduce in-plane stiffness of the diaphragm, but also significantly reduce the flexural and shear resistance of the diaphragm. One of the simple method to compensate for is to design additional chord reinforcements for the local moment and edge reinforcement to distribute the shear (Figure C23).

Figure C22—In-plane forces (moment and shear) in diaphragm and axial forces in collector elements

Figure C23—Additional design consideration for
local effects around openings in diaphragms

At the location of re-entrant corners, the cords do not extend to the full width of the building plan and chord forces have to follow the diaphragm profile round the re-entrant corner, giving rise to large stress concentration causing damage to diaphragm. If chord reinforcements are provided, they need to be extended to full development length into the diaphragm as shown in the Figure 23. Chord reinforcement in each portion of the diaphragm is designed for corresponding moment considering respective effective depth. In cases, where chord reinforcement is not necessary, the diagonal reinforcement can be provided to avoid cracking at reentrant corners.

Figure C24–Additional design consideration for re-entrant corners in diaphragms

Refer to Moehle et al. (2010) for more details about seismic behaviour and design of diaphragms.


7.6 Equivalent Static Method

As per this method, first, the design base shear $V_b$ shall be computed for the building as a whole. Then, this $V_b$ shall be distributed to the various floor levels at the corresponding centres of mass. And, finally, this design seismic force at each floor level shall be distributed to individual lateral load resisting elements through structural analysis considering the floor load.

C7.6 –

There have been instances of the designer calculating seismic design force for each 2D frame separately based on tributary mass shared by that frame. This is erroneous since only a fraction of the building mass is considered in such seismic load calculation (Figure C25).
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.

### 7.6.1

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

$$V_B = A_h W$$

where

- $A_h =$ Design horizontal acceleration coefficient value as per 6.4.2, using the approximate fundamental natural period $T_a$ as per 7.6.2 along the considered direction of shaking; and
- $W =$ Seismic weight of the building as per 7.4

### 7.6.2

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

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

  $$T_a = 0.075 h^{0.75} \text{ for RC frame buildings}$$
  $$T_a = 0.09 h^{0.75} \text{ for RC-steel Composite MRF buildings}$$
  $$T_a = 0.085 h^{0.75} \text{ for steel frame building}$$

  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:

\[
T_a = \frac{0.075h^{0.75}}{\sqrt{A_w}} \geq 0.09h/\sqrt{d}
\]

\[
T_a = \frac{0.00058h}{\sqrt{C_w}}
\]

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

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

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

where

- \(h\): height of building as defined in 7.6.2(a), in m;
- \(A_B\): area of base of structure, in m^2;
- \(A_{wi}\): effective cross-sectional area of structural wall \(i\) in first storey of building, in m^2;
- \(C_w\): 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.

(c) All other buildings:

\[
T_a = \frac{0.09h}{\sqrt{d}}
\]

Where

- \(h\): height of building, as defined in 7.6.2(a), in m; and
- \(d\): base dimension of the building at the

(b) The proposed equation rationally accounts for both shear behaviour of short walls and flexural behaviour of tall and slender walls and is currently preferred method in the US codes (Goel and Chopra, 1998, Chopra and Goel 2000). It is also lower bound to the observed natural period of real buildings (See Figure C29).


(c) As per experimental studies (ambient vibration surveys) on Indian RC buildings with masonry infills, \(T = 0.09h/(\sqrt{d})\) was found to give a good estimate. One may refer to the following:

plinth level along the considered
direction of earthquake shaking, in m.

2) Arlekar, J. N., and Murty, C. V. R., “Ambient
Vibration Survey of RC Moment Resisting Frame
Buildings with URM Infill Walls”, The Indian
Concrete Journal, Volume 74, No. 10, October 2000,
pp 581-586.

Figure C26 - Observations on steel frame buildings during San
Fernando Earthquake (From FEMA 369, 2001)

Figure C27 - Observations on RC frame buildings during San
Fernando Earthquake (From FEMA 369, 2001)
7.6.3

The design base shear \( (V_b) \) computed in 7.5.3 shall be distributed along the height of the building and in plan at each floor level as below:

a) Vertical distribution of base shear to

C7.6.3 – Distribution of Design Lateral Force

Vertical Distribution of Base Shear to
**Proposed Modifications & Commentary IS:1893 (Part 1)**

**Different Floor Levels**

The design base shear \( V_B \) computed in 7.6.1 shall be distributed along the height of the building as per the following expression

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

where

- \( Q_i \) = design lateral force at floor \( i \)
- \( W_i \) = seismic weight of floor \( i \)
- \( h_i \) = height of floor \( i \) measured from base; and
- \( n \) = number of storeys in building, that is, number of levels at which masses are located

**b) In-plan distribution of design lateral force at floor \( i \) to different lateral force resisting elements**

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

**Different Floor Levels**

Lateral load distribution with building height depends on the natural periods, mode shapes of the building, and shape of design spectrum. In low and medium rise buildings, fundamental period dominates the response and fundamental mode shape is close to a straight line (with regular distribution of mass and stiffness). For tall buildings, contribution of higher modes can be significant even though the first mode may still contribute the maximum response. Hence, NEHRP provides the following expression for vertical distribution of seismic load:

\[
Q_i = V_B \frac{W_i h_i^k}{\sum_{j=1}^{n} W_j h_j^k}
\]

Where,

- \( k=1 \) for \( T \leq 0.5 \) sec, and
- \( k=2 \) for \( T \geq 2.5 \) sec.

Value of \( k \) varies linearly for \( T \) in the range 0.5 sec to 2.5 sec.

Over the years, regardless of the natural period, \( k \) has been assigned a value 2 in IS 1893. This is a conservative value and has been retained in the current edition of the code too.
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 57).

C7.6.4 –

Integrated with the proposed clause 7.5 on diaphragm issues related to strength and flexibility.
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.

7.7 Dynamic Analysis Method

7.7.1 –
Linear dynamic analysis shall be performed to obtain the design lateral force (design seismic base shear, and its distribution to different levels along the height of the building, and to 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 50 m and $T_a$ greater than 1.5 s 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 1.5 s 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.

C7.7 – Dynamic Analysis Method

C7.7.1 –
Expressions for design load calculation and load distribution with height given in 7.6 are based on the following assumptions:

1. Fundamental mode dominates the response.
2. Mass and stiffness are evenly distributed with building height, thus giving a regular mode shape.

Mode shapes depend on the distribution of mass and stiffness in the building. In tall buildings, higher modes can be quite significant and in irregular buildings mode shapes may be somewhat irregular. Hence, for tall and irregular buildings, dynamic analysis is generally preferred. Industrial buildings may also require dynamic analysis because they may have large spans, large heights, and considerable irregularities. However, dynamic analysis may not necessarily be a solution to many irregular buildings, and it requires a good judgment on the part of the engineer to decide if dynamic analysis is warranted.

Buildings having high level of torsion irregularity are prone to severe damage when subjected to seismic forces. Therefore, in this revision of the code such buildings are restricted in zones of high seismicity (zones IV and V).

Dynamic analysis requires considerable skills. The
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 \( V_a \) estimated shall not be less than design base shear \( V_a \) calculated using a fundamental period \( T_a \), where \( T_a \) is as per 7.6.2.

When \( V_a \) is less than \( V_a \), the force response quantities (for example member stress resultants, storey shear forces, and base reactions) shall be multiplied by \( \frac{V_a}{V_a} \). For earthquake shaking considered along:

a) The two mutually perpendicular plan directions X and Y, separate multiplying factors shall be calculated.

C7.7.3 –

This clause requires that when dynamic analysis gives lower design forces, these should be scaled up to the level of forces obtained based on empirical \( T \). This implies that empirical \( T \) may be more reliable than \( T \) computed by dynamic analysis, which indeed is the intention. Dynamic analysis based on questionable assumptions may give an unduly large natural period, and hence, a much lower design seismic force. This clause intends to be a safeguard and is in line with the international practices on this issue.

There are considerable uncertainties in modeling a building for dynamic analysis, such as:

- Stiffness contribution of non-structural elements;
- Stiffness contribution of masonry infills;
- Modulus of elasticity of concrete, masonry, and soil; and
- Moment of inertia of RC members.

The mere fact that the computer program can perform dynamic analysis is not sufficient. The engineers need to have an in-depth understanding of the subject to be able to correctly model the structure and correctly interpret the results. There are approximate methods such as Rayleigh’s method and Dunkerley’s method, that one may use to check if the results obtained from computer analyses are correct.

One must be careful about use of correct units while performing dynamic analysis since it is common that huge errors occur just because units of mass and weight are mixed up. For details, the following text books are recommended:

7.7.4 – Time Response History

**Method**

The 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 $Z/2$ 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

Thus, there can be large variation in natural period, depending on how one models a building. For instance, ignoring the stiffness contribution of infill walls itself can result in a natural period several times higher.

As per NEHRP Commentary [FEMA 369, 2001]:

“If one ignores the contribution of nonstructural elements to the stiffness of the structure, the calculated period is lengthened, leading to a decrease in the design force. Nonstructural elements do not know that they are nonstructural. They participate in the behaviour of the structure even though the designer may not rely on them for contributing any strength or stiffness to the structure. To ignore them in calculating the period is to err on the unconservative side.”

Even when the results of dynamic analysis are scaled up to design force based on empirical $T$, the load distribution with building height and to different elements is still based on the results of the dynamic analysis, and therein, lies the advantage of dynamic analysis.

The scaling of results has to be performed separately for each principal direction.

C7.7.4 – Response History Method

Ground acceleration time histories are required to conduct the time history method of analysis. For this, ground motions recorded under similar site conditions in the past earthquakes may be used. Specialist literature may be referred to for help in identifying the appropriate ground motions. Alternately, synthetically generated ground motions may be used. Such ground motions should be compatible with the spectrum given in this standard or with the site-specific spectrum, whichever is applicable. Typically such ground motions should have duration of strong shaking no less than 15 s or 5 times of the fundamental period of the structure.

The scaling process for horizontal ground motions to match the code prescribed design spectrum can be summarized as follows (adapted from FEMA P751, 2009):

a) **Step 1**: Compute the 5% damped pseudo-acceleration spectrum for each unscaled component of each pair of ground motions in the set and produce the SRSS spectrum for each pair of motions within the set.

b) **Step 2**: Scale each SRSS spectrum such that the spectral ordinates of the scaled spectrum at $T_{avg}$ is equal to the spectral ordinates of the design spectrum at the same period. $T_{avg}$ is the average of fundamental natural periods in two principal
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 0.2T to 1.5T, 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 0.2T to 1.5T, where T is the fundamental natural period of the structure.

e. All response parameters shall be multiplied by \( \frac{1}{R} \). 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 used 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 $k \in \{1, N_m\}$ that need to be considered as per 7.7.5.2.

7.7.5.2 – Number of modes to be considered

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

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

C7.7.5.2 – Modes to be considered

In a multi-degree of freedom system, when the ground shakes in a particular direction, only a part of the total seismic mass of the whole structure vibrates in each mode of vibration. Thus, the net mass accounted for in the modes of vibration considered may be less than the total seismic mass of the structure. The difference between the total seismic mass of the structure and the net masses accounted for in the modes considered is called the missing mass. Often, this missing mass corresponds to the modes of vibration whose natural periods are very small (or whose natural frequencies are very large). Thus, in the missing mass correction procedure, it is assumed that the missing mass corresponds to modes of vibration that have natural periods close to zero. The corresponding Response Acceleration Coefficient ($S_a/g$) from Figure 2 of this standard is 1.0. Thus, the Design Horizontal Seismic Coefficient $A_h$ corresponding to the missing mass becomes $2I/2R$.

In the multi-degree of freedom system under consideration, the missing mass will be distributed throughout the structure. The Design Horizontal Seismic Coefficient $A_h$ corresponding to the missing mass is multiplied with these missing masses at different locations, and the equivalent static forces for the missing masses are obtained. These forces are applied on the structure and another static analysis is conducted. The results of this static analysis are combined with those of the modes considered, as per 7.7.5.3.

7.7.5.3 – Combination of modes

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

a) The peak response quantities (for example, member of forces, displacements, storey forces, storey shears, and base reactions) shall be combined as per Complete Quadratic Combination (CQC) method.

C7.7.5.3 – Modal Combination

This clause gives the complete quadratic combination (CQC) method first and then simpler method as an alternative. CQC method is applicable both when the modes are well separated and when the modes are closely spaced. Many computer programs have CQC method built-in for modal combination. For details, the following textbook may be referred to:

Proposed Modifications & Commentary IS:1893 (Part 1)

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

where

\(\lambda\) = estimate of peak response quantity;
\(\lambda_i\) = response quantity in mode \(i\) (with sign);
\(\lambda_j\) = response quantity in mode \(j\) (with sign);
\(\rho_{ij}\) = cross-modal correlation coefficient

\[
= \frac{8\zeta (1 + \beta) \beta^3}{(1 - \beta^2)^2 + 4\zeta^2 \beta (1 + \beta^2)}
\]

\(N_m\) = number of modes considered;
\(\zeta\) = modal damping coefficient ratio which shall be taken as 0.05;
\(\beta\) = natural frequency ratio = \(\frac{\omega_j}{\omega_i}\);
\(\omega_j\) = circular natural frequency in mode \(j\); and
\(\omega_i\) = circular natural frequency in mode \(i\).

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

1) If the building does not have closely-spaced modes, then the peak response quantity \(\lambda\) due to all modes considered shall be obtained as

\[
\lambda = \sqrt{\left( \sum_{k=1}^{N_m} \lambda_k \right)^2}
\]

where
\(\lambda_k\) = peak response quantity in mode \(k\), and
\(N_m\) = number of modes considered;

2) If the building has a few closely-spaced modes, then the peak response quantity \(\lambda^*\) due to these closely spaced modes alone shall be obtained as

\[
\lambda^* = \sum \left| \lambda_c \right|
\]

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

7.7.5.4 – Simplified method of dynamic analysis of buildings

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

a) Modal Mass - The modal mass \( M_k \) of mode \( k \) is given by

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

where

\( g \) = Acceleration due to gravity,

\( \phi_{ik} \) = Mode shape coefficient at floor \( i \) in mode \( k \), and

\( W_i \) = Seismic weight of floor \( i \).

\( n \) = number of floors of the structure.

b) Modal Participation Factors - The mode participation factor \( P_k \) of mode \( k \) is given by:

\[
P_k = \frac{\sum_{i=1}^{n} W_i \phi_{ik}^2}{\sum_{i=1}^{n} W_i \phi_{ik}^2}
\]

c) Design Lateral Force at Each Floor in Each Mode - The peak lateral force \( Q_{ik} \) at floor \( i \) in mode \( k \) is given by

\[
Q_{ik} = A_k \phi_{ik} P_k W_i
\]

where

\( A_k \) = Design horizontal acceleration spectrum value as per 6.4.2 using the natural period of oscillation \( T_k \) of mode \( k \) obtained from dynamic analysis.

d) Storey Shear Forces in Each Mode - The peak shear force \( V_{ik} \) acting in storey \( i \) in mode \( k \) is given by

Figure C30 – Lumped mass model

The analysis procedure is valid when a building can be modeled as a lumped mass model with one degree of freedom per floor (Figure C30).

This method of analysis does not imply that (a) the structure deforms only in the shear mode with no rotations or vertical translations at the floor levels, and (b) the beams in the structure are flexurally rigid and hence undergo no rotations.
\[ V_k = \sum_{i=t+1}^{n} Q_{ik} \]

e) **Storey Shear Force due to All Modes Considered** - The peak storey shear force \((V_i)\) in storey \(i\) due to all modes considered, shall be obtained by combining those due to each mode in accordance with 7.7.5.3

f) **Lateral Forces at Each Storey Due to All Modes Considered** - The design lateral forces, \(F_{\text{roof}}\) at roof level and \(F_i\) at level of floor \(i\) shall be obtained as:

\[
F_{\text{roof}} = V_{\text{roof}}, \text{ and } \quad F_i = V_i - V_{i+1}
\]

### 7.8 Torsion

**7.8.1** –

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

\[
e_{di} = \begin{cases} 1.5e_{si} + 0.05b_i & \text{if } e_{si} \geq e_{si} - 0.05b_i \\ e_{si} - 0.05b_i & \text{otherwise} \end{cases}
\]

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

where

- \(e_{si}\) = Static eccentricity at floor \(i\)
- \(b_i\) = Floor plan dimension of floor \(i\), perpendicular to the direction of force.

**C7.8.2 – Design Eccentricity**

Under dynamic conditions, the effect of eccentricity is higher than that under static load. Hence, a dynamic amplification is often applied to static eccentricity for computing design eccentricity. For instance, 1984 version of the code provided an amplification of 1.5 to the computed eccentricity (clause 4.2.4 of IS 1893 - 1984).

Additionally, an accidental eccentricity is also considered because (a) the computation of eccentricity is approximate, (b) during the service life of the building, there could be changes in its use that may relocate the center of mass, and (c) ground motion itself may have some torsional components (Figure C31).
The factor 1.5 represents dynamic amplification factor, and 0.05\(b_i\) 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.

![Figure C31 – Two possible cases of maximum eccentricity](image)

7.9 RC Frame Buildings with Unreinforced Masonry Infill Walls

7.9.1 –
In RC buildings with moment resisting frames and unreinforced masonry (URM) infill walls, variation of storey stiffness and storey strength shall be examined along the height of the building considering in-plane stiffness and strength of URM infill walls. If storey stiffness and strength variations along the height of the building render it to be irregular as per Tables 5 and 6, the irregularity shall be corrected especially in Seismic Zones III, IV and V.

C7.9 – RC Frame Buildings with Masonry Infills

Masonry infills possess significant in-plane stiffness and strength, and hence contribute to the overall stiffness and strength of the building. Stiffness of uncracked masonry is significant and can influence dynamic characteristics and seismic design forces and therefore, should be always considered for the not so slender masonry walls. However, their contribution to strength is not reliable as their resistance drops rather abruptly soon after cracking.

Masonry infilled frame building behaves as a shear wall structure in the elastic range. However, once cracks form along the boundary between the infill and the RC frame, the response is similar to that of a braced frame, with the infill acting as a compression diagonal element, which is responsible for the concentration of high shear stresses in the columns near joint region. Thus, introduction of masonry infills in RC frames changes the lateral load transfer mechanism of the structure from predominant frame action to predominant truss action. Such distribution of lateral loads to different members is responsible for tremendous reduction in bending moments and increase in axial forces in the frame members.

Also, infills can cause irregularities in the building, e.g., short column effect. This should be recognized at the design stage itself and given due consideration. The effect of the infills is lesser if openings are present. However, these infills pose the hazard of out-of-plane collapse. Hence, it is best to avoid situations that lead to infill panels of large width or height.


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

C7.9.2.1–
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:

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

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

where

$f_b =$ compressive strength of brick, in MPa; and

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

C7.9.2.1—

A number of empirical relationships are available established in the literature for the modulus of elasticity of brick masonry. However, it is very difficult to define the modulus of elasticity of masonry precisely.

Large variation has been reported in the relationship between elastic modulus and compressive strength of masonry, $f_m$. For the purpose of this code, therefore, Drysdale’s (1993) expression $E_m = k f_m$ was used with $k$ taken as 550. Studies conducted at IIT Kanpur showed that this value agrees with experimental data reasonably well [Kaushik et al., 2007]. The predictive relation for $f_m$ using the compressive strengths of brick units and mortar is also taken from the same study.

The experimental data for $E_m$ and $f_m$ for other units, such as fly-ash and AAC is quite limited. These results indicate somewhat larger proportionality factor, $k$ for fly-ash units (600-700) and AAC (750-900) [Basha & Kaushik 2015, Bose and Rai 2016, Bhosale at al, 2019]. A conservative value of 550 is appropriate until more data become available.


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. 78) shall be taken as

\[
\frac{w_{ds}}{L_{ds}} = 0.25L_{ds} \\
\frac{w_{ds}}{w_{ds}} = 0.17\epsilon_p^{0.4}L_{ds}
\]

where

\( L_{ds} \) is length of diagonal strut

\( \epsilon_p = h \left( \frac{E_f \sin 2\theta}{4E_m I I h} \right) \)

Where \( E_m \) and \( E_f \) are the moduli of elasticity of the materials of the URM infill and RC, \( I \) the moment of inertia of the adjoining column, \( t \) the thickness of the infill wall, and \( \theta \) 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 \( h/t < 12.30 \) and \( l/t < 12.30 \), 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) The advantages of strength contributed by the infill shall not to be considered when the height of the building is more than 12m.

e) All the RC frames shall be designed to support the vertical gravity loads, 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.

C7.9.2.2 –

While a number of finite element models have been developed and used to predict the response of masonry infilled frames, they are generally too cumbersome and time-consuming to be used in analyzing real-life infilled frame structures in design offices. Therefore, a much simplified yet reasonably accurate macro-model is needed that considers various factors that govern the behaviour of infilled frames. This is usually done by modeling the infill panel as a single diagonal strut connected to the two compressive diagonal corners, as shown in Figure C32.

Figure C32 – Equivalent diagonal strut model

The key to the equivalent diagonal strut approach lies in determination of effective width of the equivalent diagonal strut. In the last few decades, several attempts have been made to estimate the effective width of such equivalent diagonal struts. The expression for effective width adopted in this code is as per the Mainstone (1971) which is a lower bound various proposals of strut width estimation and therefore seriously underestimates the stiffness of the infill and is therefore unconservative. Relations by Holmes (1961) and Stafford-Smith provide upper bound, while Paulay and Priestley’s (1992) proposal of strut width equal to 0.25 times the diagonal strut length lies in the middle. The New Zealand code prescribes the strut width as 0.25 \( L_{ds} \) same as Paulay and Priestley's proposal.

Infilled frames with openings shall be modeled with reduced width of strut, which is given as:

\[ w_{do} = \rho_w w_{ds} \]

where \( w_{ds} \) is the width of diagonal strut for infill walls without openings and \( \rho_w \) is a reduction factor, which accounts for openings in infill, which is given by

\[ \rho_w = 1 - 2.5A_t \]
$A_r$ is the opening area ratio, which is the ratio of face area of opening to the face area of infill. If the opening area ratio is less than 0.05, i.e., the area of opening is less than 5% of the area of the infill panel, no reduction in the width of diagonal strut need to be made and the infill panel can be modeled as a solid panel. Whereas, if the opening area ratio is more than 0.4, i.e., the area of opening exceeds 40% of the area of the infill panel, the strut reduction factor shall be set to zero and the effect of infill shall be ignored in that panel.

The effect of opening in the infill wall is to reduce the lateral stiffness and strength of the frame. This can be represented by a diagonal strut of reduced width. The reduction factor $\rho_w$ is defined as ratio of reduced strut width to strut-width corresponding to fully infilled frame.

The slenderness limit of 12 is too restrictive, even slender walls can add significant in-plane stiffness. However, such walls can be made stable under out-of-plane seismic forces by sub-paneling, etc. A slenderness ratio of 30 is permitted by IS 1905 for panel walls even when the most restrictive of slenderness limits is considered.

The combined behavior of masonry-infilled RC frames is such that the total seismic force is resisted in proportion to the lateral stiffnesses of the RC frame and masonry infills at all storey levels. Masonry infills which are normally very stiff initially, attract most of the lateral forces, but may fail abruptly because of the brittle behavior. In such cases, RC frames must have sufficient backup strength to avoid collapse of the structure. Eurocode 8 requires the RC frames to resist full vertical loads and at least 50 to 65% of the total lateral loads on buildings. According to most codes, masonry infills are not expected to carry any gravity loads other than its self-weight.

In summary, the contribution of masonry infills in resisting the lateral loads can be substantial. However, to safeguard against RC frame being designed for a very low seismic forces, the frame alone is required to be designed to independently resist at least 50% of the design seismic forces in addition to the forces due to vertical loads.


Proposed Modifications & Commentary IS:1893 (Part 1)

473-478.


---

## 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)

---

## C7.10 – Buildings with Open storeys

### C7.10.1

Generally, RC frame buildings with open storeys is a soft and weak storey building. Soft/weak storey buildings are well known for their poor performance during earthquakes. During the Bhuj earthquake of

---

Fig. 78 – Equivalent Diagonal Strut of URM infill wall
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.

In 2001, most of the multi-storey buildings that collapsed had soft ground storey. Figure C33 indicates the severe deformation demands in case of a building with a soft storey.

In addition, such buildings also suffer from vertical and out-of-plane discontinuity of load path. Significant torsional irregularity is also probable due to unequal distribution of the infills in the upper storeys.

Figure C33 – Soft-storey created by open ground storey with infill in upper storeys (b) is subjected to severe deformation demands during seismic shaking compared to the regular moment frame (a) (From Murty et al, 2002).

For the vertical irregularity of masonry-infilled frames in the open ground storey, EC8/1 requires that the strength of vertical elements of the open story should be increased to make up for the deficit in shear strength due to the absence of infills. It is achieved by multiplying seismic design forces by a factor

\[ \alpha = 1 + \Delta R_{inf} / V_F \]

where \( \Delta R_{inf} \) is deficit in shear strength due to the absence of infills and \( V_F \) is the design seismic shear of the open story. The factor can assume values as high as 2.8, resulting in over reinforcing of columns which have adverse effects on ductility. So, it is advisable to increase both, strength and stiffness of column by increasing reinforcement and size of column respectively.

Kaushik et al (2009) have also shown that such a method for amplifying the design forces of only open ground storey columns (not beams), significantly alleviate the problem by reducing inelastic deformation demands on these columns. However, the problem of discontinuity of lateral load path may be further lessened by provision of additional elements in the form of structural walls, braces, buttresses, etc. Also, addition of such structural elements brings the overall lateral load behaviour of such buildings to acceptable levels (Kaushik et al. 2009).

Moreover, the commentary caution that the proposed solution may not completely fix the problem of
Since calculating infill strength is tedious, a less demanding approach was proposed in which the open story columns are designed on the basis of capacity design moments at the joints. That is, at a joint

\[ M_{co} + M_{bo} \geq 1.4M_{co} + M_{cinf} \]

where, \( M_{co} \) and \( M_{bo} \) are moment capacities of columns and beams, respectively, in the open storey, and \( M_{cinf} \) is the moment capacity of the column in the upper storey with infills. (Fardis 2000).

The approach suggested is similar to correct the major deficiencies of discontinuity irregularity of vertical elements of masonry infills in which the structural members (i.e., columns) supporting discontinuous infill walls need to be designed for the amplified seismic actions, i.e., 2.5 times of that required by usual load combinations involving seismic load. It is further suggested that provisions of additional members (infill, columns, bracings, etc.) in open storey will help to achieve near elastic behavior under design level seismic demands.


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 discontinuity of lateral load path. However, enhancing open story seismic column design moments by 2.5R times, significantly alleviate the problem by reducing inelastic deformation demands on these columns (Kaushik et al. 2004). Together with bracings, shear walls, battered columns may further improve the seismic performance (Kaushik et al. 2004).
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 $\rho_{sw}$ 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
Deformation of RC buildings shall be obtained from structural analysis using a structural model based on section properties given in 6.4.3.1.

C7.11 – Deformation

For good seismic performance, a building needs to have adequate lateral stiffness. Low lateral stiffness leads to:

- Large deformations and strains, and hence more damage in the event of strong ground shaking.
- Significant P-Δ effect.
- Damage to non-structural elements due to large deformations.
- Discomfort to the occupants during vibrations.
- Large deformations may lead to pounding with adjacent structures.

Stiff structures, though they attract more seismic loads, have generally performed better during past earthquakes.

The actual displacement in a strong shaking may be much larger than the displacement calculated for design loads because design seismic forces are reduced forces. As a rule of thumb, the maximum displacement during the MCE shaking (for example, PGA of 0.36g in zone V) should be about 2R times the computed displacement due to unfactored design seismic forces.

The higher the stiffness, lower the drift but higher the lateral loads. Hence, for computation of T for seismic design load assessment, all sources of stiffness even if unreliable should be included. And for computation of drift, all sources of flexibility even if unreliable should be incorporated.

Thus, in computation of drift the stiffness contribution of non-structural elements and non-seismic elements (i.e., elements not designed to share the seismic loads) should not be included. This is because such elements cannot be relied upon to provide lateral stiffness at large displacements. All possible sources of flexibility should be incorporated, for example, effect of joint rotation, bending and axial deformations of columns and structural walls, etc.
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.

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

7.11.2 – Deformation Capability of Non-Seismic Members

For buildings located in Seismic Zones III, IV and V, it shall be ensured that structural components, that are not a part of seismic force resisting system in considered direction of ground motion but are monolithically connected, do not lose their vertical load-carrying capacity 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 $R \Delta$ times storey displacements calculated as per 7.11.1, where $R$ is specified in Table 9.

C7.11.1 – Storey Drift Limitation

Clause 7.8.2 requires scaling up of seismic design forces from dynamic analysis, in case these are lower than those from empirical $T$. The second paragraph allows drift check to be performed as per the dynamic analysis, which may have given lower seismic forces, i.e., there is no need for scaling up of forces for the purpose of drift check. This is because in the displacement calculation even though lower forces are used, the stiffness of the structure modeled is also lower.

The storey drift limitation applies to location of vertical lateral force resisting elements in the plan of the storey.

C7.11.2 – Deformation Capability of Non-Seismic Members

This clause is particularly important when not all structural elements are expected to participate in lateral load resistance. For example, flat-plate buildings or buildings with pre-fabricated elements where seismic load is resisted by structural walls, and columns carry only gravity loads. During the 1994 Northridge Earthquake (California) many buildings collapsed due to failure of gravity columns.

During shaking, gravity columns do not carry much lateral loads, but deform laterally with the structural walls due to compatibility imposed by floor diaphragm (Figure C34). Moments and shears induced in gravity columns due to the lateral deformations may cause collapse if adequate provisions are not made. ACI 318 and IS 13920 has a separate section on detailing of gravity frames to safeguard against this kind of collapse.

Since deflections are calculated using design seismic force (which is a reduced force), the values of deflection are to be multiplied by $R$. The use of multiplier $R$ could be debated since it will only ensure safety against design basis earthquake. For safety against maximum considered earthquake, multiplier $2R$ should be used.
7.11.3 Separation between Adjacent Units

In order to avoid damage due to pounding between two adjacent buildings, or two adjacent units of the same building with a separation joint between them, shall be separated by a distance equal to the square root of the sum of squares of the amount $R$ times the sum of inelastic storey displacements, $R_1$ times $\Delta_1$ and $R_2$ times $\Delta_2$ calculated as per 7.11.1. That is,

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

where $R_1$ and $\Delta_1$ correspond to building 1, and $R_2$ and $\Delta_2$ to building 2.

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 $(R_1\Delta_1 + R_2\Delta_2)/2$, where $R_1$ and $\Delta_1$ correspond to building 1, and $R_2$ and $\Delta_2$ to building 2.

C7.11.3 – Separation between Adjacent Units

During seismic shaking, two adjacent units of the same building or two adjacent buildings may hit each other due to lateral displacements. This is known as pounding or hammering (Figure C35). This clause is meant to safeguard against pounding. As explained earlier multiplier $R$ is used since the deflection is calculated using design seismic forces, which are reduced forces. The equal displacement principle says that inelastic displacement will be $R$ times the elastic displacement. The absolute sum of story displacements is rather conservative and the SRSS value is justified as it is unlikely that two buildings would move in exactly opposite directions with both reaching their maximum displacements concurrently.

Further, it has been observed that the contact between two adjacent slabs at the same elevation are usually less damaging. However, no reduction in separation distance for such buildings is permitted as the intent of the code is to avoid any pounding damage. These provisions are in line with recent research findings and other international codes.

Structural elements adjacent to separation joints should be strengthened to safeguard against any adverse effect of pounding, such as reduction in member strength capacity, etc.
Figure C35 – Pounding of adjacent buildings
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 $N_1/N_s < 10$) 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 $Z_f$ times the seismic weight of the structure five times the design horizontal seismic coefficient $A_h$ specified in 6.4.2 for that building. In the analysis of the building, the

---

C7.12 -- Miscellaneous

C7.12.1 – Foundations

Clause 7.12.1 has been introduced to prevent the use of foundation types vulnerable to differential settlement. One may note that the note 7 in table 1 of the 2002 edition of the code has been omitted there and introduced here.

In 2002 edition of the code, ties were supposed to be designed for an axial load (in tension and compression) equal to $A_h/4$ times the larger of the column or pile cap load. This was fairly empirical, and the specification appeared to be on the lower side. Many structural engineers design the ties for 5% of the larger of the column or pile cap load. This specification, therefore, has been changed.

Tie beams may be provided either at the footing level or at the plinth level in case the difference between footing and plinth levels is not substantial.

It is required that inelastic activities (e.g. moment plastic hinges) at the base of vertical elements of seismic force resisting elements does not spread to foundations. Therefore, foundations need to be designed for forces 25% larger than the design resistance of vertical elements supported by them.

C7.12.2 – Cantilever Projections

All projections (vertical and horizontal) are highly vulnerable to damage during earthquakes. Being cantilevers, there is no redundancy and hence, the projections are designed for larger forces.

The vertical projections are treated as non-building elements and the required design seismic force was estimated considering them as rigid element with nearly elastic response for an amplified floor acceleration equal to two times of the horizontal
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.

\[
2 \times \frac{Z}{2} \times \frac{I}{1.0} \times 2.5 = 2.5 ZI
\]

**C 7.12.2.2**

For horizontal projections, the design seismic force is related to vertical component of the ground motion. Considering the rigid and nearly elastic response, the seismic force is calculated as follows:

\[
\frac{2}{3} \times \frac{Z}{2} \times \frac{I}{1.0} \times \frac{9}{3} = 0.9 ZI
\]

### 7.12.2.3 –

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

### 7.12.3– Compound Walls

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

### 7.12.4– Connections between Parts

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

### 7.12.5– Temporary Structures

Temporary structures such as scaffolding, shelters, tents, temporary excavations and other facilities during the construction of temporary structures generally have a reduced life of 1 to 5 years which is much shorter than a permanent structure. Designing such structures for same loads as the permanent structure will be
structures are meant for a limited-time use. For such structures, the $A_h$ in section 6.4.2 shall be reduced by 50%.

**C7.12.6**

Larger drifts are permissible for open parking structures because for such structures, non-structural damages occurring due to large drifts are acceptable without jeopardizing life safety.

---

**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**

This is a new section proposed for the non-structural elements. However, the provisions in this section are not underlined.

---

**7.13.1–General**

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.

---

**C 7.13.1.1-**

In several past earthquakes, it is seen that failure of nonstructural elements posed safety risk to building occupants, and critically impaired the performance of the buildings as well, for example, failure of nonstructural elements of fire and police stations, power stations, communication facilities and water supply facilities. Moreover, in most of the buildings, non-structural elements represent a high percentage of the total cost of the buildings. Therefore, nowadays it is widely recognized that good performance of nonstructural elements during earthquakes is extremely important.

Some important references on seismic performance and design of non-structural elements are:

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.

C 7.13.1.2–

When the nonstructural element significantly affects structural response of the building, the nonstructural component should be treated as structural, and the relevant structural provisions should apply. For example, in general, a masonry infill wall should be considered as structural for in-plane response, and therefore, it is within the scope of clause 7.9.

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.

C 7.13.1.4–

Particular care should be taken to identify masonry infill that could reduce the effective
**CODE**

length of adjoining columns.

**COMMENTARY**

effective length of the column, and seriously affect the building response.

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.

C 7.13.2.1–

Nonstructural components are regarded as acceleration sensitive when they are mainly affected by acceleration of the supporting structure. In such a case, structural-nonstructural interaction due to deformation of the supporting structure is not significant. Acceleration sensitive nonstructural components are vulnerable to sliding, overturning, or tilting. Mechanical and electrical components are generally acceleration sensitive.

7.13.2.2–

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

C 7.13.2.2–

Nonstructural components are regarded as deformation sensitive when they are affected by supporting structure’s deformation, especially the inter-storey drift. Good performance of deformation sensitive nonstructural elements can be ensured in two ways: (i) by limiting inter-storey drift of the supporting structure in case of important nonstructural elements, and (ii) by designing the element to accommodate the expected lateral displacement without damage.

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 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 23,

$x$ = 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.

C7.13.3.1–

The component amplification factor ($a_p$) represents the dynamic amplification of the component relative to the fundamental period of structure. In most situations, the non-structural element may need to be designed without fundamental period of the structure being available. Further, one may need to carry out experimental studies (e.g., shake table study) to evaluate fundamental period of the nonstructural element which may not be feasible.

The component response modification factor ($R_p$) represents ductility, redundancy, and energy dissipation capacity of the element and its attachment to the structures. Not much research is available on evaluation of these factors.

Hence, values of $a_p$ and $R_p$ (Tables 12 and 13) are taken same as in NEHRP provisions (FEMA 369, 2001); these empirically specified values are based on “collective wisdom and experience of the responsible committee”.

In choosing these values, it is expected that the component will behave as either flexible ($a_p = 2.5$) or rigid ($a_p = 1.0$) body. In general, values of $R_p$ are taken as 1.5, 2.5 and 3.5 for low, limited and high deformable structures, respectively.

Input acceleration at the point of attachment depends on peak ground acceleration, dynamic response of the building, and the location of the element along the height of the building. In this equation, the input acceleration at the point of attachment has been approximated as linearly varying from the acceleration at the ground ($0.5Z$) to the acceleration at the roof ($Z$).

A lower limit of $F_p$ is set to assure a minimal seismic design force.
## CODE

### COMMENTARY

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

<table>
<thead>
<tr>
<th>Component</th>
<th>Sensitivity</th>
<th>Component</th>
<th>Sensitivity</th>
</tr>
</thead>
<tbody>
<tr>
<td>A. Architectural</td>
<td></td>
<td>B. Mechanical Component</td>
<td></td>
</tr>
<tr>
<td>Exterior Skin</td>
<td></td>
<td>Mechanical Equipment</td>
<td></td>
</tr>
<tr>
<td>Adhered Veneer</td>
<td>S P</td>
<td>Boilers and Furnaces</td>
<td>P</td>
</tr>
<tr>
<td>Anchored Veneer</td>
<td>S P</td>
<td>General Manufacturing and Process Machinery</td>
<td>P</td>
</tr>
<tr>
<td>Glass Blocks</td>
<td>S P</td>
<td>HVAC Equipment, Vibration Isolated</td>
<td>P</td>
</tr>
<tr>
<td>Prefabricated Panels</td>
<td>S P</td>
<td>HVAC Equipment, Non-vibration Isolated</td>
<td>P</td>
</tr>
<tr>
<td>Glazing Systems</td>
<td>S P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Partitions</td>
<td></td>
<td>HVAC Equipment, Mounted In-line with Ductwork</td>
<td>P</td>
</tr>
<tr>
<td>Heavy</td>
<td>S P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Light</td>
<td>S P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interior Veneers</td>
<td></td>
<td>Storage Vessels and Water Heaters</td>
<td></td>
</tr>
<tr>
<td>Stone, Including Marble</td>
<td>S P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ceramic Tile</td>
<td>S P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ceilings</td>
<td></td>
<td>Pressure Piping</td>
<td>P S</td>
</tr>
<tr>
<td>a. Directly Applied to Structure</td>
<td>P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>b. Dropped, Furred, Gypsum Board</td>
<td>P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>c. Suspended Lath and Plaster</td>
<td>S P</td>
<td>Fluid Piping, not Fire Suppression</td>
<td></td>
</tr>
<tr>
<td>d. Suspended Integrated Ceiling</td>
<td>S P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>d. Suspended Integrated Ceiling</td>
<td>S P</td>
<td>Hazardous Materials</td>
<td>P S</td>
</tr>
<tr>
<td>5. Parapets and Appendages</td>
<td>P</td>
<td>Non-hazardous Materials</td>
<td>P S</td>
</tr>
<tr>
<td>6. Ductwork</td>
<td>P S</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7. Chimneys and Stacks</td>
<td>P</td>
<td></td>
<td></td>
</tr>
<tr>
<td>8. Stairs</td>
<td>P S</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Acc=Acceleration-Sensitive  P=Primary Response
Def=Deformation Sensitive  S=Secondary Response
### Table 12: Coefficients for Architectural Components (clause 7.13.3)

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

$^a$ 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)

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

\(^a\) 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)

<table>
<thead>
<tr>
<th>Description of nonstructural element</th>
<th>(I_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Component containing hazardous contents</td>
<td>1.5</td>
</tr>
<tr>
<td>Life safety component required to function after an earthquake (e.g., fire protection sprinklers system)</td>
<td>1.5</td>
</tr>
<tr>
<td>Storage racks in structures open to the public</td>
<td>1.5</td>
</tr>
<tr>
<td>All other components</td>
<td>1.0</td>
</tr>
</tbody>
</table>
6.13.3.2—
For vertical nonstructural elements $F_p$ will be the horizontal force, and for horizontal nonstructural elements $F_p$ will be the vertical force.

6.13.3.3—
For a component mounted on a vibration isolation system, the design force shall be taken as $2F_p$.

6.13.3.4— 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 resistant, ductile, and have adequate anchorages.

6.13.4— Seismic Relative Displacement
Seismic relative displacement $(D_p)$, that a nonstructural element must be designed to accommodate shall be determined as per clause 6.13.4.1, 6.13.4.2 and 6.13.4.3.

6.13.4.1—
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

\[ R(\Delta_{aA} + \Delta_{aB}) \]

where,

\( \delta_{xA} = \text{Deflection at building level } x \text{ of structure } A \text{ due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as per Table 9,} \)

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

\( h_x = \text{Height of level } x \text{ to which upper connection point is attached,} \)

\( h_y = \text{Height of level } y \text{ to which lower connection point is attached,} \)

\( \Delta_{aA} = \text{Allowable storey drift for structure } A \text{ calculated as per 7.11.1, and} \)

\( h_{sx} = \text{Storey height below level } x. \)

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

\[ R(\frac{\Delta_{aA}}{h_{sx}} + \frac{\Delta_{aB}}{h_{sx}}) \]

where,

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

C7.13.4.3-  
Seismic relative displacements must be combined with the displacements due to other loads such as thermal and static loads.
Annex A
Annex B
Annex C
Annex D

(Foreword and Clause 3.15 3.13)

MSK 64 Intensity Scale

a) Type of Structures (Buildings)

Type A - Building in field-stone, rural structures, unburnt-brick houses, clay houses.

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

Type C - Reinforced buildings, well-built wooden structures.

d) Definition of Quantity

<table>
<thead>
<tr>
<th>Single, few</th>
<th>About 5 percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Many</td>
<td>About 50 percent</td>
</tr>
<tr>
<td>Most</td>
<td>About 75 percent</td>
</tr>
</tbody>
</table>

c) Classification of Damage to buildings

<table>
<thead>
<tr>
<th>Grade 1</th>
<th>Slight damage</th>
<th>Fine cracks in plaster; fall of small pieces of plaster</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grade 2</td>
<td>Moderate damage</td>
<td>Small cracks in walls; fall of fairly larger pieces of plaster; pantiles slip off; cracks in chimneys parts of chimney fall down.</td>
</tr>
<tr>
<td>Grade 3</td>
<td>Heavy damage</td>
<td>Large and deep cracks in walls; fall of chimneys.</td>
</tr>
<tr>
<td>Grade 4</td>
<td>Destruction</td>
<td>Gaps in walls; parts of buildings may collapse; separate parts of the buildings lose their cohesion; and inner walls collapse.</td>
</tr>
<tr>
<td>Grade 5</td>
<td>Total damage</td>
<td>Total collapse of the buildings</td>
</tr>
</tbody>
</table>

d) Arrangement of the scale

Introductory letters are used in paragraphs throughout the scale as follows:

i) Persons and surroundings.

ii) Structures of all kinds.

iii) Nature.

e) Intensity Scale

I. Not Noticeable – The intensity of the vibration is below the limits of sensibility; the tremor is detected and recorded by seismograph only.

II. Scarcely noticeable (very slight) – Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.

III. Weak, partially observed only – The earthquake is felt indoors by a few people, outdoors only in favorable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects.

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

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

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

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

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

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

iii) Cracks up to widths of 1cm possible in wet ground; in mountains occasional landslips: change in flow of springs and in level of well water are observed.

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

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

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

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

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

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

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

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

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

X. General destruction of buildings

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

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

XI. Destruction

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

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

XII. Landscape changes

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

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

<table>
<thead>
<tr>
<th>Town</th>
<th>Zone</th>
<th>Zone Factor, Z</th>
<th>Town</th>
<th>Zone</th>
<th>Zone Factor, Z</th>
</tr>
</thead>
<tbody>
<tr>
<td>Agra</td>
<td>III</td>
<td>0.16</td>
<td>Kanpur</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Ahmedabad</td>
<td>III</td>
<td>0.16</td>
<td>Karwar</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Ajmer</td>
<td>II</td>
<td>0.10</td>
<td>Kochi</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Allahabad</td>
<td>II</td>
<td>0.10</td>
<td>Kohima</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>Almora</td>
<td>IV</td>
<td>0.24</td>
<td>Kolkata</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Ambala</td>
<td>IV</td>
<td>0.24</td>
<td>Kota</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Amritsar</td>
<td>IV</td>
<td>0.24</td>
<td>Kurnool</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Äsansol</td>
<td>III</td>
<td>0.24</td>
<td>Lucknow</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Aurangabad</td>
<td>II</td>
<td>0.10</td>
<td>Ludhiana</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Bahraich</td>
<td>IV</td>
<td>0.24</td>
<td>Madurai</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Bangalore (Bengaluru)</td>
<td>II</td>
<td>0.10</td>
<td>Mandi</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>Barauni</td>
<td>IV</td>
<td>0.24</td>
<td>Mangaluru</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Bareilly</td>
<td>III</td>
<td>0.16</td>
<td>Mungher</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Belgaum</td>
<td>III</td>
<td>0.16</td>
<td>Moradabad</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Bhatinda</td>
<td>III</td>
<td>0.16</td>
<td>Mumbai</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Bhilai</td>
<td>II</td>
<td>0.10</td>
<td>Mysuru</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Bhopal</td>
<td>II</td>
<td>0.10</td>
<td>Nagpur</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Bhubaneswar</td>
<td>III</td>
<td>0.16</td>
<td>Nagarjunasagar</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Bhuj</td>
<td>V</td>
<td>0.36</td>
<td>Nainital</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Bijapur</td>
<td>III</td>
<td>0.16</td>
<td>Nasik</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Bikaner</td>
<td>III</td>
<td>0.16</td>
<td>Nellore</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Bokaro</td>
<td>III</td>
<td>0.16</td>
<td>Osmanabad</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Bulandshahr</td>
<td>IV</td>
<td>0.24</td>
<td>Panjim</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Burdwan</td>
<td>III</td>
<td>0.16</td>
<td>Patiala</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Calicut (Kozhikode)</td>
<td>III</td>
<td>0.16</td>
<td>Patna</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Chandigarh</td>
<td>IV</td>
<td>0.24</td>
<td>Pilibhit</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Chennai</td>
<td>III</td>
<td>0.16</td>
<td>Pondicherry (Puducherry)</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Chitradurga</td>
<td>II</td>
<td>0.10</td>
<td>Pune</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Coimbatore</td>
<td>III</td>
<td>0.16</td>
<td>Raipur</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Cuddalore</td>
<td>II</td>
<td>0.10</td>
<td>Rajkot</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Cuttack</td>
<td>III</td>
<td>0.16</td>
<td>Ranchi</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Darbhanga</td>
<td>V</td>
<td>0.36</td>
<td>Roorkee</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Darjeeling</td>
<td>IV</td>
<td>0.24</td>
<td>Rourkela</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Dharwad</td>
<td>III</td>
<td>0.16</td>
<td>Sadiya</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>Dehra Dun</td>
<td>IV</td>
<td>0.24</td>
<td>Salem</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>City</td>
<td>Level</td>
<td>Intensity</td>
<td>City</td>
<td>Level</td>
<td>Intensity</td>
</tr>
<tr>
<td>--------------</td>
<td>-------</td>
<td>-----------</td>
<td>-------------</td>
<td>-------</td>
<td>-----------</td>
</tr>
<tr>
<td>Dharampuri</td>
<td>III</td>
<td>0.16</td>
<td>Shillong</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>Delhi</td>
<td>IV</td>
<td>0.24</td>
<td>Simla</td>
<td>IV</td>
<td>0.24</td>
</tr>
<tr>
<td>Durgapur</td>
<td>III</td>
<td>0.16</td>
<td>Sironj</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Gangtok</td>
<td>IV</td>
<td>0.24</td>
<td>Solapur</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Guwahati</td>
<td>V</td>
<td>0.36</td>
<td>Srinagar</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>Gulbarga</td>
<td>II</td>
<td>0.10</td>
<td>Surat</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Gaya</td>
<td>III</td>
<td>0.16</td>
<td>Tarapur</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Gorakhpur</td>
<td>IV</td>
<td>0.24</td>
<td>Tezpur</td>
<td>V</td>
<td>0.36</td>
</tr>
<tr>
<td>Hyderabad</td>
<td>II</td>
<td>0.10</td>
<td>Thane</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Imphal</td>
<td>V</td>
<td>0.36</td>
<td>Thanjavur</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Jabalpur</td>
<td>III</td>
<td>0.16</td>
<td>Thiruvananthapuram</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Jaipur</td>
<td>II</td>
<td>0.10</td>
<td>Tiruchirappali</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Jamshedpur</td>
<td>II</td>
<td>0.10</td>
<td>Thiruvannamalai</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Jhansi</td>
<td>II</td>
<td>0.10</td>
<td>Udaipur</td>
<td>II</td>
<td>0.10</td>
</tr>
<tr>
<td>Jodhpur</td>
<td>II</td>
<td>0.10</td>
<td>Vadodara</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Jorhat</td>
<td>V</td>
<td>0.36</td>
<td>Varanasi</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Kakrapara</td>
<td>III</td>
<td>0.16</td>
<td>Vellore</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Kalapakkam</td>
<td>III</td>
<td>0.16</td>
<td>Vijayawada</td>
<td>III</td>
<td>0.16</td>
</tr>
<tr>
<td>Kanchipuram</td>
<td>III</td>
<td>0.16</td>
<td>Vishakhapatnam</td>
<td>II</td>
<td>0.10</td>
</tr>
</tbody>
</table>
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
- \((N_1)_{60}\) = Normalized SPT blow count for 60% energy efficiency
- \(C_N\) = SPT blow count normalized for vertical effective stress of 1 atmosphere (i.e., about 100 kPa) and delivery of 60% of theoretical hammer energy
- \(\eta_1, \eta_2, \eta_3\) and \(\eta_4\) = correction factors (see Table F1)

\[C_N = \text{correction for effective overburden pressure} = \sqrt{\frac{P_a}{\sigma'_{v0}}} \text{ subjected to } C_N \leq 1.7\]

(avoiding unnecessary large values near ground surface), where, \(P_a\) = Atmospheric Pressure and \(\sigma'_{v0}\) = 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:

\[(N'_1)_{60} = 15 + \frac{1}{2} ((N_1)_{60} - 15)\]

\((N'_1)_{60}\) = 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>
<thead>
<tr>
<th>Table F1 - Correction factors for Standard Penetration Test (SPT)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Hammer energy correction, (\eta_1)</strong></td>
</tr>
<tr>
<td>System</td>
</tr>
<tr>
<td>Donut hammer and rope and pulley</td>
</tr>
<tr>
<td>Safety hammer and rope and pulley</td>
</tr>
<tr>
<td>Automatic (trip) hammer</td>
</tr>
</tbody>
</table>

| **Rod Length correction, \(\eta_2\)**                     |
| Rod length, m                        | \(\eta_2\)       |
| > 10                                  | 1.00             |
| 6 to 10                               | 0.95             |
| 4 to 6                                | 0.85             |
| 0 to 4                                | 0.75             |

| **Liner correction, \(\eta_3\)**                          |
| Presence or absence of liner, type of soil                | \(\eta_3\)       |
| Without liner, all soils                                 | 1.00             |
| With liner, dense sand and clay                          | 0.80             |
| With liner, loose sand                                   | 0.90             |

| **Borehole diameter correction, \(\eta_4\)**               |
| Borehole diameter, mm                                   | \(\eta_4\)       |
| 60 to 120                                             | 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 $r_d$:

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

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:

$$CSR = 0.65 \left( \frac{a_{max}}{g} \right) \left( \frac{σ_{vd}}{σ′_{vd}} \right) r_d$$

where $a_{max}$ 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:

$$CRR = CRR_{7.5} (MSF) K_σ K_a$$

---

where

\[ \text{CRR}_{7.5} = \text{standard cyclic resistance ratio for a 7.5 magnitude earthquake obtained using values of SPT or CPT or shear wave velocity (as per Step 6) and} \]

\[ \text{MSF} = \text{magnitude scaling given by the following equation:} \]

\[ \text{MSF} = \frac{10^{2.24}}{M_{\text{w}}^{2.56}} \]

\[ \text{MSF} = \left(\frac{M_{\text{w}}}{7.5}\right)^{3.3} \]

This factor is required when the magnitude is different than 7.5. If earthquake magnitude \( M_{\text{w}} \) for the site is not available, it can be taken according to the table below.

**Table G1: Earthquake magnitude \( M_{\text{w}} \)**

<table>
<thead>
<tr>
<th>Earthquake Zone</th>
<th>( M_{\text{w}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone – II</td>
<td>6.0</td>
</tr>
<tr>
<td>Zone – III</td>
<td>6.5</td>
</tr>
<tr>
<td>Zone – IV</td>
<td>7.0</td>
</tr>
<tr>
<td>Zone – V</td>
<td>7.5</td>
</tr>
</tbody>
</table>

The correction for high overburden stresses \( K_\sigma \) 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 \( \sigma'_{vd} \) 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 \( K_\alpha \) is required only for sloping ground and is not required in routine engineering practice. Therefore, in the scope of this standard, value of \( K_\alpha \) 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 \( \text{CRR}_{7.5} \).**

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 \( (N) \):

\[ N_{60} = NC_{60} \]

where

\[ C_{60} = C_{HT}C_{HW}C_{SS}C_{RL}C_{BD} \]

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

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

where

$$C_N = \frac{P_o}{\sigma_{vo}} \leq 1.7$$

The cyclic resistance ratio $CRR_{7.5}$ is estimated from Fig. 8G1, using $\left( N_i \right)_{60}$ value.

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

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

where

$$\alpha = \begin{cases} 
0 & \beta = 1 \\
1.76 - \left( \frac{190}{FC} \right) & \beta = 0.99 + \frac{FC^{1.5}}{1000} \\
5 & \beta = 1.2 
\end{cases}$$

for $FC \leq 5$ percent

for $5 \text{ percent} < FC < 35$ percent

for $FC \geq 35$ percent

Again, Fig. 8G1 can be used to estimate $CRR_{7.5}$, where $\left( N_i \right)_{60CS}$ shall be used instead of $\left( N_i \right)_{60}$ 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. 8G1:

$$CRR_{7.5} = \frac{1}{34 - \left( N_i \right)_{60CS}} + \frac{\left( N_i \right)_{60CS}}{135} + \frac{50}{[10 \times \left( N_i \right)_{60CS} + 45]^2} - \frac{1}{200}$$
Fig. 8G1: Relation Between $CRR_{7.5}$ and $(N)_{60}$ for sand for $M_{w} \geq 7.5$ 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:

$$q_{CIN} = C_Q \left( \frac{q_c}{P_a} \right)$$
where $q_{CN}$ is normalized dimensionless cone penetration resistance, and

$$C_Q = \left( \frac{P_a}{\sigma_{vd}} \right)^n \quad \text{subject to} \quad C_Q \leq 1.7$$

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

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

$$(q_{CN})_{CS} = k_c q_{CN}$$

where

$k_c$ = Correction factor to account for grain characteristics

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

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

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

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

Using $(q_{CN})_{CS}$ find the value of $CRR_{7,5}$ using Fig. 9G2. Alternatively, the $CRR_{7,5}$ can be found from the following equations:

$$CRR_{7,5} = \begin{cases} 
0.833 \left( \frac{(q_{CN})_{CS}}{1000} \right)^3 + 0.05, & 0 < (q_{CN})_{CS} < 50 \\
93 \left( \frac{(q_{CN})_{CS}}{1000} \right)^3 + 0.08, & 50 \leq (q_{CN})_{CS} < 160 
\end{cases}$$
Using shear wave velocity:

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

$$V_{s1} = \left( \frac{P_u}{\sigma'_v} \right)^{0.25} V_s \quad \text{(subject to } V_{s1} \leq 1.3V_s)$$

where $V_{s1}$ is overburden stress corrected shear wave velocity and $\sigma'_v$ 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. 40G3. 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. 40G3 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.

![Diagram](image)

Fig. 10G3: 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)*

<table>
<thead>
<tr>
<th>SI-No. (1)</th>
<th>Element</th>
<th>Standard Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>i)</td>
<td>Sampler</td>
<td>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)</td>
</tr>
<tr>
<td>ii)</td>
<td>Drill Rods</td>
<td>A or AW-type for depths less than 15.2 m; N- or NW-type for greater depths</td>
</tr>
<tr>
<td>iii)</td>
<td>Hammer</td>
<td>Standard (safety) hammer with: (a) weight = 63.5 kg; (b) drop = 762 mm (delivers 60 of theoretical free fall energy)</td>
</tr>
<tr>
<td>iv)</td>
<td>Rope</td>
<td>Two wraps of rope around the pulley</td>
</tr>
<tr>
<td>v)</td>
<td>Borehole</td>
<td>100- to 130-mm diameter rotary borehole with bentonite mud for borehole stability (hollow stem augers where SPT is taken through the stem)</td>
</tr>
<tr>
<td>vi)</td>
<td>Drill Bit</td>
<td>Upward deflection of drilling mud (tricone or baffled drag bit)</td>
</tr>
<tr>
<td>vii)</td>
<td>Blow Count Rate</td>
<td>30 to 40 blows per minute</td>
</tr>
<tr>
<td>viii)</td>
<td>Penetration Resistant Count</td>
<td>Measured over range of 150 to 460 mm of penetration into the ground</td>
</tr>
</tbody>
</table>
Table 12: Correction Factors for Non-Standard SPT Procedures and Equipment

{Clause F-1, Step: 6(a)}

<table>
<thead>
<tr>
<th>Correction for</th>
<th>Correction Factor</th>
</tr>
</thead>
</table>
| 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) |  
$C_{HW} = H \cdot W \over 635 \times 762$ |
| 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.  
$C_{60} = C_{HT} \cdot C_{HW} \cdot C_{SS} \cdot C_{RL} \cdot C_{BD}$  
$N_{60} = N \cdot C_{60}$  
$C_{U} =$ Correction factor for overburden pressure  
$(N_{b})_{60} = C_{U} \cdot N_{60} = C_{U} \cdot C_{60} \cdot N$
References for Commentary

1) ACI 318 (2014). *Building Code Requirements for Reinforced Concrete and Commentary*, American Concrete Institute.


4) ATC 40, Seismic evaluation and retrofit of concrete buildings (Volume 1), Applied Technology Council, Redwood City, California, USA.


19) Farzad Naeim, and James M. Kelly(1999).*Design of Seismic Isolation of Structures – from theory to practice*; John Wiley & Sons, Inc.

20) FEMA 368, 2001, NEHRP Recommended Provisions for Seismic Regulations for New Buildings and


29) Hanson, R.D., and Soong, T.T., 2001, Seismic Design with Supplemental Energy Dissipation Devices; Earthquake Engineering Research Institute, USA.


49) SEAOC. (1999). Recommended lateral force requirements and commentary, Seismology Committee, Structural Engineers Association of California Sacramento, CA.


