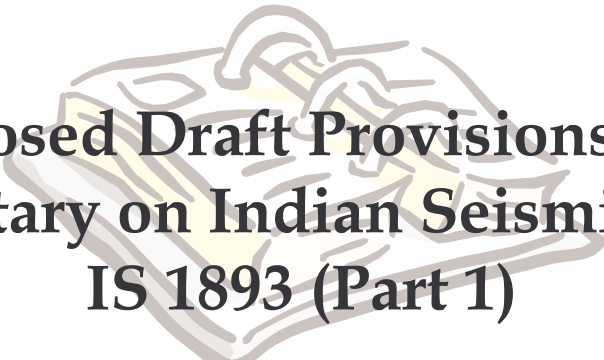


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Final Report :: A - Earthquake Codes

IITK-GSDMA Project on Building Codes



Proposed Draft Provisions and Commentary on Indian Seismic Code IS 1893 (Part 1)

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- The views and opinions expressed are those of the authors and not necessarily of the GSDMA, the World Bank, IIT Kanpur, or the Bureau of Indian Standards.
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Foreword

Foreword

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

Himalayan-Nagalushai region, Indo-Gangetic Plain, Western India, Kutch and Kathiawar regions are geologically unstable parts of the country, and some devastating earthquakes of the world have occurred there. A major part of the peninsular India has also been visited by strong earthquakes, but these were relatively few in number occurring at much larger time intervals at any site, and had considerably lesser intensity. The earthquake resistant design of structures taking into account seismic data from studies of these Indian earthquakes has become very essential, particularly in view of the intense construction activity all over the country. It is to serve this purpose that IS 1893: 1962 'Recommendations for earthquake resistant design of structures' was published and revised first time in 1966.

As a result of additional seismic data collected in India and further knowledge and experience gained since the publication of the first revision of this standard, the

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sectional committee felt the need to revise the standard again incorporating many changes, such as revision of maps showing seismic zones and epicenters, and adding a more rational approach for design of buildings and sub-structures of bridges. These were covered in the second revision of IS 1893 brought out in 1970.

As a result of the increased use of the standard, considerable amount of suggestions were received for modifying some of the provisions of the standard and, therefore, third revision of the standard was brought out in 1975. The following changes were incorporated in the third revision:

- a) The standard incorporated seismic zone factors (previously given as multiplying factors in the second revision) on a more rational basis.
- b) Importance factors were introduced to account for the varying degrees of importance for various structures.
- c) In the clauses for design of multi-storeyed buildings, the coefficient of flexibility was given in the form of a curve with respect to period of buildings.
- d) A more rational formula was used to combine modal shear forces.
- e) New clauses were introduced for determination of hydrodynamic pressures in elevated tanks.
- f) Clauses on concrete and masonry dams were modified, taking into account their dynamic behaviour during earthquakes. Simplified formulae for design forces were introduced based on results of extensive studies carried out since second revision of the standard was published.

The fourth revision, brought out in 1984, was prepared to modify some of the provisions of the standard as a result of experience gained with the use of the standard. In this revision, a number of important basic modifications with-respect to load factors, field values of N , base shear and modal analysis was introduced. A new concept of performance factor depending on the structural framing system and on the ductility of construction was incorporated. Figure 2-3 for average design acceleration spectra was also

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modified and a curve for zero percent damping incorporated.

In the fifth revision brought out in 2002, with a view to keep abreast with the rapid development and extensive research that has been carried out in the field of earthquake resistant design of various structures, the committee ~~has~~ decided to cover the provisions for different types of structures in separate parts. Hence, IS 1893 ~~has been was~~ split into the following five parts:

- a) Part 1 General provisions and buildings
- b) Part 2 Liquid retaining tanks - Elevated and ground supported
- c) Part 3 Bridges and retaining walls
- d) Part 4 Industrial structures including stack like structures
- e) Part 5 Dams and embankments

Part 1 contains provisions that are general in nature and applicable to all structures. Also, it contains provisions that are specific to buildings only. Unless stated otherwise, the provisions in Parts 2 to 5 shall be read necessarily in conjunction with the general provisions in Part 1.

~~NOTE - Pending finalization of Parts 2 to 5 of IS 1893, provisions of Part I will be read along with the relevant clauses of IS 1893: 1984 for structures other than buildings.~~

The following are the major and important

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The note pertaining to pending finalization of other parts of the code causes serious difficulties. For instance, for design of an overhead water tank, one can not combine the two codes with out a specified value of response reduction factor (R). Pending finalization, designers may consider using the following documents as relevant:

1. Jain,S.K., and Jaiswal,O.R., *Proposed Draft Provisions and Commentary on Seismic Design of Liquid Storage Tanks*, IITK-GSDMA Codes Project Report Nos. IITK-GSDMA-EQ08-V2.0 and IITK-GSDMA-EQ14-V1.0 Department of Civil Engineering, IIT Kanpur, October 2004
2. IRC 6, *Interim Provisions for Seismic Design of Bridges*, Indian Roads Congress, New Delhi
3. Dayal,U., Roy,D., and Jain,S.K., *Draft Code Provision and Commentary for Seismic Design of Dams and Embankments*, IITK-GSDMA Codes Project Report Nos. IITK-GSDMA-EQ09-V3.0 and IITK-GSDMA-EQ17-V1.0 Department of Civil Engineering, IIT Kanpur, October 2004

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modifications made in the fifth revision:

- a) The seismic zone map is revised with only four zones, instead of five. Erstwhile Zone I has been merged to Zone II. Hence, Zone I does not appear in the new zoning; only Zones II, III, IV and V do.
- b) The values of seismic zone factors have been changed; these now reflect more realistic values of ~~effective~~ peak ground acceleration considering Maximum Considered Earthquake (MCE). ~~and service life of structure in each seismic zone.~~
- c) ~~Response-Design Acceleration~~ spectra are now specified for three types of founding strata, namely rock and hard soil, ~~medium-stiff~~ soil and soft soil.
- d) Empirical expression for estimating the fundamental natural period T_a of multi-storeyed buildings with regular moment resisting frames has been revised.
- e) This revision adopts the procedure of first calculating the actual force that may be experienced by the structure during the ~~probable~~ maximum considered earthquake, if it were to remain elastic. Then, the concept of response reduction due to ductile deformation or frictional energy dissipation in the cracks is brought into the code explicitly, by introducing the 'response reduction factor' in place of the earlier performance factor.
- f) A lower bound is specified for the design base shear of buildings, based on empirical estimate of the fundamental natural period T_a .
- g) The soil-foundation system factor is dropped. Instead, a clause is introduced to restrict the use of foundations vulnerable to differential settlements in severe seismic zones.
- h) Torsional eccentricity values have been revised upwards in view of serious damages observed in buildings with irregular plans.
- i) Modal combination rule in dynamic

The values of seismic zone factor (Z) in the code are empirical and have been arrived at on the basis of engineering judgment rather than on any rational analysis of expected PGA and service life.

In the earlier editions, design seismic force depended on the factor ' β ', which in turn depended on type of soil and type of foundation. It is well recognized that ground shaking depends on the type of soil and not on the type of foundation. Hence in the 2002 edition of the code, factor β was dropped.

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analysis of buildings has been revised.

- j) Other clauses have been redrafted where necessary for more effective implementation.

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

- a) Specific treatment for different types of irregularity has been specified.
- b) Explicit treatment on RC frame buildings with masonry infill walls has been included.
- c) Torsional provisions have been simplified.
- d) Simplified method for liquefaction potential analysis has been included.

It is not intended in this standard to lay down regulation so that no structure shall suffer any damage during earthquake of all magnitudes. It has been endeavored to ensure that, as far as possible, structures are able to respond, without structural damage to ~~shocks—ground shaking~~ of moderate intensities and without total collapse to ~~shocks—ground shaking~~ of heavy intensities. While this standard is intended for the earthquake resistant design of normal structures, it has to be emphasized that in ~~the case of~~ special structures, such as large and tall dams, long-span bridges, and major industrial projects, ~~etc~~, site-specific detailed investigation should be undertaken to develop seismic design criteria, unless otherwise specified in the relevant clauses.

Though the basis for the design of different types of structures is covered in this standard, it is not implied that detailed dynamic analysis should be made in every case. In highly seismic areas, construction of a type which ~~entails—may result in~~ heavy debris and consequent loss of life and property, such as masonry, particularly mud masonry and rubble masonry, should preferably be avoided. For guidance on precautions to be observed in the construction of buildings, reference may be

This code primarily specifies the criteria to determine the seismic design force. The other codes like IS 4326, IS 13920 etc., which contain the seismic requirement for design, detailing and construction act as adjuncts to IS 1893. For a proper design of an earthquake resistant structure, an engineer must follow IS 1893 along with other seismic design and detailing codes.

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made to IS 4326, IS 13827 and IS 13828.

Earthquake can cause damage not only on account of the shaking which results from them but also due to other chain effects like landslides, floods, tsunamis, fires and disruption to communication. It is, therefore, important to take necessary precautions in the siting, planning and design of structures so that they are safe against such secondary effects also.

Earthquakes can cause damage in a number of ways like 1) damage of construction due to ground shaking, or damage done by 2) surface rupture, 3) landslide, 4) liquefaction etc. There are other possibilities like flood and fire that may break out as a side effect of an earthquake.

The code generally addresses only the first aspect, i.e., the inertia force on the structure. Hence the engineer may need to also cater to the other effects in certain cases.

The Sectional Committee has appreciated that there cannot be an ~~entirely scientific~~ absolutely quantitative basis for zoning in view of the scanty data available. Though the magnitudes of different earthquakes which have occurred in the past are known to a reasonable degree of accuracy, the intensities of the ~~shocks~~ ground shaking caused by these earthquakes have so far been mostly estimated by damage surveys and there is little instrumental evidence to corroborate the conclusions arrived at. Maximum intensity at different places can be fixed on a scale only on the basis of the observations made and recorded after the earthquake and thus a zoning map which is based on the maximum intensities arrived at, is likely to lead in some cases to an incorrect conclusion in view of (a) incorrectness in the assessment of intensities, (b) human error in judgment during the damage survey, and (c) variation in quality ~~and of~~ design and construction of structures causing variation in type and extent of damage to the structures for the same intensity of shock. The Sectional Committee has therefore, considered that a rational approach to the problem would be to arrive at a zoning map based on known magnitudes and the known epicentres (see Annex A) assuming all other conditions as being average and to modify such an idealized isoseismal map in light of tectonics (see Annex B), lithology (see Annex C) and the maximum intensities as recorded from damage surveys. The Committee has also reviewed such a map in the light of the past history ~~and future possibilities~~ and ~~also~~ attempted to draw the lines demarcating the different zones so as to be clear of important towns, cities and industrial areas, after

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making special examination of such cases, as a little modification in the zonal demarcations may mean considerable difference to the economics of a project in that area. Maps shown in Fig. 1 and Annexes A, B and C are prepared based on information available upto 1993.

In the seismic zoning map, Zone I and II of the ~~contemporary-1970 zone~~ map have been merged and assigned the level of Zone II. The Killari area has been included in Zone III and necessary modifications made, keeping in view the probabilistic hazard evaluation. The Bellary isolated zone has been removed. The parts of eastern coast areas have shown similar hazard to that of the Killari area, the level of Zone II has been enhanced to Zone III and connected with Zone III of Godawari Graben area.

The seismic hazard level ~~with respect to ZPA at 50 percent risk level and 100 years service life~~ goes on progressively increasing from southern peninsular portion to the Himalayan main seismic source, the revised seismic zoning map has given status of Zone III to Narmada Tectonic Domain, Mahanandi Graben and Godawari Graben. This is a logical normalization keeping in view the apprehended higher strain rates in these domains on geological consideration of higher neotectonic activity recorded in these areas.

Attention is particularly drawn to the fact that the intensity of ~~shock-ground shaking~~ due to an earthquake could vary locally at any place due to variation in soil conditions. Earthquake response of systems would be affected by different types of foundation system in addition to variation of ground motion due to various types of soils. Considering the effects in a gross manner, the standard gives guidelines for arriving at design seismic coefficients based on stiffness of base soil.

It is important to note that the seismic coefficient, used in the design of any structure, is dependent on many variable factors and it is an extremely difficult task to determine the exact seismic coefficient in each given case. It is, therefore, necessary to indicate broadly the seismic coefficients that

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The 2002 edition of the code mentioned “50 percent risk level and 100 years of service life.” This was somewhat misleading, as it gave a false impression that the values of Z given in the code are for 50 percent risk level and a structure with 100 years service life. Hence, this mention has now been dropped.

The seismic zoning map broadly classifies India into zones where one can expect earthquake shaking of the more or less the same maximum intensity. The shaking intensity associated with different zones (Table C1) is as follows.

Table C 1 – MSK Intensity related to different seismic zones in India

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could generally be adopted in different parts or zones of the country though, of course, a rigorous analysis considering all the factors involved has to be made in the case of all important projects in order to arrive at a suitable seismic coefficients for design. The Sectional Committee responsible for the formulation of this standard has attempted to include a seismic zoning map (see Fig. 1) for this purpose. The object of this map is to classify the area of the country into a number of zones in which one may reasonably expect earthquake shaking of more or less same maximum intensity in future. The ~~Intensity-intensity~~ as per ~~Comprehensive Intensity Scale~~ (MSK 64) ~~Intensity Scale~~ (see Annex D) broadly associated with the various zones is VI (or less), VII, VIII and IX (and above) for Zones II, III, IV and V respectively. The maximum seismic ground acceleration in each zone cannot be presently predicted with accuracy either on a deterministic or on a probabilistic basis. The basic zone factors included herein are reasonable estimates of ~~effective~~ peak ground accelerations for the design of various structures covered in this standard. Zone factors for some important towns are given in Annex E.

Base isolation and energy absorbing devices may be used for earthquake resistant design. Only standard devices having detailed experimental data on the performance should be used. The designer must demonstrate by detailed analyses that these devices provide sufficient protection to the buildings and equipment as envisaged in this standard. Performance of locally assembled isolation and energy absorbing devices should be evaluated experimentally before they are used in practice. Design of buildings and equipment using such device should be reviewed by the competent authority.

Base isolation systems are found useful for short period structures, say those with fundamental periods less than 0.7 s including

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Zone	Area liable to shaking intensity (MSK)
II	VI (and lower)
III	VII
IV	VIII
V	IX (and higher)

The intensity of ground shaking is commonly measured in terms of Modified Mercalli scale or MSK (Medvedev-Sponhener-Karnik) intensity scale (Annex D of this code). The MSK intensity scale is quite comparable to the Modified Mercalli intensity scale but is more convenient for application in field and is widely used in India. Hence, the mention of Modified Mercalli intensity scale in the earlier editions of the code has been replaced by MSK scale.

The zoning criterion of the map is based on likely intensity. It does not give us any idea regarding how often a shaking of certain intensity may take place in a location (that is, probability of occurrence or return period). For example, say area A experiences a maximum intensity VIII every 50 years and area B experiences a maximum intensity VIII every 300 years. But both these areas will be placed in zone IV, even though area A has higher seismicity. The current trend worldwide is to specify the zones in terms of ground acceleration that has a certain probability of being exceeded in a given number of years.

The code does not provide specifications for the use of base isolation systems or energy absorbing devices. It is expected that the designer will use specialist literature and codes of other countries for design of structures with either of these. This paragraph emphasizes the need for extensive testing of base isolation and energy absorbing devices, which may be used for earthquake resistant design. One may refer to following publications and code for more information on base isolation systems and energy absorbing devices.

- 1) Constantinou, M.C., Soong, T.T., and Dargush, G.F., *Passive Energy Dissipation Systems for Structural Design and Retrofit*, Monograph Series, MCEER, 1998.
- 2) Soong, T.T., and Dargush G. F., *Passive*

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soil-structure interaction less than 0.7 s.

Energy Dissipation Systems in Structural Engineering; John Wiley & Sons, Inc, 1997.

- 3) Farzad Naeim, and James M. Kelly, *Design of Seismic Isolation of Structures – from theory to practice*; John Wiley & Sons, Inc., 1999.
- 4) Hanson, R.D., and Soong, T.T., *Seismic Design with Supplemental Energy Dissipation Devices*; Earthquake Engineering Research Institute, 2001.
- 5) IBC 2003, *International Building Code*, International Code Council, USA.

The first application of base isolation in India was made on two small one-story public buildings in Killari village, close to the epicenter of 1993 earthquake. The project was completed in February 1999. A brief discussion on this is available in:

‘Innovative Earthquake Recovery in India’, Lessons Learned Over Time, Learning from Earthquake Series, Volume II, Earthquake Engineering Research Institute, USA, 1999.

The above report is also available at the following web site:

http://www.nicee.org/readings/EERI_Report.htm

The second application of base isolation system in India is on a 300-bed hospital at Bhuj (Gujarat) after the 2001 Bhuj earthquake in Gujarat.

Base isolation increases the natural frequency of the structure. A higher natural period implies a lower acceleration and hence lower inertia force is experienced by the structure (For example, refer Fig.2 of the code). It can thus be inferred that base isolation is effective for short period structures.

In the formulation of this standard, due weightage has been given to international coordination among the standards and practices prevailing in different countries in addition to relating it to the practices in the field in this country. Assistance has particularly been derived from the following publications:

- a) ~~UBC 1994, Uniform Building Code, International Conference of Building Officials, Whittier, California, U.S.A. 1994.~~ IBC 2003, *International Building*

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Code, International Code Council, USA.

- b) NEHRP ~~1994~~2003, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Part 1: Provisions, Report No. FEMA ~~222~~ 450, Federal Emergency Management Agency, Washington, D.C., U.S.A., ~~1992~~2004.
- c) NEHRP ~~1994~~2003, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures*, Part 2: Commentary, Report No. FEMA ~~223~~ 450, Federal Emergency Management Agency, Washington, D.C., U.S.A., January ~~1992~~2004.
- d) NZS 4203: 1992, *Code of Practice for General Structural Design and Design Loadings for Buildings*, Standards Association of New Zealand, Wellington, New Zealand, 1992.
- e) Murty, C.V.R. and Jain, S.K., *A proposed draft for IS 1893 – Provisions on seismic design of buildings; Part I: Code*, Journal of Structural Engineering, SERC Chennai, Vol. 22, No 1, April 1995, pp 21-29.
- f) Jain, S.K., *A proposed draft for IS 1893 – Provisions on Seismic Design of Buildings; Part II: Commentary and Examples*, Journal of Structural Engineering, SERC Chennai, Vol. 22, No 1, April 1995, pp 21-29.
- g) SEI /ASCE 7-02, ASCE Standard, *Minimum Design Loads for Buildings & Other Structures*, American Society of Civil Engineers, Reston, Virginia.

In the preparation of this standard considerable assistance has been given by the Department of Earthquake Engineering, University of Roorkee; Indian Institute of Technology Kanpur; ~~IT~~ Indian Institute of Technology—Bombay, Mumbai; Geological Survey of India; India Meteorological Department, and several other organizations. Significant improvements have also been made to the code through 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

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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 ~~FG~~.

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.

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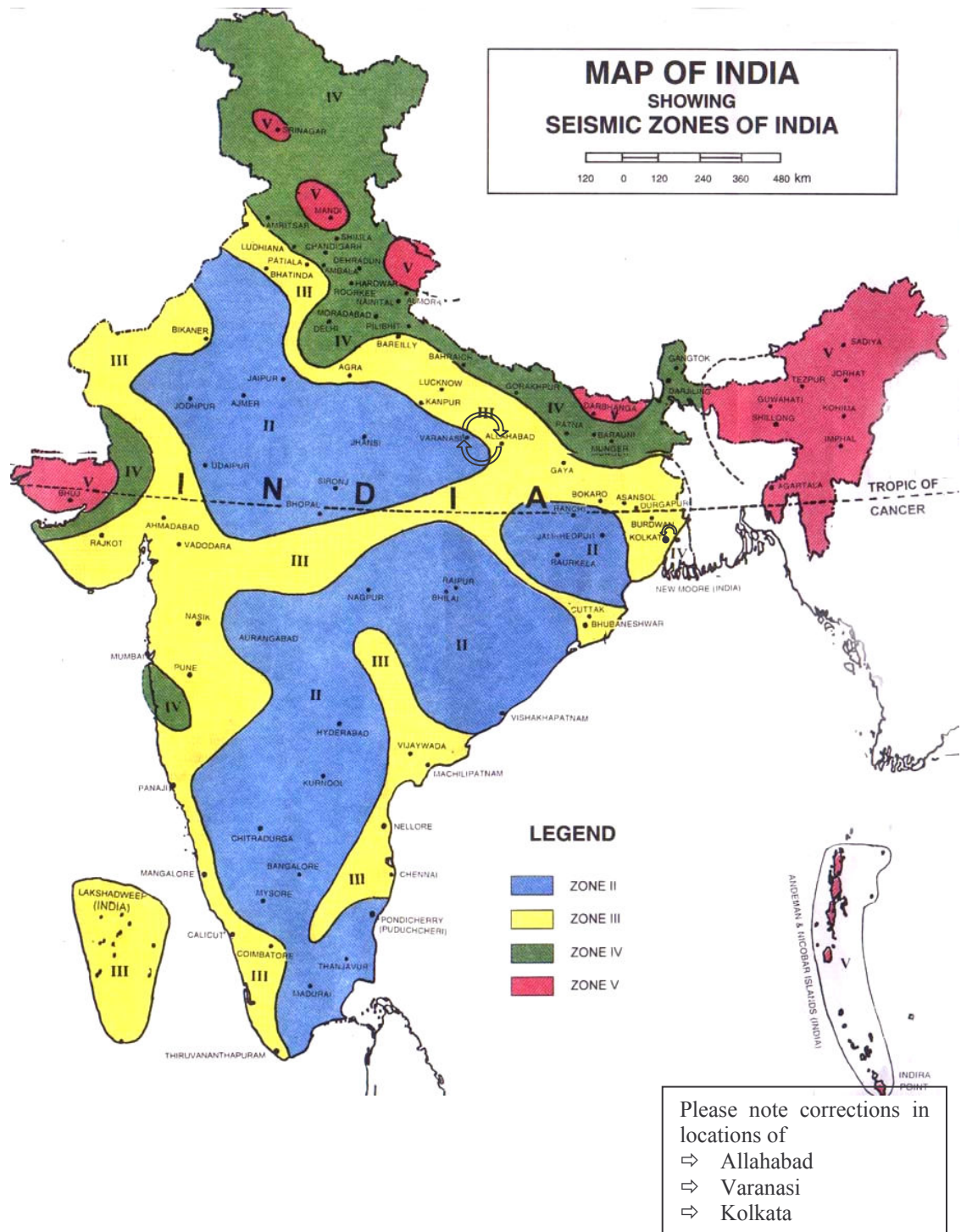


Figure 1 – Seismic Zones of India

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1. – Scope

1.1 –

This standard (Part 1) primarily deals with assessment of seismic ~~loads–design input parameters~~ on various structures, and earthquake resistant design of buildings. Its basic provisions on seismic design input parameters are also applicable to ~~buildings; elevated-liquid retaining~~ structures; industrial and stack-like structures; bridges; concrete, masonry and earth dams; embankments; and retaining walls ~~and other structures~~.

1.2 –

Temporary elements such as scaffolding, temporary excavations need not be designed for earthquake forces.

1.3 –

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 the following Indian Standards:

IS 4326, IS 13827, IS 13828, IS 13920 and IS 13935.

C1.1 –

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

C1.3 –

The latest versions of the codes mentioned are as follows:

- IS 4326 : 1993 (Reaffirmed 1998) Edition 3.2 (2002-2004) – Earthquake Resistant Design and Construction of Buildings – Code of Practice
- IS 13827 : 1993 (Reaffirmed 1998) – Improving Earthquake Resistance of Earthen Buildings – Guidelines
- IS 13828 : 1993 (Reaffirmed 1998) – Improving Earthquake Resistance of Low Strength Masonry buildings – Guidelines
- IS 13920 : 1993 (Reaffirmed 1998) Edition 1.2 (2002-2003) – Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces – Code of Practice
- IS 13935 : 1993 (Reaffirmed 1998) Edition 1.1 (2002-2004) – Repair and Seismic Strengthening of Buildings –Guidelines

CODE**COMMENTARY****2. – References****2.1 –**

The following Indian Standards are necessary adjuncts to this standard:

IS No.	Title
456: 2000	Code of practice for plain and reinforced concrete (<i>fourth revision</i>)
800: 1984	Code of practice for general construction in steel (<i>Second revision</i>)
875	Code of practice for design loads (other than earthquake) for buildings and structures:
(Part 1): 1987	Dead loads – Unit weights of building material and stored materials (<i>second revision</i>)
(Part 2): 1987	Imposed loads (<i>second revision</i>)
(Part 3): 1987	Wind loads (<i>second revision</i>)
(Part 4): 1987	Snow loads (<i>second revision</i>)
(Part 5): 1987	Special loads and load combinations (<i>second revision</i>)
1343: 1980	Code of practice for pre-stressed concrete (<i>first revision</i>)
1498: 1970	Classification and identification of soils for general engineering purposes (<i>first revision</i>)
1888: 1982	Method of load test on soils (<i>second revision</i>)
1893(Part 4)	Criteria for earthquake resistant design of structures: Part 4 Industrial structures including stack like structures
2131: 1981	Method of standard penetration test for soils (<i>first revision</i>)
2809:1972	Glossary of terms and symbols relating to soil engineering (<i>first revision</i>)
2810: 1979	Glossary of terms relating to soil dynamics (<i>first revision</i>)
4326:1993	Earthquake resistant design and construction of buildings - Code of practice (<i>second revision</i>)

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6403: 1981	Code of practice for determination of bearing capacity of shallow foundations (<i>first revision</i>)
13827:1993	Improving earthquake resistance of earthen buildings – Guidelines
13828:1993	Improving earthquake resistance of low strength masonry buildings – Guidelines
13920:1993	Ductile detailing of reinforced concrete structures subjected to seismic forces - Code of practice
13935:1993	Repair and seismic strengthening of buildings – Guidelines
SP 6 (6): 1972	Handbook for structural engineers: Application of plastic theory in design of steel structures

CODE**COMMENTARY****3. – Terminology ~~of~~ in Earthquake Engineering****3.1 –**

For the purpose of this standard, the following definitions shall apply which are applicable generally to all structures.

NOTE - For the definitions of terms pertaining to soil mechanics and soil dynamics references may be made to IS 2809 and IS 2810.

3.2 – Closely-Spaced Modes

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

3.3 – Critical Damping

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

3.4 – Damping

The effect of internal friction, imperfect elasticity of material, slipping, sliding, etc in reducing the amplitude of vibration and is expressed as a percentage of critical damping ([see 3.3](#)).

C3.4 – 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.

CODE**COMMENTARY****3.5 – Design Acceleration Spectrum**

Design acceleration spectrum refers to ~~an average-smoothened-plot-graph~~ of maximum acceleration as a function of natural frequency or ~~time-natural~~ period of vibration of a single degree of freedom system for a specified damping ratio, to be used in the design of structures for earthquake excitations at the base of a single degree of freedom system.

C3.5 – Design Acceleration Spectrum

See commentary of clause 6.4.

Use of two different terms used in earlier edition of the code, namely ‘Response Spectrum’ and ‘Design Acceleration Spectrum’, for essentially 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.

3.6 – Design Basis Earthquake

~~It is the earthquake which can reasonably be expected to occur at least once during the design life of the structure. Design Basis Earthquake motion has been assumed as 50% of the ground motion for the Maximum Considered 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.

C3.6 – Design Basis Earthquake

See commentary of clause 3.19.

3.7– Design Horizontal Acceleration Coefficient (A_h)

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

C3.7 – 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.

3.8 – Design Lateral Horizontal Force

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

3.9 – Ductility**C3.9 – Ductility**

CODE

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

3.10 – Epicentre

The geographical point on the surface of earth vertically above the focus of the earthquake.

3.11– ~~Effective~~ Peak Ground Acceleration (~~EPGA~~)

~~It is 0.4 times the 5 percent damped average spectral acceleration between period 0.1 to 0.3 s. This shall be taken as Zero Period acceleration (ZPA). Peak ground acceleration is the maximum acceleration of the ground in a given direction of ground shaking. It refers to horizontal motion unless specified otherwise.~~

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Ductility is a very important property, specially 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.

C3.10 – Epicentre

Distance from epicenter to any point of interest is called epicentral distance.

For more information on seismological terms one may refer <http://www.nicee.org/EOTips/EOTip03.pdf>

C3.11 – Peak Ground Acceleration (PGA)

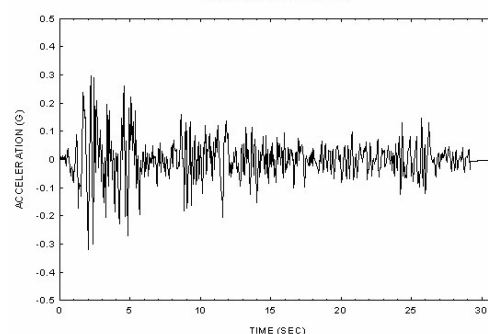


Figure C 1 Horizontal ground motion at El Centro during Imperial Valley earthquake

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 sec 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

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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 C 3.27 for Response Spectrum.

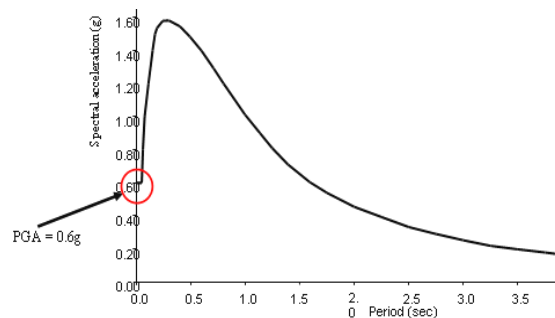


Figure C 2 – Typical shape of an acceleration response spectrum

Figure C 2 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 ≈ 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.

3.12 – Floor Response Spectra

Floor response ~~spectra~~ spectrum is the response ~~spectra~~ spectrum for a time history motion of a floor excited by a given ground motion. This floor motion time history is obtained by an analysis of multi-storey building-the structure for appropriate material damping values subjected to a specified earthquake motion at the base of the structure.

C3.12 – 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 equipments, and auxiliary systems mounted on a floor of a structure.

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3.13 – Focus

~~The originating earthquake source of the elastic waves inside the earth which cause shaking of ground due to earthquake. The point on the fault where the slip starts.~~

C3.13 – Focus

It is also termed as hypocenter. The depth of focus from the ground is called focal depth and is an important parameter in determining the damaging potential of an earthquake.

For more information on seismological terms refer to

<http://www.nicee.org/EOTips/EOTip03.pdf>

3.14 – Importance Factor (*I*)

It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterized by ~~hazardous consequences of the risk resulting from its failure. Here, the risk is associated with hazardous consequences of the failure of the structure,~~ its post-earthquake functional need, historic value, ~~or and~~ economic importance.

3.15 – Intensity of Earthquake

The intensity of an earthquake at a place is a measure of the strength of shaking ~~manifested at that place~~ during the earthquake, and is indicated by a number according to the ~~modified Mercalli Scale or~~ M.S.K. Scale of seismic intensities (see Annex D).

C3.15 – 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 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).

3.16 – Liquefaction

Liquefaction is a state ~~primarily~~ in saturated cohesionless soils wherein the effective shear strength is reduced to negligible value for all engineering purposes ~~due to when the~~ pore pressure caused by vibrations during an earthquake ~~when they~~ approaches the total confining pressure. In this condition, the soil tends to behave like a fluid mass (See Annex F).

C3.16 – Liquefaction

During earthquake shaking, loose saturated sand tends to densify, leading to build up of pore water pressure. As the loading and unloading during ground shaking occurs suddenly, the pore pressure is not able to dissipate. As the total stress remains constant, an increase in pore water pressure leads to a decrease in effective stress. As the pore pressure approaches total stress, the effective stress tends to diminish and the soil loses all its strength and tends to behave like a

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fluid mass.

3.17 – Lithological Features

These reflect the nature of the geological formation of the ~~earths~~ Earth's crust above bed rock characterized on the basis of ~~such characteristics as~~ colour, structure, mineralogical composition and grain size.

3.18 – Magnitude of Earthquake (Richter's Magnitude)

The magnitude of earthquake is a number, which is a measure of energy released in an earthquake. It is defined as logarithm to the base 10 of the maximum trace amplitude, expressed in microns, which the standard short-period torsion seismometer (with a period of 0.8 s, magnification 2800 and damping nearly critical) would register due to the earthquake at an epicentral distance of 100 km.

C3.18 – Magnitude of Earthquake (Richter's Magnitude)

Magnitude is quantitative measure of total size of earthquake. An increase in magnitude by 1.0 implies about 10 times higher waveform amplitude and about 31 times higher energy released. Most of the energy released goes into heat and fracturing the rock, and only a small fraction of it goes into the seismic waves that travel to large distances causing shaking of the ground en-route and hence damage to the structures.

A number of definitions are used for magnitude, namely Richter (or Local) magnitude, surface wave magnitude, body wave magnitude and wave energy magnitude. Details of different magnitude scales are available in standard books on seismology.

3.19 – Maximum Considered Earthquake (MCE)

The most severe earthquake ~~effects~~ considered by this standard.

C3.19 – Maximum Considered Earthquake (MCE)

A number of terms with somewhat similar meaning are used in the literature. Maximum Credible Earthquake is the largest reasonably conceivable earthquake that appears possible along a recognized fault or within a tectonic 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'

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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 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 3.6.

CODE**COMMENTARY****3.20 – Modal Mass (M_k)**

Modal mass of a structure subjected to horizontal or vertical, (as the case may be), ground motion is a part of the total seismic mass of the structure that is effective in mode k of vibration. The modal mass for a given mode has a unique value irrespective of scaling of the mode shape.

C3.20 – 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 7.8.4.5. 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.

3.21 – Modal Participation Factor (P_k)

Modal participation factor of mode k of vibration is the amount by which mode k contributes to the overall vibration of the structure under horizontal and vertical earthquake ground motions. Since the amplitudes of ~~95 percent~~ mode shapes can be scaled arbitrarily, the value of this factor depends on the scaling used for mode shapes.

C3.21 – Modal Participation Factor (P_k)

It is a term used in dynamic analysis (clause 7.8.4.5).

3.22 – Modes of Vibration

(See ~~Normal Mode~~ 3.25)

3.23 – Mode Shape Coefficient (Φ_{ik})

The spatial pattern of vibration when the structure is vibrating in its normal mode k is called the mode shape of vibration of mode k .

A structure with n degrees of freedom possesses N natural frequencies and N mode shapes. The matrix $[\Phi]$, in which each mode shape appears as a column, is called the modal matrix. The element Φ_{ik} is called the mode shape coefficient associated with degree of freedom i and mode k . A linear transformation, with $[\Phi]$ as the transformation matrix, uncouples the N coupled equations of motion into a set of N independent single degree of freedom systems.

When a system is vibrating in normal mode k , at any particular instant of time, the amplitude of mass i expressed as a ratio of

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~~the amplitude of one of the masses of the system, is known as mode shape coefficient (Φ_{ik}).~~

3.24 – Natural Period (T)

~~Natural period of a structure is its time period of undamped free vibration. The frequencies at which normal mode vibrations are possible for a structure are called the natural frequencies of the structure. The structure is said to be vibrating in its k^{th} normal mode when its frequency is equal to the k^{th} natural frequency.~~

~~The k^{th} natural period is the reciprocal of the k^{th} natural frequency expressed in Hz.~~

3.24.1 – Fundamental Natural Period (T_1)

~~It is the first (longest) modal time period of vibration. The lowest of the natural frequencies of a structure is called its fundamental natural frequency. The associated natural period is called the fundamental natural period.~~

3.24.2 – Modal Natural Period (T_k)

The modal natural period of mode k is the ~~time natural~~ period of vibration in mode k .

3.25 – Normal Mode

~~These are special undamped free vibrations in which all points on the structure vibrate harmonically at the same frequency such that all these points reach their individual maximum responses simultaneously. A system is said to be vibrating in a normal mode when all its masses attain maximum values of displacements and rotations simultaneously, and pass through equilibrium positions simultaneously.~~

3.26 – Response reduction**C3.24 – 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.

C3.26 – Response Reduction

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Factor (R)

It is the factor by which the ~~actual base shear force, that would be generated if the structure were to remain elastic during its response to the Design Basis Earthquake (DBE) shaking, shall be reduced to obtain the design lateral force~~ elastic responses of the structure, such as base shear and element forces (e.g., axial forces, shear forces and bending moments), generated under the action of the earthquake shaking as specified in this code, are reduced to obtain the design values of the responses.

3.27 – Response Spectrum

The representation of the maximum response of idealized single degree freedom systems having certain period and damping, during earthquake ground motion. The maximum response is plotted against the undamped natural period and for various damping values, and can be expressed in terms of maximum absolute acceleration, maximum relative velocity, or maximum relative displacement.

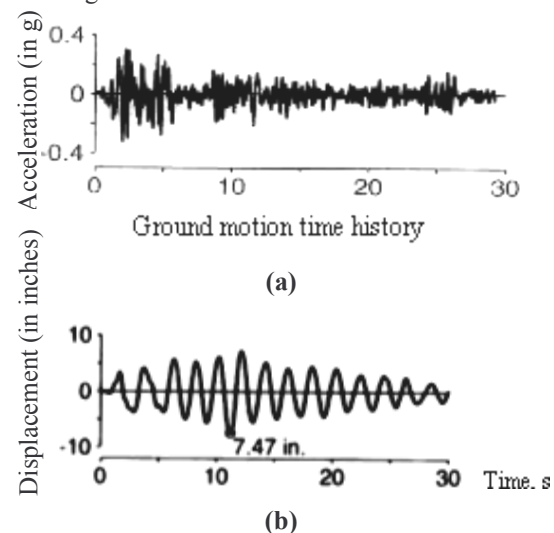
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Factor (R)

See commentary of clause 6.4.2.

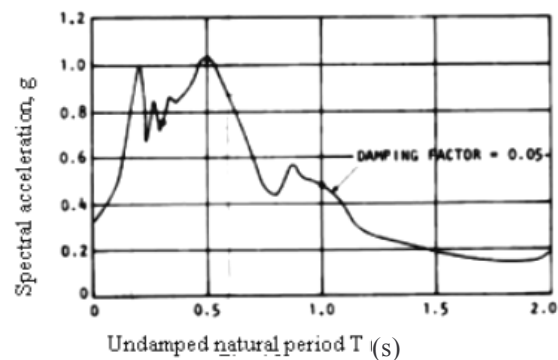
C3.27 – Response Spectrum

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 ‘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 C 3). 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.



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(c)

Figure C 3 – (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 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*.

3.28 – Seismic Mass

It is the seismic weight divided by

C3.28 – Seismic Mass

It is the seismic weight divided by the acceleration due to gravity, i.e., it is in units of

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acceleration due to gravity.

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.

3.29 – Seismic Weight (W)

It is the total dead load plus appropriate amounts of specified imposed load.

**3.30 – Structural Response
Factors Response
Acceleration Coefficient (S_a/g)**

It is a factor denoting the design acceleration ~~response~~-spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure.

C3.30 – Response Acceleration Coefficient (S_a/g)

2002 edition of code used several terms, namely ‘acceleration response factor’, ‘structural response factor’, ‘average response acceleration coefficient’, etc., essentially for the same thing, i.e., S_a/g . The term ‘Response Acceleration Coefficient’ is now used consistently throughout the code.

3.31 – Tectonic Features

The nature of geological formation of the bed rock in the earth's crust revealing regions characterized by structural features, such as dislocation, distortion, faults, folding, thrusts, volcanoes with their age of formation, which are directly involved in the earth movement or quake resulting in the above consequences.

3.32 – Time History Analysis

It is an analysis of the dynamic response of the structure at each increment of time, when its base is subjected to a specific ground motion time history.

C3.32 – Time History Analysis

Dynamic analysis can be performed either as time history analysis, or as response spectrum analysis. Time history analysis is a more sophisticated method and is rarely used for the design of ordinary structures. Time history analysis can be performed by modal superposition method or by using direct integration of equations of motion. A number of text books are available that cover dynamic analysis.

3.33 – Zone Factor (Z)

It is a factor to obtain the design acceleration

C3.33 – Zone Factor (Z)

The values of zone factor for different seismic

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spectrum depending on the perceived ~~maximum~~ seismic risk hazard characterized by ~~Maximum Considered Earthquake (MCE)~~ in the zone in which the structure is located. The basic zone factors included in this standard are reasonable estimate of ~~effective~~ peak ground acceleration.

zones in India are given in clause 6.4.2. These have been arrived at empirically using engineering judgment.

3.34— Zero Period Acceleration (ZPA)

~~It is the value of acceleration response spectrum for period below 0.03 s (frequencies above 33 Hz.)~~

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4. – Terminology ~~for~~ in Earthquake Engineering of Buildings

4.1 –

For the purpose of earthquake resistant design of buildings in this standard, the following definitions shall apply.

4.2 – Base

It is the level at which inertia forces generated in the structure are transferred to the foundation, which then transfers these forces to the ground.

4.3 – Base Dimensions (*d*)

Base dimension of the building along a direction of shaking is the dimension (in meters) at its base, ~~in metre,~~ along that direction.

4.4 – Centre of Mass

The point through which the resultant of the masses of a system acts. This point corresponds to the centre of gravity of masses of system.

4.5 – Centre of **Stiffness** Rigidity

~~The point through which the resultant of the restoring forces of a system acts.~~

Single storey buildings:

If the 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 the center of rigidity.

Multi-storey buildings:

Either of the following two definitions may be

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used. The two definitions may give somewhat different values of design eccentricity but the difference is not very substantial. Hence, choice of the definition is left to the designer.

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 2). As per this definition, location of CR is dependent on building the stiffness properties as well as on the applied lateral load profile.

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 2). This definition is independent of applied lateral load.

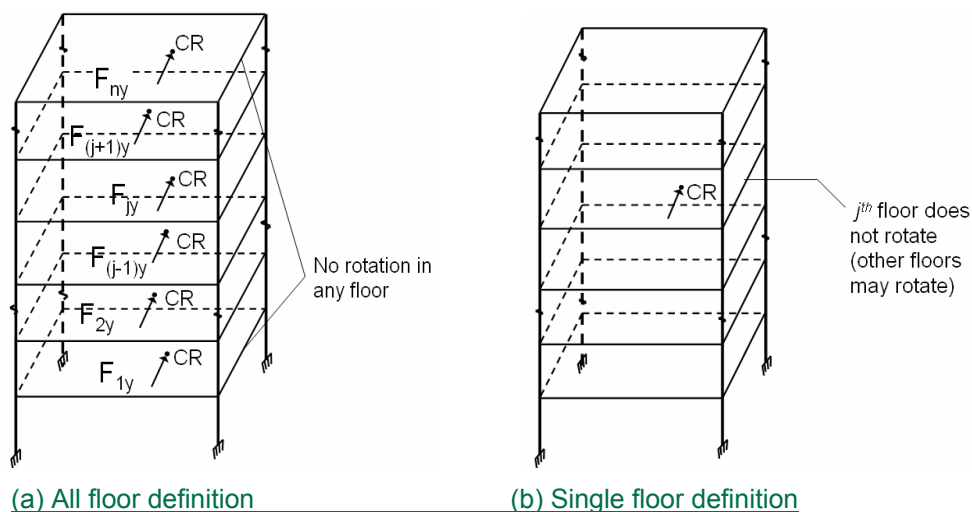


Figure 2 – Definition of Center of Rigidity

4.6 – Design Eccentricity (e_{di})

It is the value of eccentricity to be used at for the j^{th} floor i in torsion calculations. ~~for design.~~

C4.6 – Design Eccentricity (e_{di})

See commentary of clause 7.9.2.

4.7 – Design Seismic Base Shear (V_B)

It is the total design lateral force at the base

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of a structure.

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 the vertical resisting elements. ~~for example, reinforced concrete floors and horizontal bracing systems.~~

4.9 – Dual System

Buildings with dual system consist of shear walls (or braced frames) and moment resisting frames such that:

- a) The two systems are designed to resist the total design lateral force in proportion to their lateral stiffness considering the interaction of the dual system at all floor levels; and
- b) The moment resisting frames are designed to independently resist at least 25 percent of the design base shear.

C4.9 – Dual System

Consider buildings with shear walls and moment resisting frames. The 1984 version of the code (Table 5) implied that frames should be designed to take at least 25% of the total design seismic loads.

The code now allows for two possibilities: (a) the frames are designed for at least 25% of total base shear (termed as dual system), or (b) frames may not be designed for at least 25% of total base shear.

Example 1: Analysis indicates that the frames are taking 30% of total seismic load while 70% loads go to shear walls. Frames and walls will be designed for these forces and the system will be termed as dual system.

Example 2: Analysis indicates that frames are taking 10% and walls take 90% of the total seismic load. To qualify for dual system, design the walls for 90% of total load, but design the frames to resist 25% of total seismic load.

Example 3: Analysis indicates that shear walls take 90% and frames take 10% of seismic load. The frames will be designed for 10% while walls will be designed for 90% of total seismic loads (footnote 4 (b) of table 7). This will not be a dual system.

Example 4: Analysis indicates that shear walls are taking 90% and frames are taking 10% of seismic load. The designer has the option to assume that the entire load is carried by the shear walls. The shear walls will be designed for 100% of the total seismic loads, and the frames will be treated as gravity frames, i.e., it is assumed that frames carry no seismic loads.

Table 7 in IS 1893: 2002 gives a different value

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of response reduction factor (R) for dual systems. The dual systems perform much better when subjected to seismic conditions and are designed for lower values of design force.

4.10 – Height of Floor (h_i)

It is the difference in levels between the base of the building and that of floor i .

**4.11 – Height of ~~Structure~~
Building (h)**

~~Height of building, (in meters), excludes the basement storeys if basement walls are connected with the ground floor slab or fitted between the building columns, but includes the basement storeys if they are not so connected. It is the difference in levels, in metres, between its base and its highest level.~~

4.12 – Horizontal Bracing System

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

4.13 – Joints

It is the portion of the column that is common to other members, for example, beams, framing into it.

4.14 – Lateral Force Resisting System

It is part of the structural system assigned to resist lateral forces.

4.15 – Moment-Resisting Frame

It is a frame in which members and joints are capable of resisting forces primarily by

C4.15 – 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

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flexure.

4.15.1– Ordinary Moment-Resisting Frame **(OMRF)**

It is a moment-resisting frame not meeting special detailing requirements for ductile behaviour.

4.15.2– Special Moment-Resisting Frame **(SMRF)**

It is a moment-resisting frame specially detailed to provide ductile behaviour and comply with the requirements given in ~~IS 4326~~ or IS 13920 or SP 6 (6).

4.15.3– Intermediate Moment Resisting Frame **(IMRF)**

It is an RC moment resisting frame detailed as per IS:13920 to have ductility intermediate between Ordinary Moment Resisting Frame and Special Moment Resisting Frame.

4.16– Number of Storeys (n)

~~Number of storeys of a building is the number of levels above the base. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.~~

4.17 4.16 – Principal Axes

Principal axes of a building are generally two mutually perpendicular horizontal directions in plan of a building along which the geometry of the building is oriented.

4.18 4.17 – $P-\Delta$ Effect

It is the secondary effect on shears and moments of frame members due to action of the vertical loads, interacting with the lateral displacement of building resulting from seismic forces.

comparing the R values in Table 7.

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.

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4.19 4.18 – Shear Wall

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

C4.18 – Shear Wall

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

4.20 4.19 – Soft Storey

It is one in which the lateral stiffness is less than ~~70-60~~ percent of that in the storey above or less than ~~80-70~~ percent of the average lateral stiffness of the three storeys above.

4.21 4.20 – Static Eccentricity (e_{si})

It is the distance between centre of mass and centre of rigidity of floor i .

4.22 4.21 – Storey

It is the space between two adjacent floors.

4.23 4.22 – Storey Drift

It is the relative displacement of one level relative to the other level above or below between the floors above and below the storey under consideration.

4.24 4.23 – Storey Shear (V_i)

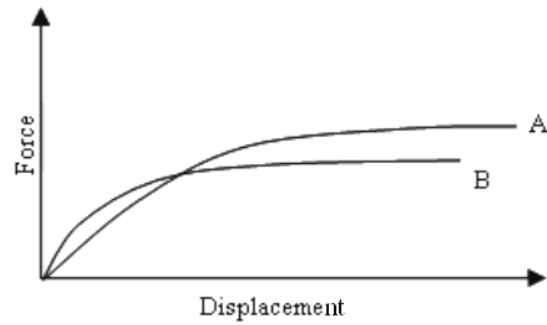
It is the sum of design lateral forces at all levels above the storey i under consideration.

4.25 4.24 – Weak Storey

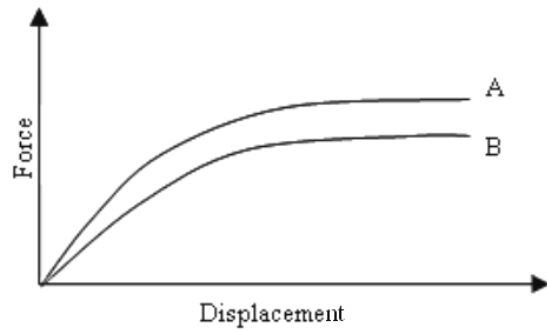
It is one in which the storey lateral strength is less than ~~80-70~~ percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

C4.24 – Weak 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 (Figure C 4). Soft storey refers to stiffness and weak storey refers to strength. Usually, a soft storey may also be a weak storey.

CODE**COMMENTARY**

Structure A has higher strength and lower stiffness as compared to structure B



Structure A has higher strength and higher stiffness as compared to structure B

Figure C 4 – Stiffness versus Strength

CODE**COMMENTARY****5. – Symbols**

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

A_h	Design horizontal seismic coefficient
A_k	Design horizontal acceleration spectrum value for mode k of vibration
b_i	i^{th} Floor plan dimension of the building perpendicular to the direction of force
c	Index for the closely-spaced modes
D	Base dimension of the building, in metres, in the direction in which the seismic force is considered.
DL	Response quantity due to dead load
e_{di}	Design eccentricity to be used at floor i calculated as per 7.8.2
e_{si}	Static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity
EL_x	Response quantity due to earthquake load for horizontal shaking along x-direction
EL_y	Response quantity due to earthquake load for horizontal shaking along y-direction
EL_z	Response quantity due to earthquake load for vertical shaking along z-direction
F_{roof}	Design lateral forces at the roof due to all modes considered
F_i	Design lateral forces at the floor i due to all modes considered
g	Acceleration due to gravity
h	Height of structure, in metres
h_i	Height measured from the base of the building to floor i
I	Importance factor
IL	Response quantity due to imposed load
M_k	Modal mass of mode k
n	Number of storeys
N	SPT value for soil
P_k	Modal participation factor of mode k Lateral force at floor i
Q_i	Lateral force at floor i
Q_{ik}	Design lateral force at floor i in mode k
r	Number of modes to be considered as per 7.8.4.2

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R	Response reduction factor
S_a/g	Average Response acceleration coefficient for rock or soil sites as given by Figure 2-3 and Table 3 based on appropriate natural periods and damping of the structure
T	Undamped natural period of vibration of the structure (in second)
T_a	Approximate fundamental period (in seconds)
T_k	Undamped natural period of mode k of vibration (in second)
T_1	Fundamental natural period of vibration (in second)
V_B	Design seismic base shear
$\bar{V}_B$	Design base shear calculated using the approximate fundamental period T_a
V_i	Peak storey shear force in storey i due to all modes considered
V_{ik}	Shear force in storey i in mode k
V_{roof}	Peak storey shear force at-in the roof-top <u>storey</u> due to all modes considered
W	Seismic weight of the structure
W_i	Seismic weight of floor i
Z	Zone factor
Φ_{ik}	Mode shape coefficient at floor i in mode k
λ	Peak response (for example member forces, displacements, storey forces, storey shears or base reactions) due to all modes considered
λ_k	Absolute value of maximum response in mode k
λ_c	Absolute value of maximum response in mode c , where mode c is a closely-spaced mode.
λ^*	Peak response due to the closely-spaced modes only
ρ_{ji}	Coefficient used in the Complete Quadratic Combination (CQC) method while combining responses of modes i and j
ω_i	Circular frequency in rad/second in the i^{th} mode

CODE**COMMENTARY****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, etc) of seismic ground vibrations ~~expected~~ at any location depends ~~upon the~~ magnitude of earthquake, its depth of focus, distance from the epicenter, characteristics of the path through which the seismic waves travel, and ~~the~~ soil strata on which the structure stands. The random earthquake ground motions, which cause the structure to vibrate, can be resolved in any three mutually perpendicular directions. The predominant direction of ground vibration is usually horizontal.

Earthquake-generated vertical inertia forces are to be considered in design ~~wherever appropriate. unless checked and proven by specimen calculations to be not significant.~~ Vertical acceleration should be considered in structures with large spans, those in which stability is a criterion for design, or for overall stability analysis of structures. Reduction in gravity force due to vertical component of ground motions can be particularly detrimental in cases of prestressed horizontal members and of cantilevered members. Hence, special attention should be paid to the effect of vertical component of the ground motion on prestressed or cantilevered beams, girders and slabs.

6.1.2 –

The response of a structure of ground vibrations is a function of the nature of foundation soil; materials, form, size and mode of construction of structures; and the duration and characteristics of ground motion. This standard specifies design forces for structures ~~standing-founded~~ on rocks or soils which do not settle, liquefy or slide due to loss of strength during ~~earthquake~~ ground vibrations.

C6.1.1 – Ground Motion

All structures experience a constant vertical acceleration (downward) equal to gravity (g) at all times. Hence, the vertical acceleration during ground shaking can be just added or subtracted to the gravity depending on the direction of ground motion at that instant. For instance, consider a roof accelerating up and down with $0.20g$. It implies that it is experiencing acceleration in the range $1.20g$ to $0.80g$ in place of $1.0g$ that it would experience without an earthquake. The factor of safety for gravity loads, e.g., dead and live loads, is usually sufficient to cover the earthquake induced vertical accelerations. Thus, safety during horizontal acceleration is the main concern in seismic design of normal structures.

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

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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—(~~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 greater than the design forces specified in this standard. However, ductility, arising from inelastic material behaviour and detailing, and overstrength, arising from the additional reserve strength in structures over and above the design strength, are relied upon to account for this difference in actual and design lateral loads.

Reinforced and prestressed concrete members shall be suitably designed to ensure that premature failure due to shear or bond does not occur, subject to the provisions of **IS 456** and **IS 1343**. Provisions for appropriate ductile detailing of reinforced concrete members are given in **IS 13920**.

In steel structures, members and their connections should be so proportioned that high ductility is obtained, vide **SP 6** (Part 6), avoiding Premature failure due to elastic or inelastic buckling of any type.

The specified earthquake loads are based upon post-elastic energy dissipation in the structure and because of this fact, the provision of this standard for design, detailing and construction shall be satisfied even for structures and members for which load combinations that do not contain the earthquake effect indicate larger demands than combinations including earthquake.

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.nicee.org/EOTips/EOTip08.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.

The earthquake resistant structures should generally be ductile. IS13920: 1993 deals with the ductile detailing requirements for reinforced concrete structures. Ductile detailing provisions for steel structures are not yet available in the Indian codes. 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 for ductile detailing of steel structures until similar Indian codal provisions are developed.

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.

Clause 6.1.3 implies that DBE relates to the “moderate shaking” and MCE relates to the

CODE**COMMENTARY****6.1.4 – Soil Structure Interaction**

The soil-structure interaction refers to the effects ~~if—of~~ the supporting foundation medium on the motion of structure. The soil-structure interaction may not be considered in the seismic analysis for structures supported on rock or rock-like material.

6.1.5 –

~~The design lateral force specified in this standard shall be considered in each of the two orthogonal horizontal directions of the structure.~~ For structures which have lateral force resisting elements in the two orthogonal

“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.

C6.1.4 – 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 provisions for seismic regulations (FEMA 368), which provide 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.

C6.1.5 –

See commentary of clause 6.3.2.1.

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directions only, the design lateral force shall be considered along one direction at a time, and not in both directions simultaneously. Structures, having lateral force resisting elements (for example frames, shear walls) in directions other than the two orthogonal directions, shall be analyzed considering the load combinations specified in **6.3.2**.

Where both horizontal and vertical seismic forces are taken into account, load combination specified in **6.3.3** shall be considered.

6.1.6 –

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

C6.1.6 –

See clause 3.12.

6.1.7 – Additions to Existing Structures

Additions shall be made to existing structures only as follows:

- a) An addition that is structurally independent from an existing structure shall be designed and constructed in accordance with the seismic requirements for new structures.
- b) An addition that is not structurally independent from 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. The addition shall comply with the requirements for new structures,
 2. The addition shall not increase the seismic forces in any ~~structure~~ structural elements of the existing structures by more than 5 percent unless the capacity of the element subject to the increased force is still in compliance with ~~the~~ this standard,

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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 (*I*), the structure shall conform to the seismic requirements 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 causes ~~impulsive~~ ground motions, which are complex and irregular in character, changing in period and amplitude each lasting for a small duration. Therefore, resonance of the type as visualized under steady-state sinusoidal excitations, will not occur as it would need time to build up such amplitudes.

NOTE- However, 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, ~~or~~ maximum flood or maximum sea waves.

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 400kms 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.

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- c) The value of elastic modulus of materials, wherever required, may be taken as for static analysis unless a more definite value is available for use in ~~such~~ dynamic conditions (see IS 456, IS 1343 and IS 800)

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. Modulus of elasticity of masonry has even larger variation than that for concrete. Hence, the code simply allows the modulus of elasticity for static analysis to be used for earthquake analysis also.

6.3 – Load Combinations and Increase in Permissible Stresses.

6.3.1 – Load Combinations

When earthquake forces are considered on a structure, these shall be combined as per **6.3.1.1** and **6.3.1.2** where the terms DL , IL and EL stand for the response quantities due to dead load, imposed load and designated earthquake load respectively.

6.3.1.1 – Load factors for plastic design of steel structures

In the plastic design of steel structures, the following load combination shall be accounted for:

1. $1.7 (DL+IL)$
2. $1.7 (DL \pm EL)$
3. $1.3 (DL+IL \pm EL)$

~~3.4.0.9~~ $DL \pm 1.7 EL$

C6.3.1.1 -

Since the horizontal loads are reversible in direction, in some cases design is governed by effect of horizontal load minus effect of gravity loads. In such situations, a load factor higher than 1.0 for gravity loads will make the calculations unconservative. Hence, a load factor of 0.9 is specified on gravity loads in the fourth combination. Many designs of footings, columns, and positive steel in beams at the ends in frame structures are governed by this load combination.

6.3.1.2 – Partial safety factors for limit state design of reinforced concrete and prestressed concrete structures.

In the limit state design of reinforced and prestressed concrete structures, the following load combinations shall be accounted for:

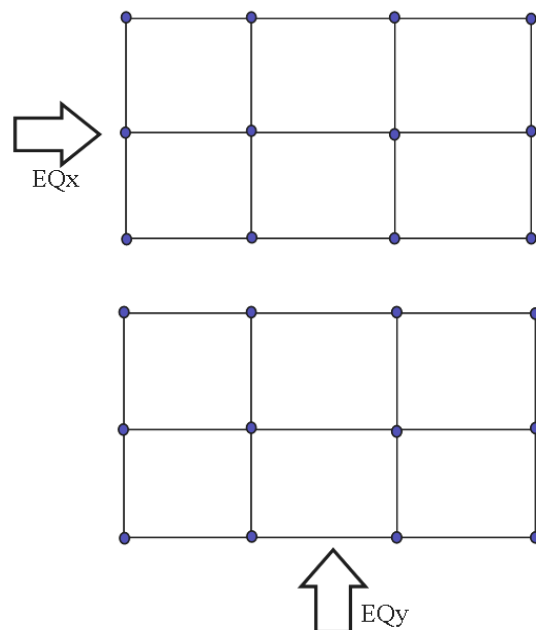
~~1.5~~ $1.5 (DL+IL)$

CODE**COMMENTARY**~~e)2.~~ 1.2(DL+IL ± EL)~~f)3.~~ 1.5(DL ± EL)~~g)4.~~ 0.9 DL ± 1.5 EL**6.3.2 – Design Horizontal Earthquake Load****6.3.2.1 –**

When the lateral load resisting elements are oriented along orthogonal horizontal direction, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction at a time.

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 or walls oriented in two orthogonal directions, say X and Y (see Figure C 5). 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 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.

**Figure C 5 –Earthquake loading in both**

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directions

The load EL in clause 6.3.1 implies earthquake load in $+X$, $-X$, $+Y$ and $-Y$ directions. A reinforced concrete building with orthogonal system therefore needs to be designed for the following 13 load combinations:

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

where

EL_x = Design earthquake load in X direction and,
 EL_y = Design earthquake load in Y direction.

6.3.2.2 –

When the lateral load resisting elements are not oriented along the orthogonal horizontal directions or, when the building is torsionally irregular (as per clause 7.1) about both the horizontal axes, the structure shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30 percent of the design earthquake load in the other horizontal direction.

NOTE- For instance, the building should be designed for $(\pm EL_x \pm 0.3EL_y)$ as well as $(\pm 0.3EL_x \pm EL_y)$, where x and y are two orthogonal horizontal directions, EL in 6.3.1.1 and 6.3.1.2 shall be replaced by $(EL_x \pm 0.3EL_y)$ or $(EL_y \pm 0.3EL_x)$.

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 C 6):

- 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 analyzed as per the

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“100% + 30% rule” described above.

The directions of earthquake forces are reversible. Hence, all combinations of directions are to be considered. Thus, *EL* in 6.3.1.1 and 6.3.1.2 now implies following eight possibilities:

$+(EL_x + 0.3 EL_y)$	$+(0.3 EL_x + EL_y)$
$+(EL_x - 0.3 EL_y)$	$+(0.3 EL_x - EL_y)$
$-(EL_x + 0.3 EL_y)$	$-(0.3 EL_x + EL_y)$
$-(EL_x - 0.3 EL_y)$	$-(0.3 EL_x - EL_y)$

Therefore, the total design load combinations will be 25.

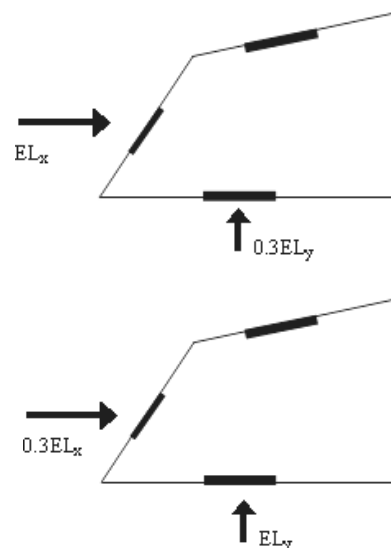


Figure C 6 – 100% + 30% rule for non-orthogonal systems

The 100%+30% rule can be used for buildings with orthogonal lateral load resisting systems also. In particular, the corner columns of such buildings are governed by the 100%+30% rule. However, 6.3.2.1 does not require the 100%+30% rule for corner columns just to save the design effort.

6.3.3 – Design Vertical Earthquake Load

When effects due to vertical earthquake loads are to be considered, the design vertical force shall be calculated [for vertical](#)

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ground motion as detailed in ~~accordance with~~
6.4.5.

6.3.4 – Combination for Two or Three Component Motion

6.3.4.1 –

When responses from the three earthquake components are to be considered, the responses due to each component may be combined using the assumption that when the maximum response from one component occurs, the responses from the other two components are 30 percent of their maximum. All possible combinations of the three components (EL_x , EL_y and EL_z) including variations in sign (plus or minus) shall be considered. Thus, the response due earthquake force (EL) is the maximum ~~of~~ response from the following three sets of load cases:

1. $\pm EL_x \pm 0.3EL_y \pm 0.3EL_z$
2. $\pm EL_y \pm 0.3EL_x \pm 0.3EL_z$
3. $\pm EL_z \pm 0.3EL_x \pm 0.3EL_y$

where x and y are two orthogonal directions and z is vertical direction.

6.3.4.2 –

As an alternative to the procedure in 6.3.4.1, the response (EL) due to the combined effect of the three components can be obtained on the basis of 'square root of the sum of the square (SRSS)' that is

$$EL = \sqrt{(EL_x)^2 + (EL_y)^2 + (EL_z)^2}$$

NOTE – The combination procedure of 6.3.4.1 and 6.3.4.2 apply to the same response quantity (say, moment in a column about its major axis, or storey shear in a frame) due to different components of the ground motion.

6.3.4.3 –

When two component motions (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

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 as per "100%+30% rule". Then, EL in 6.3.1.1 and 6.3.1.2 means the following 24 combinations:

- (a) $\pm EL_x \pm 0.3EL_y \pm 0.3EL_z$ (8 Combinations)
- (b) $\pm EL_y \pm 0.3EL_x \pm 0.3EL_z$ (8 Combinations)
- (c) $\pm EL_z \pm 0.3EL_x \pm 0.3EL_y$ (8 Combinations)

Therefore, the total design load combinations will be 73.

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.

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considered.

6.3.5 – Increase in Permissible Stresses**6.3.5.1 – Increase in Permissible Stress in Materials**

When earthquake forces are considered along with other normal design forces, the permissible stresses in material, in the elastic method of design, may be increased by one-third. However, for steels having a definite yield stress, the stress be limited to the yield stress; for steels without a definite yield point, the stress will be limited to 80 percent of the ultimate strength or 0.2 percent proof stress, whichever is smaller; and that in prestressed concrete members, the tensile stress in the extreme fibers of the concrete may be permitted so as not to exceed two-thirds of the modulus of rupture of concrete.

6.3.5.2 – Increase in allowable pressure in Soils

When earthquake forces are included, the allowable bearing pressure in soils shall be increased as per Table 1, depending upon type of foundation of the structure and the type of soil.

In soil deposits consisting of submerged loose sands and soils falling under classification SP with standard penetration N -values less than 15 in seismic Zones III, IV, V and less than 10 in seismic Zone II, the vibration caused by earthquake may cause liquefaction or excessive total and differential settlements. Such sites should preferably be avoided while locating new settlements or important projects. Otherwise, this aspect of the problem needs to be investigated and appropriate methods of compaction or stabilization adopted to achieve suitable N -values as indicated in Note 3 under Table 1. Alternatively, deep pile foundation may be provided and taken to depths well into the layer which is not likely to liquefy. Marine clays and other sensitive clays are also known to liquefy due to collapse of soil structure and will need special treatment according to site condition.

NOTE- Specialist literature may be referred

C6.3.5 – Increase in Permissible Stress**C6.3.5.1 – 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.

C6.3.5.2 – Increase in allowable pressure in 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

It is advisable to use the average properties in the top 30m rather than just at the founding level.

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.

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for determining liquefaction potential of a site.
[One method for evaluation of liquefaction potential is given in Annex F.](#)

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Table 1 - Percentage of Permissible Increase in Allowable Bearing Pressure or Resistance of Soils

(Clause 6.3.5.2)

SI No.	Foundation	Type of soil Mainly Constituting the Foundation		
		Type I Rock or Hard Soil: Well graded gravel and sand gravel mixtures with or without clay binder, and clayey sands poorly graded or sand clay mixtures (GB, CW, SB, SW, and SC) ^{††} Having N ² above 30, where N is the standard penetration value	Type II Medium-Stiff Soil: All soils with N between 10 and 30, and poorly graded sands or gravelly sands with little or no fines (SP ^{††}) with N > 15	Type III Soft Soils: All soils other than SP ^{††} with N < 10
(1)	(2)	(3)	(4)	(5)
i)	Piles passing through any soil but resting on soil type I	50	50	50 --
ii)	Piles not covered under item i	--	25	25 --
iii)	Raft foundations	50	50	50 --
iv)	Combined isolated RCC footing with tie beams	50	25	25 --
v)	Isolated RCC footing without tie beams, or unreinforced strip foundations.	50	25	--
vi)	Well foundation	50	25	25 --

NOTES

- The allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.
- If any increase in bearing pressure has already been permitted for forces other than seismic forces, the total increase in allowable bearing pressure when seismic force is also included shall not exceed the limits specified above.
- Desirable minimum field values of N- If soils of smaller N-values are met, compacting may be adopted to achieve these values or deep pile foundations going to stronger strata should be used.

Seismic Zone Level	Depth below Ground (in meters)	N-Values	Remarks
III, IV and V	≤ 5 ≥ 10	15 20	For values of depths between 5m and 10m, linear interpolation is recommended.
II (for important Structures only)	≤ 5 ≥ 10	15 20	

- The values of N (uncorrected values) are at the founding level and the allowable bearing pressure shall be determined in accordance with IS 6403 or IS 1888.

Seismic Zone Level (in meters)	Depth Below Ground	N-Values	Remark
III, IV and V	≤ 5 ≥ 10	15 25	For values of depths between 5 m and 10 m, linear interpolation is recommended
II (for important Structure only)	≤ 5 ≥ 10	15 20	

- The piles should be designed for lateral loads neglecting lateral resistance of soil layers liable to liquefy.
- IS 1498 and IS 2131 may also be referred.

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~~7. Isolated R.C.C. footing without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with $N < 10$.~~

7. The soils are classified as follows:

<u>Soil Type</u>	<u>Definition</u>
<u>Type I: Rock or Hard Soils</u>	<ul style="list-style-type: none"> • <u>Well graded gravel (GW) or well graded sand (SW) both with less than 5% passing 75 μm sieve (Fines);</u> • <u>Well graded Gravel – Sand mixtures with or without fines (GW-SW);</u> • <u>Poorly graded Sand (SP) or clayey sand (SC), all having N above 30;</u> • <u>Stiff to hard clays having N above 16, where N is the Standard Penetration Test value.</u>
<u>Type II: Stiff Soils</u>	<ul style="list-style-type: none"> • <u>Poorly graded sands or Poorly graded sands with gravel (SP) with little or no fines having N between 10 and 30;</u> • <u>Stiff to medium stiff fine-grained soils, like Silts of Low compressibility (ML) or Clays of Low Compressibility (CL) having N between 10 and 16.</u>
<u>Type III: Soft Soils</u>	<p><u>All soft soils other than SP with $N < 10$. The various possible soils are</u></p> <ul style="list-style-type: none"> • <u>Silts of Intermediate compressibility (MI);</u> • <u>Silts of High compressibility (MH);</u> • <u>Clays of Intermediate compressibility (CI);</u> • <u>Clays of High compressibility (CH);</u> • <u>Silts and Clays of Intermediate to High compressibility (MI-MH or CI-CH);</u> • <u>Silt with Clay of Intermediate compressibility (MI-CI);</u> • <u>Silt with Clay of High compressibility (MH-CH).</u>

See IS 1498 and IS 2131 for soil notation and N value.

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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 C 7. 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 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.

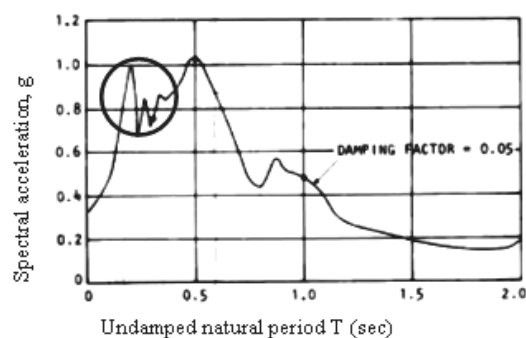


Figure C 7 –Acceleration Response Spectrum

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

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can be lowered if the structure has higher ductility.

6.4.1–

For the purpose of determining seismic forces, the country is classified into four seismic zones as shown in Figure 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 the 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 the following expression:

$$A_h = \frac{ZI(S_a/g)}{2R}$$

~~Provided that for any structure with $T \leq 0.1$ s, the value of A_h will not be taken less than $Z/2$~~

C6.4.2 –**Zone Factor (Z)**

Zone factor (Z) accounts for the expected intensity of shaking in different seismic zones (Table 2, IS 1893:2002). Efforts have been made to specify Z values that represent a reasonable estimate of PGA in the respective zone. For instance, Z value of 0.36 in zone V implies that a value of 0.36g is reasonably expected in zone V. But it does not imply that acceleration in zone V will not exceed 0.36g. For example, during 2001

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~~whatever be the value of I/R~~

Where

Z = Zone factor given in Table 2, is for the Maximum Considered Earthquake (MCE) ~~and service life of structure in a zone~~. The factor 2 in the denominator of Z is used so as to reduce the Maximum Considered Earthquake (MCE) zone factor to the factor for Design Basis Earthquake (DBE).

I = Importance factor, depending upon the functional use of the structures, characterized by hazardous consequences of its failure, post-earthquake functional needs, historical value, or economic importance (Table 6).

R = Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformations. ~~However, the ratio (I/R) shall not be greater than 1.0 (Table 7).~~ The values of R for buildings are given in Table 7.

S_a/g = ~~Average~~ ~~r~~ Response acceleration coefficient. ~~For rock or soil~~ as given by Figure 32 and Table 3 based on appropriate natural periods and damping of the structure. These curves represent free field ground motion.

For rocky, or hard soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15T; & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.40 \\ 1.00/T & 0.40 \leq T \leq 4.00 \end{cases}$$

$$\frac{S_a}{g} = \begin{cases} 2.50 & 0.00 \leq T \leq 0.10 \text{ (for fundamental mode)} \\ 1+15T & 0.00 \leq T \leq 0.10 \text{ (for higher modes)} \\ 2.50 & 0.10 \leq T \leq 0.40 \\ 1.00/T & 0.40 \leq T \leq 4.00 \\ 0.25 & T > 4.0 \end{cases}$$

For ~~medium stiff~~ soil sites

$$\frac{S_a}{g} = \begin{cases} 1+15T; & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.55 \\ 1.36/T & 0.55 \leq T \leq 4.00 \end{cases}$$

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Bhuj earthquake, peak ground acceleration of approximately 0.6g was inferred from data obtained from the Structural Response Recorder located at Anjar, 44kms away from the epicenter.

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. Importance factor is meant to account for this by increasing the design force level for critical and important structures.

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. Now, the code provides for realistic force for elastic structure and then divides that force by ($2R$). This gives the designer a realistic picture of the design philosophy. A building is expected to undergo damage in case of strong shaking and therefore should be detailed for ductility.

Consider a building in zone V and designed as per the code. $Z=0.36g$ gives a realistic indication of ground acceleration. For $T=0.3\text{sec}$, $S_a/g=2.5$ (Figure 2, IS 1893: 2002), which implies that if the building remains elastic, it may experience a maximum horizontal force equal to 90% of its weight ($0.36 \times 2.5 = 0.90$). 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 designer knows that he is designing for only one tenth of the maximum elastic force, and hence, should provide adequate ductility and quality control for good post yield behaviour.

Over strength, redundancy and ductility together contribute to the fact that an earthquake resistant structure can be designed for much lower force than is implied by the strong shaking (Figure C 8).

a) Over strength – 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

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$\frac{S_a}{g} = \begin{cases} 2.50 & 0.00 \leq T \leq 0.10 \text{ (for fundamental mode)} \\ 1+15T & 0.00 \leq T \leq 0.10 \text{ (for higher modes)} \\ 2.50 & 0.10 \leq T \leq 0.55 \\ 1.36/T & 0.55 \leq T \leq 4.00 \\ 0.34 & T > 4.0 \end{cases}$	<ul style="list-style-type: none">Partial safety factor on materials										
For soft soil sites	<ol style="list-style-type: none">Material Properties<ul style="list-style-type: none">Member size or reinforcement larger than requiredStrain hardening in materialsConfinement of concrete improves its strengthHigher material strength under cyclic loadsStrength contribution of non-structural elementsSpecial ductile detailing which also adds to strength										
$\frac{S_a}{g} = \begin{cases} 2.50 & 0.00 \leq T \leq 0.10 \text{ (for fundamental mode)} \\ 1+15T & 0.00 \leq T \leq 0.10 \text{ (for higher modes)} \\ 2.50 & 0.10 \leq T \leq 0.67 \\ 1.67/T & 0.67 \leq T \leq 4.00 \\ 0.42 & T > 4.00 \end{cases}$	<ol style="list-style-type: none">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 “over strength” term.Ductility – As the structure yields, two things happen:<ol style="list-style-type: none">There is more energy dissipation in the structure due to <u>hysteresis</u>.The structure becomes softer and its natural period increases, which usually implies that the structure has to now resist a lower seismic force. <p>Higher ductility indicates that the structure can withstand stronger shaking without collapse.</p>										
<p>NOTE – For various types of structures, the values of Importance Factor <i>I</i>, Response Reduction Factor <i>R</i>, and damping values are given in the respective parts of this standard. The method (empirical or otherwise) to calculate the natural periods of the structure to be adopted for evaluating <i>S_a/g</i> is also given in the respective parts of this standard.</p> <p>Table 1 - Zone Factor, <i>Z</i> (Clause 6.4.2)</p> <table><tr><th>Seismic Zone</th><th>II</th><th>III</th><th>IV</th><th>V</th></tr><tr><td><i>Z</i></td><td>0.10</td><td>0.16</td><td>0.24</td><td>0.36</td></tr></table>	Seismic Zone	II	III	IV	V	<i>Z</i>	0.10	0.16	0.24	0.36	
Seismic Zone	II	III	IV	V							
<i>Z</i>	0.10	0.16	0.24	0.36							

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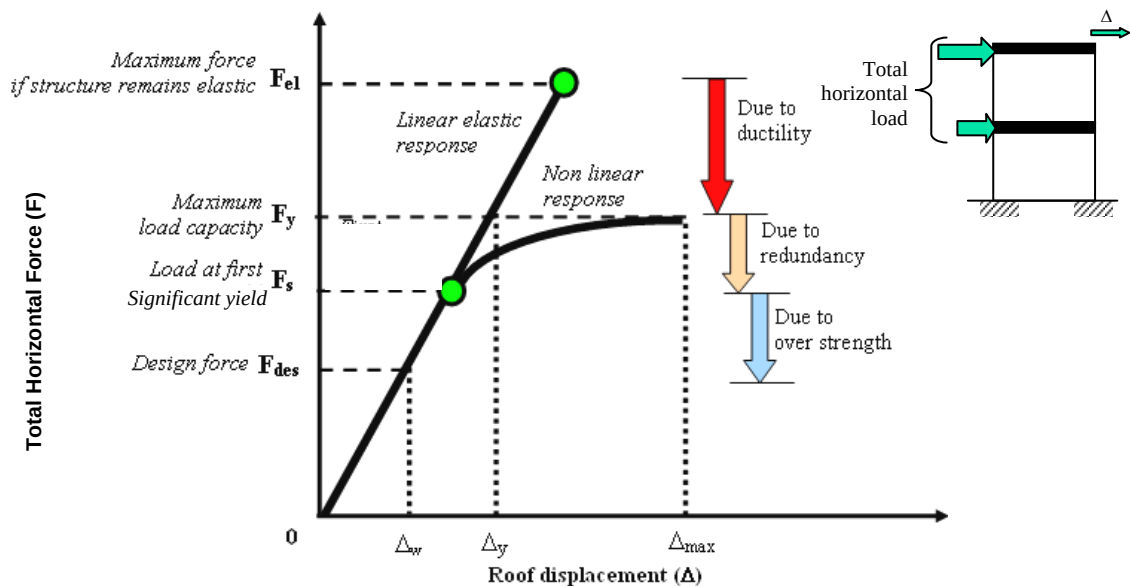


Figure C 8- Concept of Response Reduction Factor

Response Acceleration Coefficient

For very stiff structures (i.e., natural period for first mode $< 0.1\text{sec}$), ductility is not helpful in reducing the design force. Further, structures falling on the rising arm of the spectra (i.e., those with $T < 0.1\text{s}$) 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. The second paragraph of clause 6.4.2 in 2002 edition of the code attempted to ensure a minimum design force for stiff structures. However, there are difficulties with this restriction and hence, to address this issue, the graphs and the equations giving the values for response acceleration coefficient (S_a/g) have been modified in this revision of the code such that the rising part of S_a/g plot between zero and 0.1 sec cannot be used for the fundamental modes of vibration.

Soil Effect

Recorded earthquake motions show that response spectrum shape varies with the soil profile at the site (Figure C 9).

This variation in ground motion characteristics for different sites is accounted for by providing different shapes of response spectrum for each of

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the sites (Figure 9). The soil types I, II and III have been defined in Table 1 of the code.

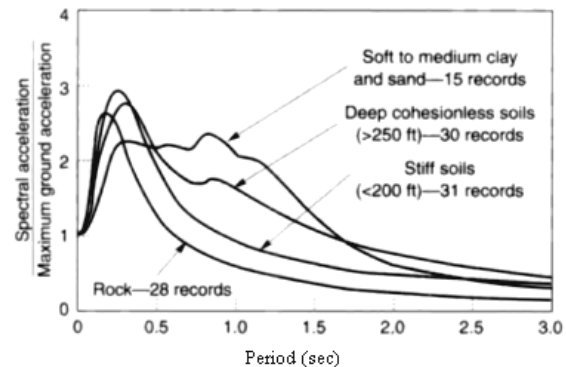


Figure C 9 – Recorded earthquake motions for different types of soil sites (From Geotechnical Earthquake Engineering by Kramer, 1996)

Damping Factors

The response spectrum value at zero period is equal to peak ground acceleration (see commentary of clause C3.11) regardless of damping. The design acceleration spectrum given in Figure 3 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 3. 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 $T \geq 0.1\text{sec}$ only. For $T = 0$, multiplication factor will be 1, and values for $0 \leq T < 0.1\text{sec}$ should be interpolated accordingly.

6.4.3 –

Where a number of modes are to be considered for dynamic analysis, the value of A_h as defined in 6.4.2 for each mode shall be determined using the natural period of vibration of that mode.

6.4.4 –

For underground structures and foundations at depths of 30 m or below, the design horizontal acceleration spectrum value shall be taken as half the value obtained from 6.4.2. For structures and foundations placed between the ground level and 30 m depth, the design horizontal acceleration spectrum

C6.4.4 –

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

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value shall be linearly interpolated between A_h and $0.5 A_h$, where A_h is as specified in 6.4.2.

6.4.5 –

The design acceleration spectrum for vertical motions, when required, may be taken as two-thirds of the design horizontal acceleration spectrum specified in 6.4.2.

6.4.6 –

Figure 2-3 shows the ~~proposed~~ 5 percent spectra for ~~rocky and different~~ soils sites and Table 3 gives the multiplying factors for obtaining spectral values for various other damping.

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interpolation is 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.

C6.4.5 –

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. While the 1984 edition of the code specified vertical coefficient as one-half of horizontal, in the 2002 edition peak vertical acceleration has been specified as two-thirds of the peak horizontal acceleration.

C6.4.6 –

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 C 10 for the example case of Type II stiff soil site and 10% damping.

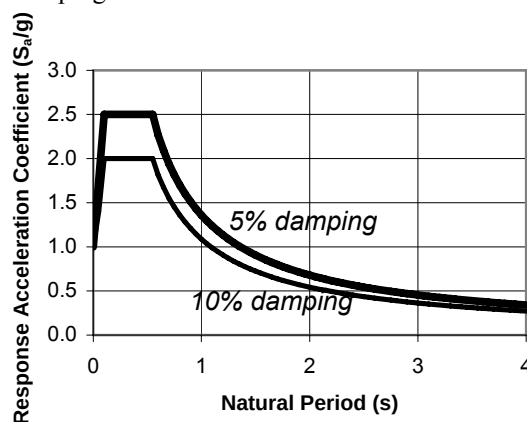


Figure C 10 – Scaling for acceleration spectrum for damping other than 5 %

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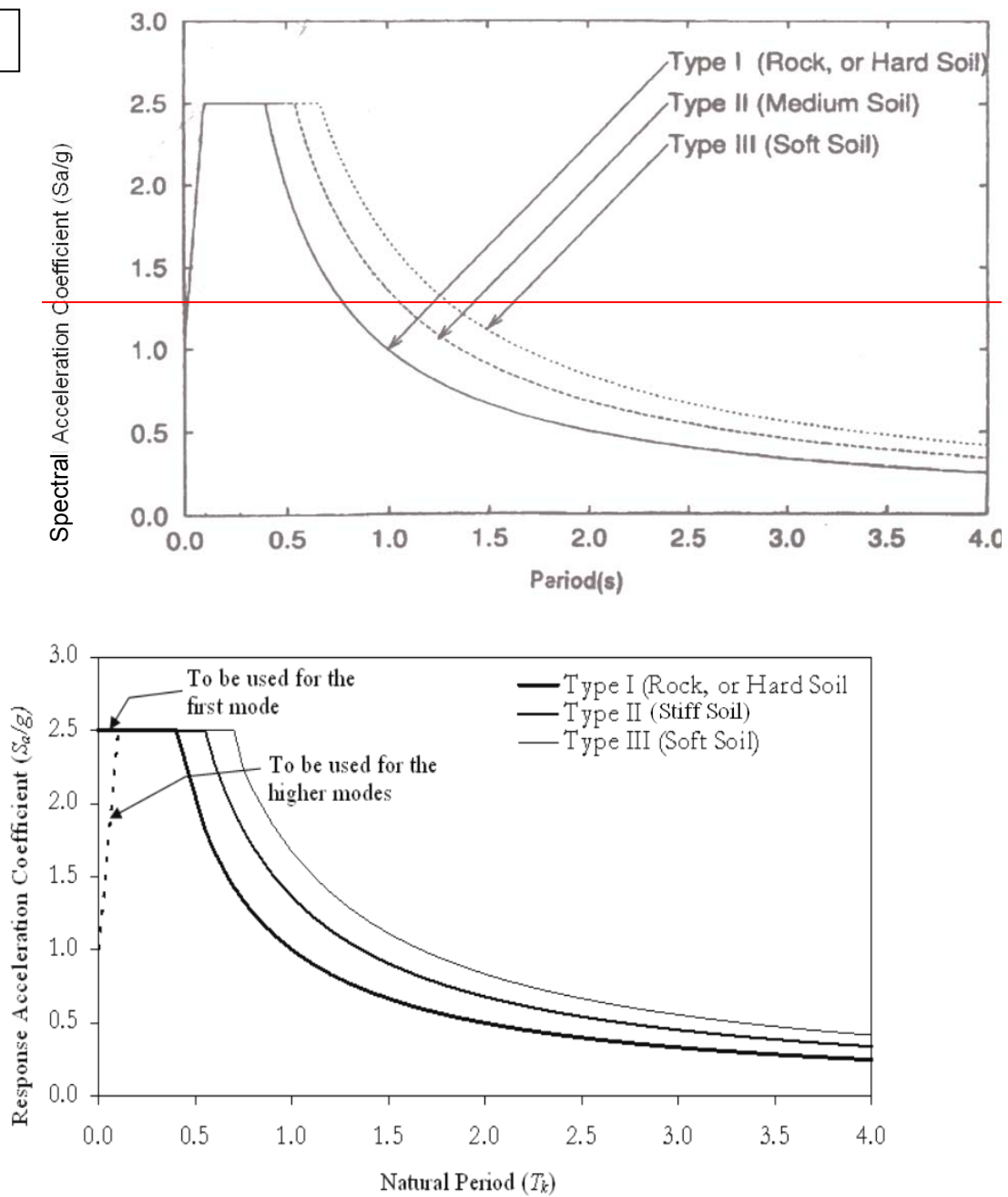


Figure 23 - Response spectra-acceleration coefficient for 5 percent damping

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Table 3 - Multiplying factors for obtaining (S_a/g) values for other damping¹⁾

(Clause 6.4.2)

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

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

6.4.7 –

In case design acceleration spectrum is developed specific to a project site, the same may be used for design of the project as per the discretion of the project authorities.

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.

- 1) Reiter L., *Earthquake Hazard Analysis: Issues and Insights*; Columbia University Press, New York.
- 2) Kramer S.L., *Geotechnical Earthquake Engineering*; Indian Reprint, Pearson Education, New Delhi, 2003.
- 3) Housner, G.W. and Jennings P.C., *Earthquake Design Criteria*; Earthquake Engineering Research Institute, 1982.
- 4) AERB (1990), *Seismic Studies and Design Basis Ground Motion for Nuclear Power Plant Sites*, AERB Safety Guide No. AERB/SG/S-11, Atomic Energy Regulatory Board, India.

CODE**COMMENTARY****7. – Buildings****C7. – Buildings****7.1 – Regular and Irregular Configuration**

To perform well in an earthquake, a building should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan and in elevation, suffer much less damage than buildings with irregular configurations. A building shall be considered as irregular for this standard, if at least one of the conditions given in Tables 4 and 5 is applicable.

**Table 4 – Definition of Irregular Buildings
– Plan irregularity (Fig 34)**

(Clause 7.1)

i) (a) Torsion Irregularity

To be considered when floor diaphragms are rigid in their own ~~plan plane~~ 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 structures transverse to an axis is more than 1.2 times the average of storey drifts at the two ends of the structure.

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

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”.

(b) 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 structures transverse to an axis is more than 1.4 times the average of storey drifts at the two ends of the structure

ii) *Re-entrant Corners*

Plan configurations of a structure and its lateral force resisting system contain re-

Buildings with large re-entrant corners, (i.e., plan shapes such as L, V, +, Y, etc.) show poor performance during earthquakes. Each wing of such a building tends to vibrate as per its own

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entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction.

dynamic characteristic, causing a stress concentration at the junctions of the wings.

iii) *Diaphragm discontinuity*

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next

Diaphragm discontinuity changes the lateral load distribution to different elements as compared to what it would be with rigid floor diaphragm.

iv) *Out-of-Plane Offsets*

Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements

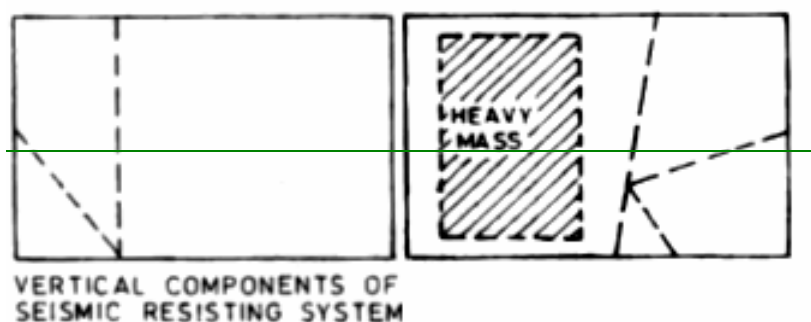
Out-of-Plane offset is a serious irregularity having an out-of-plane offset of the vertical element (for example, shear wall) that carries the lateral loads. Such an offset imposes excessive vertical and lateral load effects on horizontal elements.

v) *Non-Parallel System*

The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting elements.

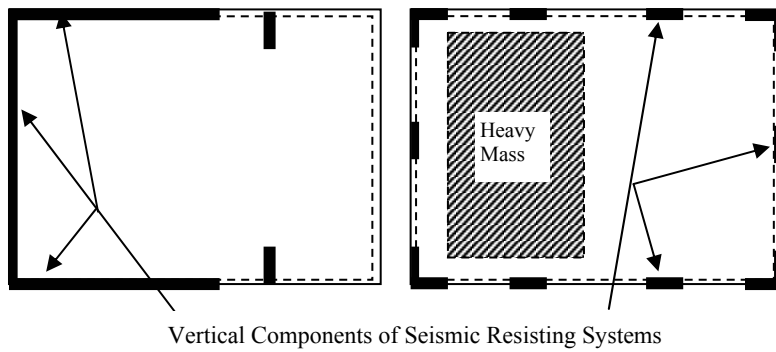
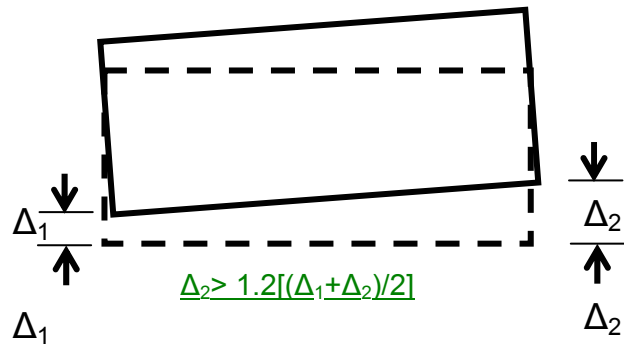
These systems are also known as non-orthogonal systems. See commentary of clause 6.3.2.

* Extreme torsion irregularity (Type i (b)) is not permitted in zones IV and V.



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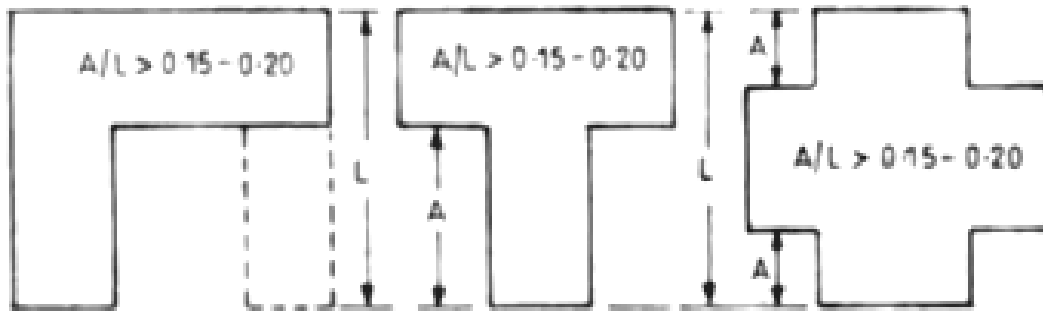
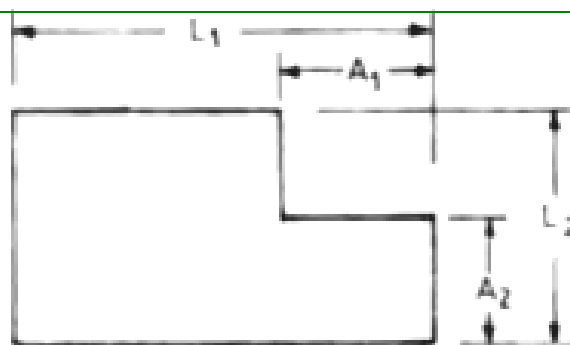
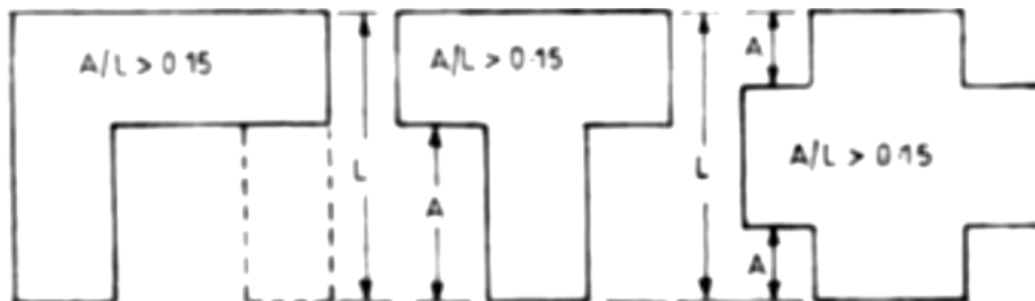
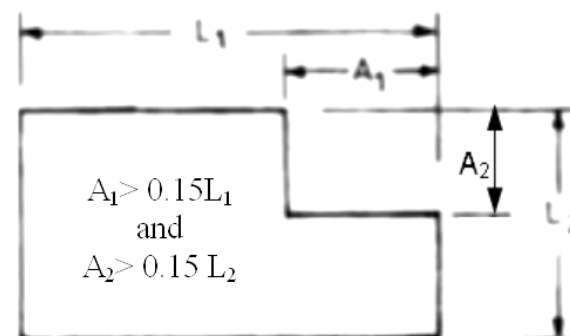
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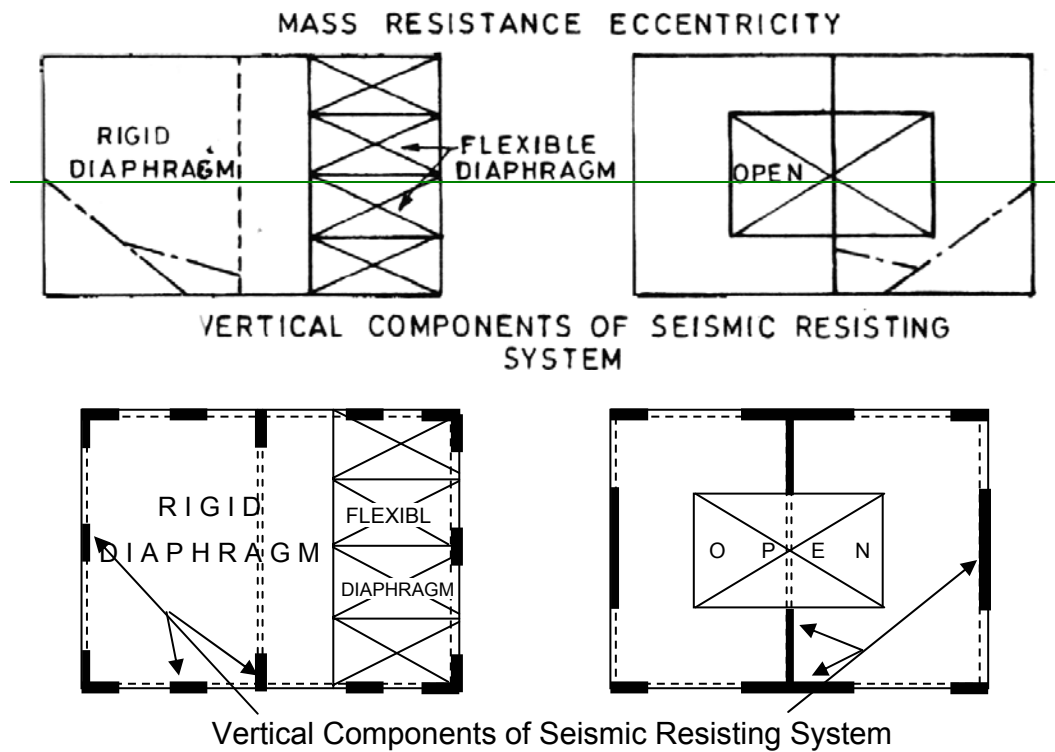
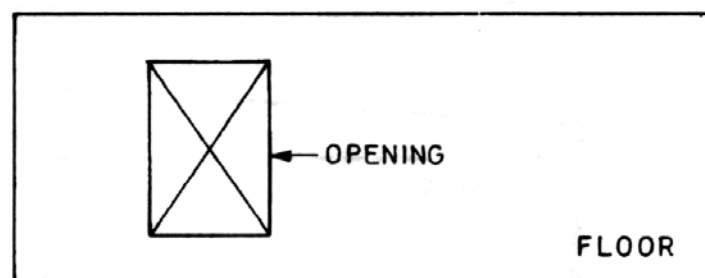
~~3A-4A~~ Torsional Irregularity

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3B-4B Re-entrant Corners

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3C4C Diaphragm Discontinuity

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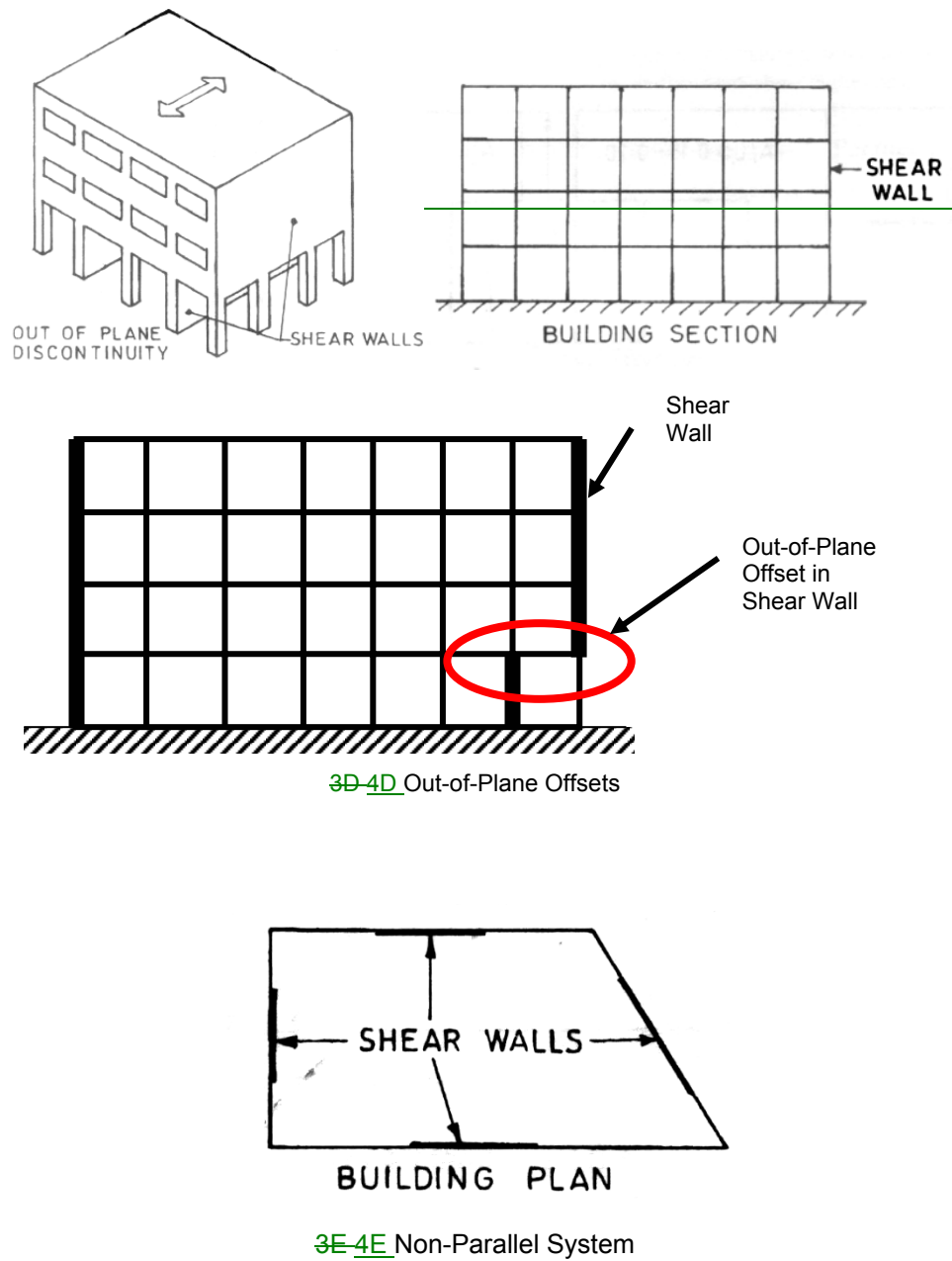


Figure 43 – Plan irregularity

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**Table 5 – Definition of irregular buildings
– Vertical irregularities (Fig . 4)**

(Clause 7.1)

i) ~~(a)~~ *Stiffness Irregularity (Soft Storey)[#]*

A soft storey is one in which the lateral stiffness is less than ~~70-60~~ percent of that in the storey above or less than ~~80%~~70% of the average lateral stiffness of the three storeys above.

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.

~~i) (b) *Stiffness Irregularity (Extreme Soft Storey)*~~

~~A extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category.~~

ii) *Mass Irregularity[#]*

Mass irregularity shall be considered to exist where the seismic weight of any ~~storey floor~~ is more than 200 percent of that of its adjacent ~~storeys floors~~. ~~The irregularity need not be considered. This provision of 200 percent may be relaxed somewhat~~ in case of roofs.

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 a swimming pool.

NEHRP code is more conservative on this issue. It considers a building to be irregular even if a storey is 150 percent heavier than adjacent storeys.

iii) *Vertical Geometric Irregularity*

Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey

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, shear 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

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percent of that in its adjacent storey.

iv) *In-Plane Discontinuity in Vertical Elements Resisting Lateral Force*

A—An in-plane offset of the lateral force resisting elements greater than the length of those elements

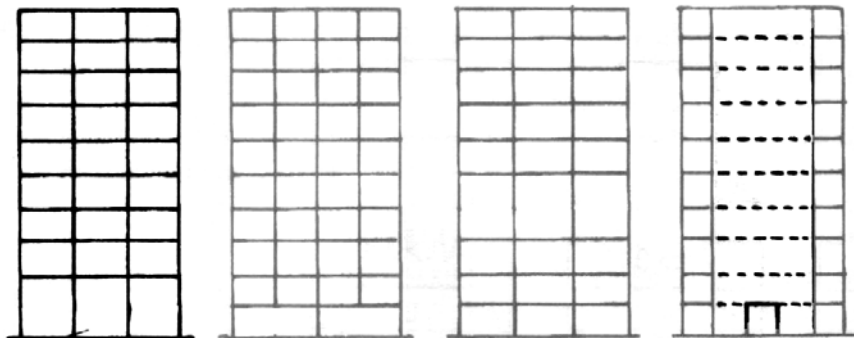
v) *Discontinuity in Capacity (Weak Storey)[±]*

A weak storey is one in which the storey lateral strength is less than ~~80~~70 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

Vertical irregularity of Type (i) and Type (ii) do not apply if the inter-storey drift ratio under design seismic loads is within 130% of the storey drift ratio of the adjacent storey. For this calculation of storey drift, torsional effects need not be considered.

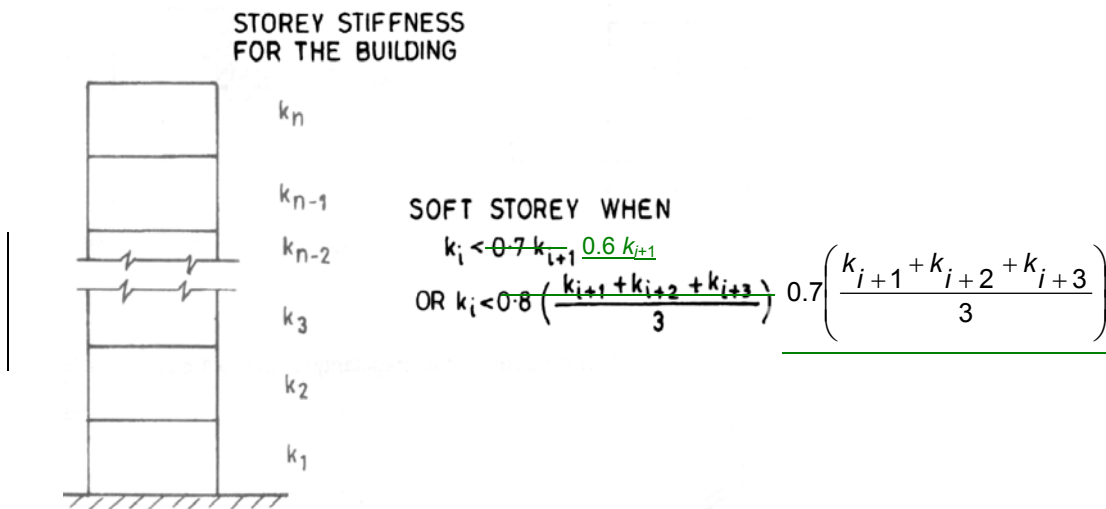
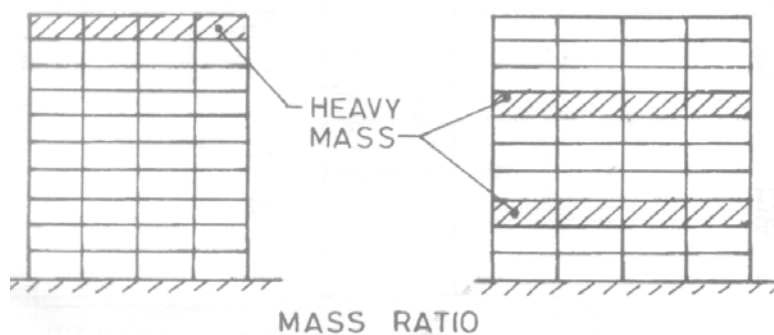
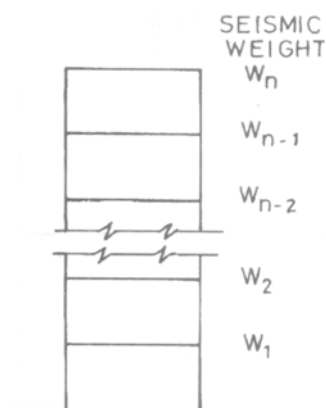
If a floor of a building is comparatively heavier than the adjacent floors, the effect of this irregularity can be nullified by making that storey stiffer in comparison to adjacent storeys. Therefore, if the mass-to-stiffness ratio of two adjacent storeys is similar, the storey drift ratio will be comparable and hence the footnote allows a waiver on this basis.

* Vertical irregularity of Type (V) is not permitted in zones IV and V for more than 2 storey buildings.



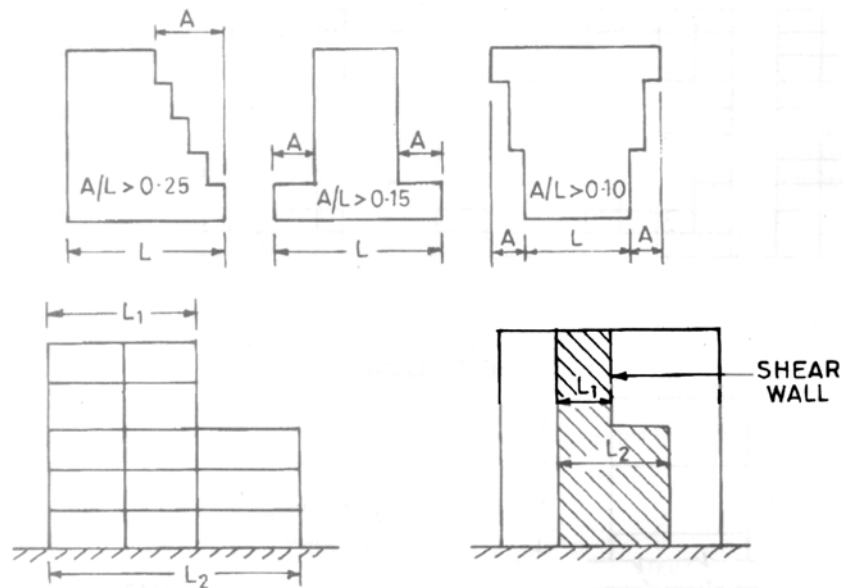
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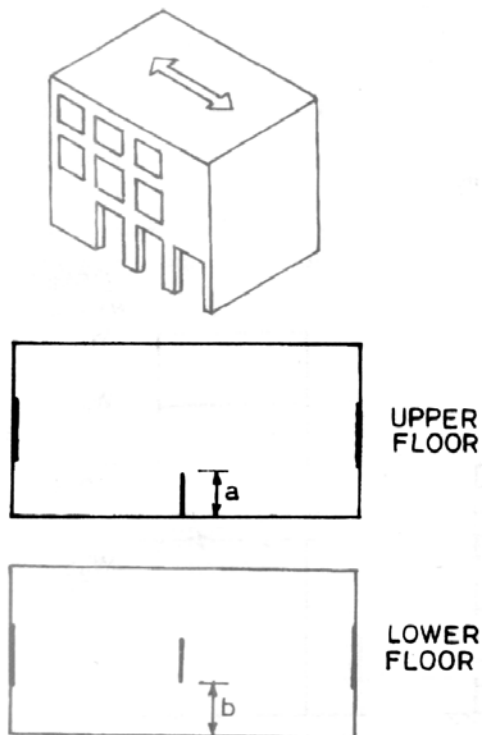
~~4A-5A~~ Stiffness Irregularity

MASS IRREGULARITY
WHEN, $W_i > 2.0 W_{i-1}$
OR $W_i > 2.0 W_{i+1}$

~~4B-5B~~ Mass Irregularity

CODE**COMMENTARY**

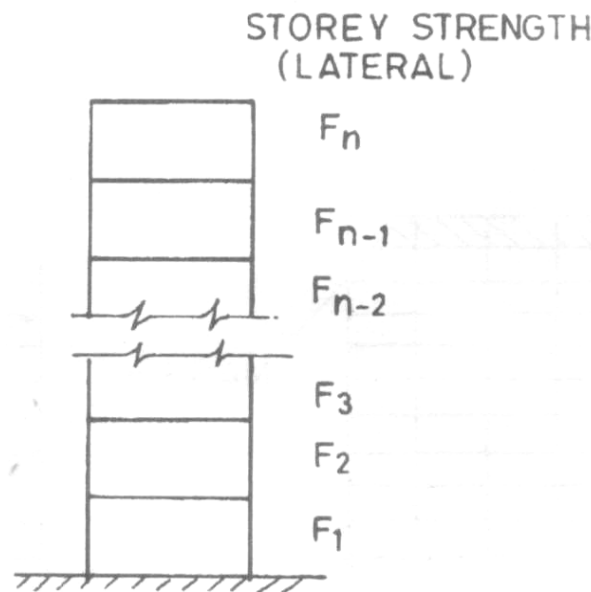
~~4C-5C~~ Vertical Geometric Irregularity when $L_2 > 1.5 L_1$



~~4D-5D~~ In-Plane Discontinuity in Vertical Elements Resisting Lateral Force when $b > a$

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4-5E Weak Storey when $F_i < 0.8 \frac{F_{i+1}}{4}$

Figure 4-5 – Vertical Irregularity

7.2 – Importance Factor I and Response Reduction Factor R

7.2.1 –

The minimum value of importance factor, I , for different building systems shall be as given in Table 6. The response reduction factor, R , for different building systems shall be as given in Table 7.

7.2.2 – Redundancy

Building should have a high degree of redundancy for lateral load resistance. More redundancy in the structure leads to increased level of energy dissipation and more overstrength. The values of response reduction factor (R) given in Table 7 for buildings are based on the assumption that the building has sufficient level of redundancy. The design engineer may adopt the value of R in the range of 0.75 to 0.90 times the values given in Table 7 for buildings with low redundancy, e.g., lateral load resistance provided by only two or three shear walls in a given direction, lateral load resisted by one-bay frames, etc.

C7.2 – Importance Factor I and Response Reduction Factor R

The values of response reduction factor specified in Table 7 have been arrived at empirically based on engineering judgment. The concept of response reduction factor has been discussed in commentary of clause C 6.4.2.

C7.2.2 – Redundancy

Response reduction factors (R) were originally developed assuming that structures possess sufficient level of redundancy. High R values 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. However, due to economic pressures, much less redundant special moment frames with relatively few bays of moment resisting framing supporting large floor and roof areas are being constructed. To provide aesthetics to the buildings and to get more space, buildings have many fewer walls than were once commonly provided in such buildings. Similar observations have been made of other types of construction as

CODE**COMMENTARY****Table 6 – Importance Factors, I**

Sl. No	Structure	I
i)	Important service and community buildings, such as hospitals; schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations; <u>and</u> power stations.	1.5
ii)	All other buildings	1.0

NOTES:

1) The design engineer may choose values of importance factor I greater than those mentioned above.

2) Buildings not covered in Sl No. (i) and (ii) above may be designed for higher value of I , depending on economy, strategy considerations like multi-storey buildings having several residential units.

3) This does not apply to temporary structures like excavations, scaffolding etc of short duration.

4) Importance factor for industrial structures including those containing hazardous materials shall be taken as per IS:1893 (Part 4).

well.

This clause is incorporated in this edition, which will reduce the R values for less redundant structures and should provide greater economic incentive 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, and number of elements per line are all important and will essentially influence the level of redundancy in systems and their reliability.

Table 7 – Response Reduction Factor, R , for Building Systems

Sl No	Lateral Load Resisting System	' R '
<i>Building Frame Systems</i>		
i)	Ordinary RC moment resisting frame (OMRF) ²⁾	3.0
ii)	<u>Intermediate RC moment resisting frame</u>	<u>4.0</u>
iii)	Special RC moment-resisting frame (SMRF) ³⁾	5.0
iv)	Steel frame with	
	a) Concentric braces	4.0
	b) Eccentric braces	5.0

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- ~~iv)~~ v) Steel moment resisting frame 5.0
designed as per SP 6 (6)

Buildings with Shear Walls⁴⁾

- ~~v)~~ vi) Load bearing masonry wall buildings⁵⁾
- a) Unreinforced masonry without special seismic strengthening⁵⁾ 1.5
- b) ~~Reinforced~~ Unreinforced masonry strengthened with horizontal RC bands and vertical bars at corners of rooms and jambs of openings⁶⁾ ~~2.5~~ 2.25
- c) ~~Reinforced with horizontal RC bands and vertical bars at corners of rooms and jambs of openings~~ Ordinary reinforced masonry shear wall⁷⁾ 3.0
- d) Special reinforced masonry shear wall⁸⁾ 4.0
- ~~vi)~~ vii) Ordinary reinforced concrete shear walls⁶⁾ 3.0
- ~~vii)~~ viii) Ductile shear walls⁷⁻⁹⁾ 4.0

Buildings with Dual Systems^{8,10)}

- ~~viii)~~ ix) Ordinary shear wall with OMRF 3.0
- ~~ix)~~ x) Ordinary shear wall with SMRF 4.0
- ~~x)~~ xi) Ductile shear wall with OMRF 4.5
- ~~xi)~~ xii) Ductile shear wall with SMRF 5.0

¹⁾ The values of response reduction factors are to be used for buildings with lateral load resisting elements, and not just for the lateral load resisting elements built in isolation.

²⁾ OMRF are those designed and detailed as per IS 456 or IS 800 but not meeting ductile detailing requirement as per IS 13920 or SP 6(6) respectively.

³⁾ SMRF and IMRF are defined in 4.15.2. and 4.15.3 respectively.

⁴⁾ Buildings with shear walls also include buildings having shear walls and 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'.

⁵⁾ Buildings designed unreinforced as per IS 1905.

~~⁵⁾~~ ⁶⁾ Reinforcement should be as per IS 4326 or designed as unreinforced with minimum reinforcement as per IS 1905.

~~⁶⁾ Prohibited in zone IV and V.~~

⁷⁾ Designed as ordinary reinforced masonry with minimum reinforcement as per IS 1905.

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⁸⁾ Designed as reinforced masonry with special reinforcement for ductility as per IS 1905.

⁷⁹⁾ Ductile shear walls are those designed and detailed as per IS 13920.

⁸¹⁰⁾ Buildings with dual systems consist of shear walls (or braced frames) and moment resisting frames such that:

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

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

Note: Some of the above systems may not be allowed in high seismic zones as per IS 4326 or IS 13920.

7.3 – Design Imposed Loads for Earthquake Force Calculation

7.3.1 –

For various loading classes as specified in IS 875 (Part 2), the earthquake force shall be calculated for the full dead load plus the percentage of imposed load as given in Table 8.

Table 8 – Percentage of Imposed Load to be Considered in Seismic Weight Calculation

(Clause 7.3.1)

Imposed Uniformity Distributed Floor Loads (kN/m ²)	Percentage of Imposed Load
Up to and including 3.0	25
Above 3.0	50

7.3.2 –

For calculation the design seismic forces of the structure, the imposed load on roof need not be considered.

7.3.3 –

The percentage of imposed loads given in 7.3.1 and 7.3.2 shall also be used for 'Whole frame loaded' condition in the load

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.

Earlier the code had permitted an engineer to use "reduced imposed load" when considering both live load and seismic load. For example, in

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~~combinations specified in 6.3.1.1 and 6.3.1.2 where the gravity loads are combined with the earthquake loads [that is, in load combinations (3) in 6.3.1.1, and (2) in 6.3.1.2]. No further reduction in the imposed load will be used as envisaged in IS 875 (Part 2) for number of storeys above the one under consideration or for large spans of beams or floors.~~

buildings with imposed load of 3 kN/m^2 , the combination $1.2(DL+IL+EL)$ effectively became $1.2DL+0.3IL+1.2EL$. This provision is now dropped and the design will now be based on $1.2(DL+IL+EL)$. In other words, even though seismic load is calculated on the basis of seismic weight which includes only 25% of IL, one must consider full design imposed load in different load combinations. This of course, still permits reduction in IL in view of the large floor area or large number of storeys supported by columns or foundations as permitted in IS:875 (Part II).

7.3.3 –

The proportions of imposed load indicated above for calculating the lateral design forces for earthquakes are applicable to average conditions. Where the probable loads at the time of earthquake are more accurately assessed, the designer may alter the proportions indicated or even replace the entire imposed load proportions by the actual assessed load. In such cases, where the imposed load is not assessed as per 7.3.1 and 7.3.2 only that part of imposed load, which possesses mass, shall be considered. Lateral design force for earthquakes shall not be calculated on contribution of impact effects from imposed loads.

7.3.4 –

Other loads apart from those given above (for example snow and permanent equipment) shall be considered as appropriate.

7.4– Seismic Weight**C7.4 – Seismic Weight****7.4.1 – Seismic Weight of Floors**

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 etc.

The seismic weight of each floor is its full dead load plus appropriate amount of imposed load, as specified in 7.3.1 and 7.3.2. While computing the seismic weight of each floor, the weight of columns and walls in any storey shall be ~~equally distributed~~ appropriately apportioned to the floors above and below the storey.

7.4.2 – Seismic Weight of Buildings

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The seismic weight of the whole building is the sum of the seismic weights of all the floors.

7.4.3 –

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

7.5 – Design Lateral Force

7.5.1 –

Buildings and portions thereof shall be designed and constructed, to resist the effects of design lateral force specified in 7.5.3, 7.5.4 as a minimum. However, regardless of the design earthquake or wind forces on a building, it shall have lateral load resisting system capable of carrying a horizontal force not less than 1.5% (one and a half percent) of the seismic weight of the building. This load may be applied at different floor levels in proportion to the seismic weight of the respective floor.

7.5.2 –

The design lateral force shall first be computed for the building as a whole. This design lateral force shall then be distributed to the various floor levels. The overall design seismic force thus obtained at each floor level shall then be distributed to individual lateral load resisting elements depending on the floor diaphragm action.

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C7.5 – Design Lateral Force

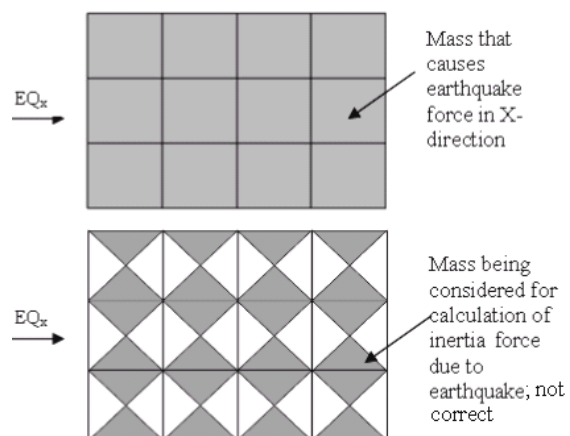
It may be mentioned that the code no longer talks of the two methods: seismic coefficient method and response spectrum method as was the case in 1984 version.

The procedure of clause 7.5 to 7.7 does not require dynamic analysis. Hence, this procedure may be mentioned as *static procedure* or *equivalent static procedure* or *seismic coefficient method*. It can be noticed that this procedure accounts for dynamics of the building in an approximate manner.

In this edition, a new provision of minimum lateral force for seismic design is included. The minimum load is a structural integrity issue related to load path.

C7.5.2 –

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 C 11).



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Figure C 11 - Calculation of design seismic force on the basis of tributary mass on 2D frames leads to significant under-design

Now, clause 7.5.2 makes it clear that one has to evaluate seismic design force for the entire building first and then distribute it to different frames/walls. But that does not mean that one has to carry out 3D analysis. One could still work with 2D frame systems.

7.5.3 –

The value of damping for buildings may shall be taken as 5 percent of the critical, for the purposes of both seismic coefficient method (as per 7.5.4) and dynamic and static analysis (as per 7.8) for buildings of all materials (of steel, reinforced concrete, and or masonry) buildings.

C7.5.3 –

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.

7.5.3.7.5.4 – Design Seismic Base Shear

The total design lateral force or design seismic base shear (V_B) along any principal direction of a building shall be determined by the following expression:

$$V_B = A_h W$$

where

A_h = Design horizontal acceleration spectrum value as per 6.4.2, using the approximate fundamental natural period T_a as per 7.6 in the considered direction of vibration; and

W = Seismic weight of the building as per 7.4.2.

7.6 – Approximate Fundamental Natural Period**C7.6 – Approximate Fundamental Natural Period**

CODE**COMMENTARY****7.6.1 –**

The approximate fundamental natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick infill panels may be estimated by the empirical expression:

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

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

where

h = Height of building, in m. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.

7.6.2 –

The approximate fundamental natural period of vibration (T_a), in seconds, of all other buildings, including moment-resisting frame buildings with ~~brick infill panels~~ masonry infill panels, may be estimated by the empirical expression:

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

Where

h = Height of building, in m, as defined in 7.6.1; and

d = Base dimension of the building at the plinth level, in m, along the considered direction of the lateral force.

7.6.3 –

For buildings with concrete or masonry shear walls, the approximate fundamental period shall be permitted to be evaluated by the following expression,

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

Where A_w is the total effective area of the walls in the first storey of the building, in m^2 , which may be calculated as:

C7.6.1 –

The two equations for frame buildings were taken from NEHRP's earlier provisions. These equations are based on observed natural period values on real buildings during the 1971 San Fernando earthquake in California (See Figures C 12, C 13 and C 14).

C7.6.2 –

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:

1) Jain, S. K., Saraf V. K., and Malhotra B., "Period of RC Frame Buildings with Brick Infills", *Journal of Structural Engineering*, Madras, Volume 23, No. 4, pp 189-196.

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.

C7.6.3 –

This expression, since it considers the cross sectional area and length of the walls, may give a better estimate of the fundamental natural period of buildings with concrete or masonry shear walls.

L_{wi}/h can become very large for squat type buildings in which length or breadth of building is large compared to its height. An upper limit of 0.9 on L_{wi}/h is specified to prevent larger values of A_w .

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

A_{wi} is the effective cross sectional area of the wall i in the first storey of the building, in m^2 ; L_{wi} is the length of the shear wall i in the first storey in the considered direction of the lateral forces, in m. The value of L_{wi}/h to be used in this equation shall not exceed 0.9.

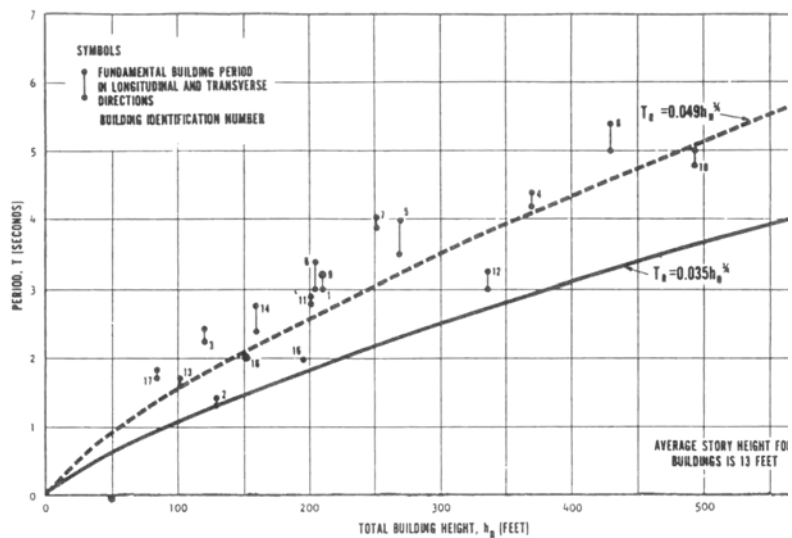


Figure C 12 - Observations on steel frame buildings during San Fernando Earthquake (From FEMA 369, 2001)

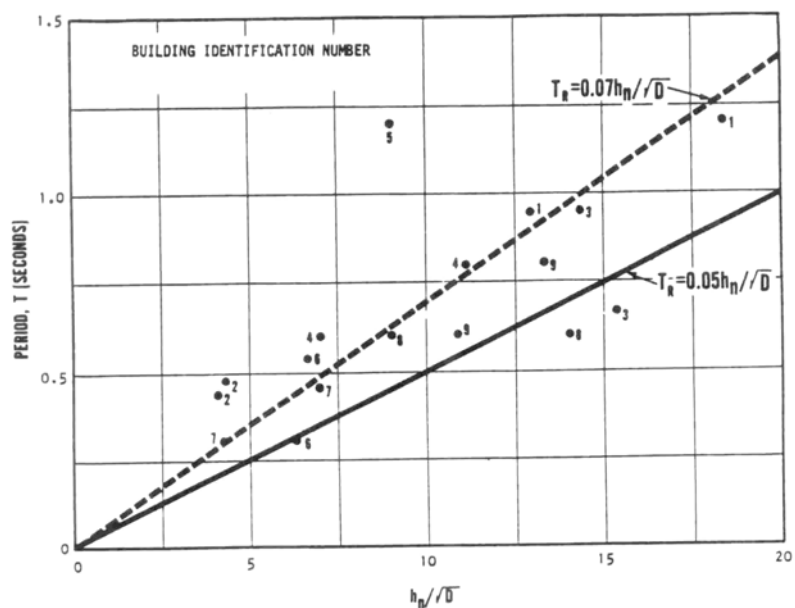


Figure C 13 - Observations on RC frame buildings during

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San Fernando Earthquake (From FEMA 369, 2001)

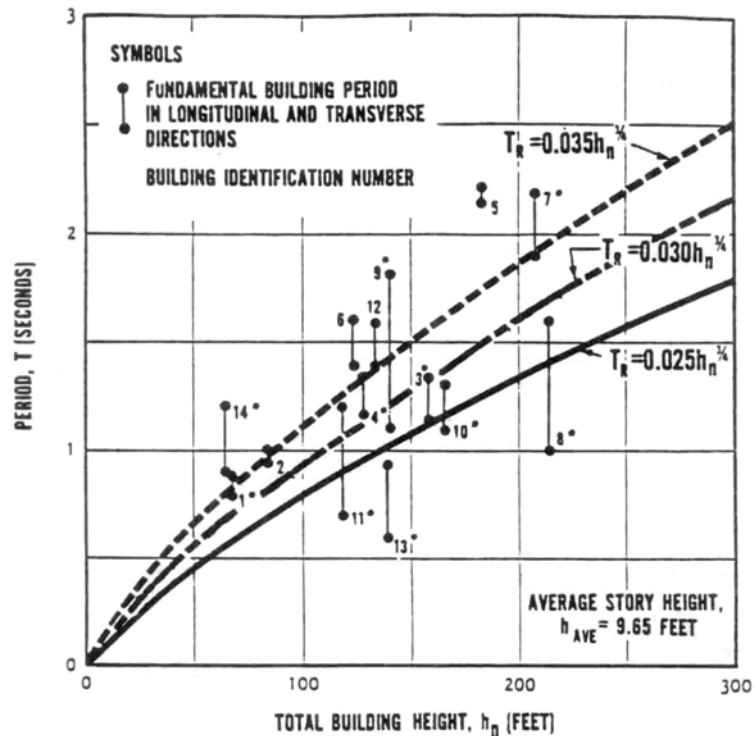


Figure C 14 - Observations on RC shear wall buildings during San Fernando Earthquake (From FEMA 369, 2001)

7.7 – Distribution of Design Lateral Force

7.7.1 – Vertical Distribution of Base Shear to Different Floor Levels

The design base shear (V_B) computed in 7.5.3 shall be distributed along the height of the building as per the following expression:

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

where

Q_i = Design lateral force at floor i ,

W_i = Seismic weight of floor i ,

h_i = Height of floor i measured from

C7.7 – Distribution of Design Lateral Force

C7.7.1 – Vertical Distribution of Base Shear to 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:

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base, and

n = Number of storeys in the building is the number of levels at which the masses are located.

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

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.

7.7.2 – Distribution of Horizontal Design Lateral Force to Different Lateral Force Resisting Elements

7.7.2.1 –

In case of buildings whose floors are capable of providing rigid horizontal diaphragm action, the total shear in any horizontal plane shall be distributed to the various vertical elements of lateral force resisting system, assuming the floors to be infinitely rigid in the horizontal plane.

7.7.2.2 –

In case of building whose floor diaphragms can not be treated as infinitely rigid in their own plane, the lateral shear at each floor shall be distributed to the vertical elements resisting the lateral forces, considering the in-plane flexibility of the diaphragms.

NOTES:

1. A floor diaphragm shall be considered to be flexible, ~~if~~ 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 1.5 times the average displacement of the entire diaphragm.
2. Reinforced concrete monolithic slab-beam floors or those consisting of prefabricated / ~~Precast~~ precast elements with topping reinforced screed can be taken as rigid diaphragms.

C7.7.2– Distribution of Horizontal Design Lateral Force to Different Lateral Force Resisting Elements

Floor diaphragm plays an important role in seismic load distribution in a building. Consider the RC slab. For horizontal loads, it 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 and symmetrical loading, the floorslabs 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 C 15).

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

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floor flexibility. Note 1 of clause 7.7.2.2 gives the criterion when the floor diaphragm is not to be treated as rigid (Figure C 16). Alternatively, one can take the design force as an envelop 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)

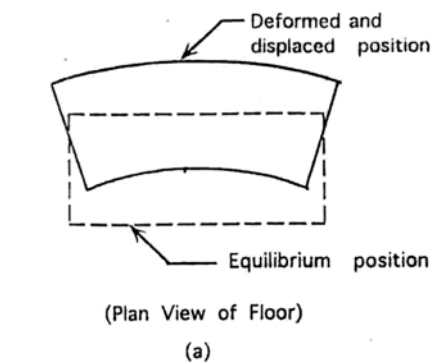
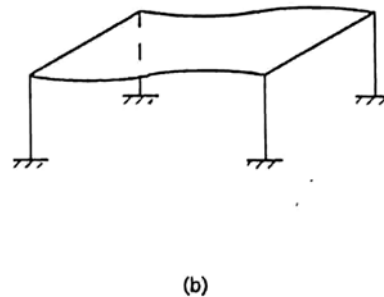
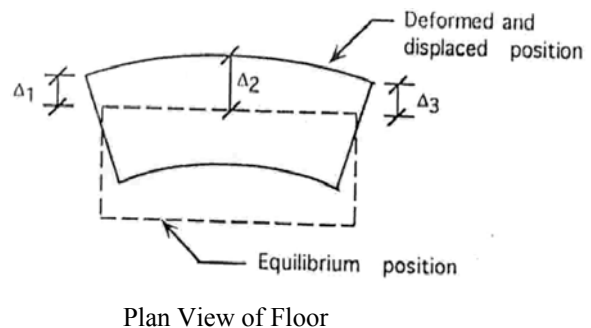


Figure C 15 -- (a) In plane floor deformation, (b) Out-of-plane floor deformation. (From Jain, 1995)



In-plan flexibility of diaphragm to be considered when $\Delta_2 = 1.5 \{0.5(\Delta_1 + \Delta_2)\}$

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Figure C 16 – Definition of Flexible Floor Diaphragm (From Jain, 1995)

7.8 – Dynamic Analysis Method

C7.8 – Dynamic Analysis Method

7.8.1 –

~~Linear Dynamic~~ dynamic analysis shall be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral load resisting elements, for the following buildings:

- a) *Regular buildings* - Those greater than 40 m in height in Zones IV and V, and those greater than 90 m in height in Zones II and III. Modeling as per **7.8.4.5** can be used.
- b) *Irregular buildings with plan irregularities of Type (i)a, (ii), (iii), (iv) or (v) of Table 4 or vertical irregularities of Type (iv) or (v) of Table 5 (as defined in 7.1) - All framed buildings higher than 12 m in Zones IV and V, and those greater than 40 m in height in Zones II and III. It may be noted that vertical irregularity of Type (v) is not permitted in zones IV and V for more than two storey buildings.*

- c) *Irregular buildings with plan irregularity of Type (i)b of Table 4 or vertical irregularities of Type (i), (ii) or (iii) of Table 5 – All buildings higher than 12 m in all zones. It may be noted that buildings with plan irregularity of Type (i)b are not permitted in zones IV and V.*

The analytical model for dynamic analysis of buildings with unusual configuration should be such that it adequately models the types of irregularities present in the building configuration. Buildings with plan irregularities, as defined in Table 4 (as per 7.1), cannot be modeled for dynamic analysis by the method given in **7.8.4.5**.

C7.8.1 –

Expressions for design load calculation and load distribution with height given in 7.5 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 judgement on the part of 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 prohibited in zones of high seismicity (zones IV and V) (see note at the end of Table 4).

Dynamic analysis requires considerable skills. 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

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NOTE - ~~For irregular buildings, lesser than 40 m in height in Zones II and III. Dynamic analysis, even though not mandatory, is recommended. Dynamic analysis is recommended for irregular buildings of lower height even though it may not be mandatory for lower heights as per clause 7.8.1.~~

7.8.2 –

Dynamic analysis may be performed either by the Time History Method or by the Response Spectrum Method. However, in either method, the design base shear (V_B) shall be compared with a base shear ($\bar{V}_B$) calculated using a fundamental period T_a , where T_a is as per 7.6. Where V_B is less than $\bar{V}_B$, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $\bar{V}_B/V_B$.

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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:

1. Chopra, A.K., *Dynamics of Structures : A Primer*, Earthquake Engineering Research Institute, Oakland, California, USA, 1980.
2. Chopra, A.K., *Dynamics of Structures : Theory and Applications to Earthquake Engineering*, Pearson Education, New Delhi, 2001.
3. Paz, M., *Structural Dynamics : Theory and Computations*, 3rd Edition, Van Nostrand Reinhold, 1991.
4. Clough, R.W., and Penzien, J., *Dynamics of Structures*, 2nd Edition, McGraw Hill, 1973.
5. Wilson, E.L., *Three Dimensional Static and Dynamic Analysis of Structures – A physical approach with emphasis on earthquake engineering*, Computer and Structures Inc., Berkeley, CA, USA, 2000.

C7.8.2 –

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.

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

CODE**COMMENTARY**

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.

7.8.2.1–

~~The value of damping for buildings may be taken as 2 and 5 percent of the critical, for the purposes of dynamic analysis of steel and reinforced concrete buildings, respectively.~~

7.8.3– Time History Method

Time history method of analysis, when used, shall be based on an appropriate ground motion and shall be performed using accepted principles of dynamics.

7.8.4 – Response Spectrum Method

Response spectrum method of analysis shall be performed using the design acceleration spectrum specified in 6.4.2, or by a site-specific design acceleration spectrum mentioned in 6.4.7.

C7.8.3 – Time 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.

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7.8.4.1 – Free Vibration Analysis

Undamped free vibration analysis of the entire building shall be performed as per established methods of mechanics using the appropriate masses and elastic stiffness of the structural system, to obtain natural periods (T) and mode shapes $\{\phi\}$ of those of its modes of vibration that need to be considered as per 7.8.4.2.

7.8.4.2 – Modes to be considered

The number of modes to be used in the analysis for a considered direction of earthquake shaking should be such that the sum total of modal masses of all modes considered is at least 90 percent of the total seismic mass ~~and missing mass correction beyond 33 percent~~. If modes with natural ~~frequency-frequencies~~ beyond 33 Hz are to be considered, the modal combination shall be carried out only for modes up to 33 Hz ~~and -t~~. The effect of higher-modes with natural frequencyies beyond 33 Hz shall be included by considering-the missing mass correction procedure following well established ~~proceduresprinciples~~.

7.8.4.3 – Analysis of Buildings subjected to Design Forces

The building may be analyzed by accepted principles of mechanics for the design forces considered as static forces.

7.8.4.4 – Modal Combination

C7.8.4.2 – 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 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 mass of the structure. The difference between the total 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 3 of this standard is 1.0. Thus, the Design Horizontal Seismic Coefficient A_h corresponding to the missing mass becomes $Z/2R$.

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 7.8.4.4.

C7.8.4.4 – Modal Combination

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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.

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

where

r = Number of modes being considered,

ρ_{ij} = Cross-modal coefficient,

λ_i = Response quantity in mode i (including sign),

λ_j = Response quantity in mode j (including sign),

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

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

ζ = Modal damping ratio (in fraction) as specified in **7.8.2.1 7.5.3**,

β = Frequency ratio = ω_j / ω_i ,

ω_i = Circular frequency in i th mode, and

ω_j = Circular frequency in j th mode.

~~Alternatively~~ **Alternately**, the peak response quantities may be combined as follows:

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

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

where

λ_k = Absolute value of quantity in mode k , and

r = Number of modes being

COMMENTARY

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:

Chopra, A.K., *Dynamics of Structures : Theory and Applications to Earthquake Engineering*, Pearson Education, New Delhi, 2001.

CODE

considered.

- b) If the building has a few closely-spaced modes (see 3.2), then the peak response quantity (λ^*) due to these modes shall be obtained as

$$\lambda^* = \sum \lambda_c$$

where the summation is for the closely-spaced modes only. This peak response quantity (λ^*) due to the closely spaced modes (λ^*) is then combined with those of the remaining well-separated modes by the method described in 7.8.4.4 (a).

7.8.4.5 –

Buildings with regular, or nominally irregular plan configurations may be modelled as a system of masses lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration. In such a case, the following expressions shall hold in the computation of the various quantities:

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

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

where

g = Acceleration due to gravity,

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

W_i = Seismic weight of floor i .

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

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

- c) *Design Lateral Force at Each Floor in Each Mode* - The peak lateral force (Q_{ik})

COMMENTARY**C7.8.4.5 –**

The analysis procedure is valid when a building can be modeled as a lumped mass model with one degree of freedom per floor (Figure C 17).



Figure C 17 – Lumped mass model

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.

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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 vibration (T_k) of mode k .

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

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

- e) *Storey Shear Force due to All Modes Considered* - The peak storey shear force (V_i) in storey i due to all modes considered is obtained by combining those due to each mode in accordance with **7.8.4.4**.

- f) *Lateral Forces at Each Storey Due to All Modes Considered* - The design lateral forces, F_{roof} and F_i , At roof and at floor i :

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

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

7.9 – Torsion**7.9.1 –**

Provision shall be made in all buildings for increase in shear forces on the lateral force resisting elements resulting from the horizontal Torsional moment arising due to eccentricity between the centre of mass and centre of rigidity. The design forces calculated as in **7.8.4.5** are to be applied at the centre of mass appropriately displaced so as to cause design eccentricity (**7.9.2**) between the displaced centre of mass and centre of rigidity. ~~However, negative Torsional shear shall be neglected.~~

7.9.2 – Design Eccentricity

The design eccentricity, e_{di} to be used at floor i shall be taken as:

$$e_{di} = 1.5 e_{si} + 0.05 b_i$$

C7.9.2 – 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

$$\text{Or } e_{si} - 0.05 b_i$$

whichever of these gives the more severe effect in the shear of any frame where

e_{si} = Static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity, and

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

~~NOTE—The factor 1.5 represents dynamic amplification factor, while the factor 0.05 0.10 represents the extent of accidental eccentricity.~~

NOTES –

1. The factor 1.5 represents dynamic amplification, while the factor 0.5 represents accidental eccentricity.
2. In case 3D dynamic analysis is carried out, the dynamic amplification factor of 1.5 be replaced with 1.0.

COMMENTARY

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.

The factor 1.5 is intended for use with equivalent static analyses only. However, when 3D dynamic analysis is conducted, the dynamic amplification is inherent in the analysis. Thus, Note 2 seeks to eliminate the factor 1.5.

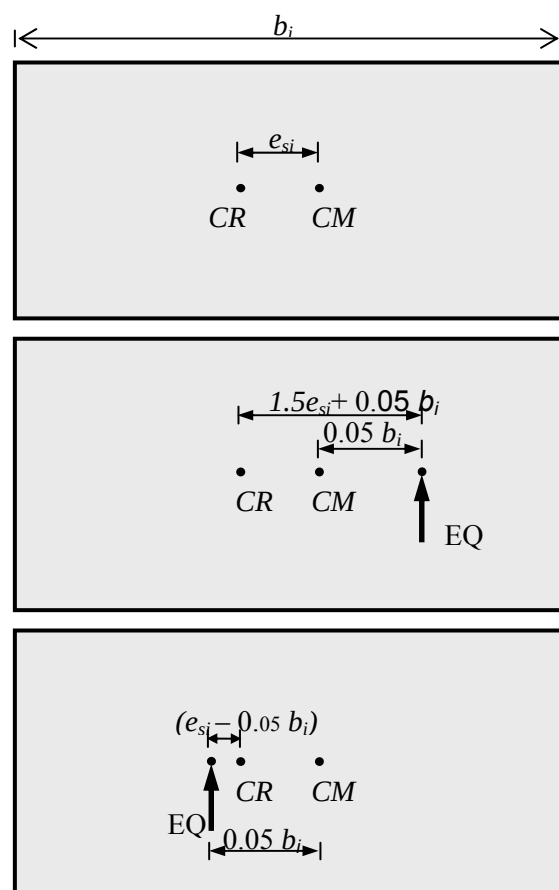


Figure C 18 – Two possible cases of maximum eccentricity

7.9.3–

~~In case of highly irregular buildings analyzed~~

CODE**COMMENTARY**

~~according to 7.8.4.5, additive shears will be superimposed for a statically applied eccentricity of $\pm 0.05b$, with respect to the centre of rigidity.~~

7.10 – RC Frame Buildings with Masonry Infills

Provisions in 7.10 intend to incorporate the stiffness and strength due to in-plane behaviour of infills in the design of buildings.

The advantages of strength contributed by the infill shall not to be considered when the height of the building is more than 12m.

7.10.1 –

The modulus of elasticity (in MPa) of masonry, E_m , may be taken as:

$$E_m = 550f_m$$

Where f_m is the compressive strength of masonry prism in MPa.

7.10.2 –

Infill wall may be modeled by using an equivalent diagonal strut as follows per 7.10.2.1, 7.10.2.2 and 7.10.2.3.

7.10.2.1 –

The ends of diagonal struts shall be pin-jointed to the RC frame such that moment transfer does not take place from RC frame to struts.

C7.10 – 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. 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. Also, infills can cause irregularities in the building, e.g., short column effect. This should be recognized at the design stage itself.

C7.10.1 –

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. A limited number of tests conducted recently at IIT Kanpur showed that this value agrees with experimental data reasonably well.

C7.10.2 –

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 19.

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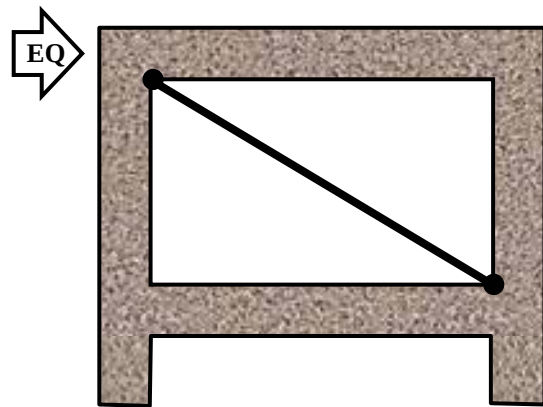


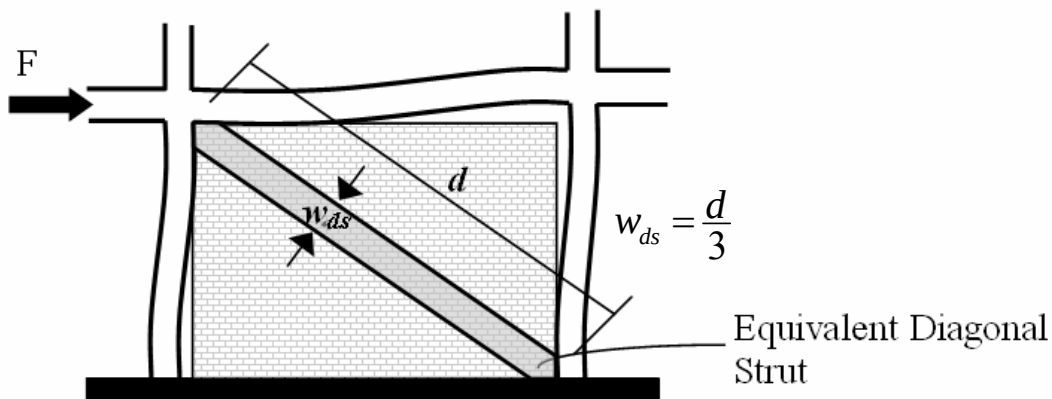
Figure C 19 – Equivalent diagonal strut model

7.10.2.2 –

For the solid walls (without any openings), width of equivalent diagonal strut (w_{ds}) shall be taken as one third of the diagonal length of the infill wall (d) as shown in Figure 6.

C7.10.2.2 –

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 value of effective width adopted in this code is as per the following: Holmes, M., 1961, "Steel Frames with Brickwork and Concrete Infilling," *Proceedings of the Institution of Civil Engineers*, Vol. 19, August, pp. 473-478.

**Figure 6 – Details of equivalent strut****7.10.2.3 –**

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

C7.10.2.3 –

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 ρ_w is defined as

CODE**COMMENTARY**

$$w_{do} = \rho_w w_{ds}$$

where w_{ds} is the width of diagonal strut for infill walls ~~with~~ **without** openings and ρ_w is a reduction factor, which accounts for openings in infill, which is given by

$$\rho_w = 1 - 2.5A_r$$

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.

7.10.2.4 –

Thickness of the strut shall be taken as the actual thickness of the wall.

7.10.3 –

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.

7.107.11 – Special Provisions for Irregular Buildings**7.10.1 7.11.1 – Buildings with Soft or Weak Storey**

In ~~case of~~ buildings with a ~~flexible storey~~ **vertical irregularity of Type (i) or Type (v) in Table 5**, such as the ground storey consisting of open spaces for parking that is ~~sit~~ **on stilts**, special arrangement

ratio of reduced strut width to strut-width corresponding to fully infilled frame. The equation for ρ_w is based on the following:

Mondal, G., 2003, *Lateral Stiffness of Unreinforced Brick Infilled RC Frame with Central Opening*, Master of Technology Thesis, Department of Civil Engineering, Indian Institute of Technology Kanpur, India, July.

C7.10.3 –

Other than self weight, masonry infill is not expected to carry any gravity loads.

The contribution of the infill in resisting the lateral loads can be substantial. However, to safeguard against RC frame being designed for a very low seismic force, this clause requires that the frame alone (without infill walls) should be designed to resist at least 50% of the total seismic force.

C7.11 – Special Provisions for Irregular Buildings**C7.11.1 – Buildings with Soft or Weak Storeys**

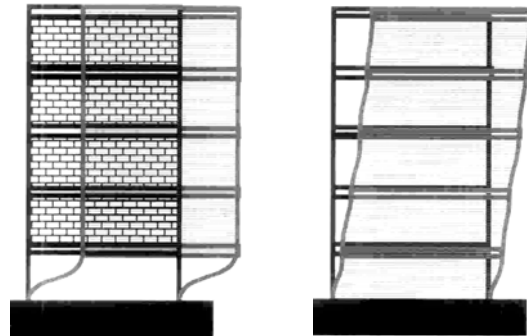
Generally, soft storey building is also a weak storey building. Soft/weak storey buildings are well known for their poor performance during earthquakes. During the Bhuj earthquake of 2001, most of the multi-storey buildings that collapsed

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needs to be made to increase the lateral strength and stiffness of the soft/~~open~~ weak storey.

COMMENTARY

had soft ground storey. Figure C 20 indicates the severe deformation demands in case of a building with a soft storey.



(a) Open ground storey

Figure C 20 - Soft-storey is subject to severe deformation demands during seismic shaking (From Murty et al, 2002)

7.10.27.11.1.1 -

~~Dynamic Non-linear push-over~~ analysis of building ~~is~~ should ~~shall~~ be carried out including the strength and stiffness effects of infills, and ~~the~~ inelastic deformations in the members, particularly, those in the soft /weak storey, ~~and~~ The members ~~should~~ shall be designed accordingly considering these deformation and ductility demands. Specialist literature may be referred to for this purpose.

C7.11.1.1 -

Pushover analysis is a static, nonlinear procedure in which the magnitude of the structural loading is increased incrementally in accordance with a certain predefined pattern. By increasing the magnitude of loading, weak links and failure modes of the structure are found.

Non-linear pushover analysis can be used to estimate the ultimate lateral load carrying capacity of the structure and the ultimate displacement up to which the structure can be displaced laterally without collapse. Ductility and overstrength of the structure can be found out from the pushover curve, (i.e., the plot of base shear versus roof displacement).

While performing pushover analysis, inelastic properties of all the elements in the buildings (including infill walls) are to be modeled carefully. Also, the mass, stiffness and strength of all the elements in the building should be modeled properly.

The elements should be designed for the seismic demands given by pushover analysis for a given level of ductility.

While performing the non-linear pushover analysis, the following publication may be referred to:

ATC 40, *Seismic Evaluation and Retrofit of Concrete Buildings*, Applied Technology Council, Redwood City, CA, USA, 1996.

CODE**COMMENTARY****7.10.37.11.1.2 –**

Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis; neglecting the effect of infill walls in other storeys:

the columns and beams of the soft/weak storey (excluding the beams between the stilt storey and the infilled storey) are to be designed for 2.5 times the storey shears and moments calculated under seismic loads specified in the other relevant clauses; or,

besides the columns designed and detailed for the calculated storey shears and moments, shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible; to be designed exclusively for 1.5 times the lateral storey shear force calculated as before.

7.11.2 –

In case of plan irregularity of Type (iv) in Table 4 or vertical irregularity Type (iv) in Table 5, columns, beams or trusses supporting discontinuous walls or frames shall be designed for 2.5 times the forces obtained under seismic loads specified in other relevant clauses for all Zones. All the other members of buildings in Zones IV and V shall be designed for the seismic forces, calculated as per relevant clauses, increased by 20%; this increase is not required for buildings in Zones II and III.

7.11.3 –

In case of plan irregularity of Type (ii) and

C7.11.1.2 –

Pushover analysis is fairly sophisticated and requires considerable expertise. It is therefore not always feasible to perform a non-linear pushover analysis. Hence, an alternative design procedure is given in the code.

All the columns of the soft/weak storey should be designed for 2.5 times the seismic demand. Beams between the stilt storey and the infilled storey are not to be designed for the increased demands because stronger beams would further increase the seismic demands on the columns. Other elements in the building on the other stories are to be designed for the respective seismic force resultants given by the static analysis.

If it is not feasible to increase the capacity of the columns in soft/weak storey, shear walls should be provided, preferably on the periphery of the building. Care should be taken to ensure symmetric arrangement of the shear walls to avoid the torsional effects. The shear walls should be designed for 1.5 times the seismic demand for the storey as per calculations while the columns are designed for 100% of seismic demand.

C7.11.2 –

An out-of-plane offset of the lateral load carrying vertical element imposes excessive demands on vertical elements. Similarly, an in-plane offset of the lateral force resisting element, greater than the length of those elements, impose vertical and lateral load demands on the supporting 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. Hence, the supporting elements are required to be designed for 2.5 times the force resultants obtained by the static analysis as specified in other relevant clauses of the code.

In the zones of high seismicity (Zone IV and V), irregular buildings are prone to severe damage when subjected to seismic forces. It is, therefore, recommended to design all the other elements of such buildings for 1.2 times the force resultants obtained by the static analysis as specified in other relevant clauses of this code.

C7.11.3 –

The plan irregularities such as, re-entrant corners

CODE**COMMENTARY**

Type (iii) in Table 4, buildings in Zones IV and V shall be designed for the seismic forces, calculated as per relevant clauses, increased by 20%. Such increase is not required for buildings in Zones II and III.

and diaphragms discontinuity, change lateral load distribution to different vertical elements. To take care of increase in the seismic demands on the structure because of such plan irregularities, it is recommended to design all the elements of such buildings for 1.2 times the force resultants obtained by the static analysis as specified in other relevant clauses of this code in zones IV and V.

7.11.4 –

For buildings that have plan irregularity of Type (v) in Table 4, ground motion in two horizontal directions will be considered as per 6.3.2.2 or 6.3.4.2.

C7.11.4 –

See commentary of clause 6.3.2.

7.11.5 – Buildings with Torsional Irregularity**C7.11.5 - Buildings with Torsional Irregularity**

Torsional irregularities arise due to non-uniform distribution of mass and stiffness. Because of torsion, the seismic force resultants in some elements of the building are increased.

7.11.5.1–

In ~~case of~~ buildings located in Zones IV and V with torsional irregularity Type (i)a in Table 1, the design seismic forces, calculated as per relevant clauses, shall be increased by 20%.

C7.11.5.1 –

In zones of high seismicity, the torsionally irregular buildings are prone to very severe damage. Hence, buildings with extreme torsional irregularities (Type (i)b in Table 4) are not permitted in zones IV and V. However, for building in zones IV and V with torsional irregularity of Type (i)a in Table 4, it is recommended to design all the elements for 1.2 times the force resultants obtained by the seismic analysis.

7.11.5.2–

In ~~case of~~ buildings located in Zones II and III with extreme torsional irregularity (Type (i)b in Table 1), the design seismic forces, calculated as per relevant clauses, shall be increased by 20%.

7.11.5.3–

If torsional irregularity of Type (i)a or Type (i)b in Table 1 is about both the orthogonal axes, the building shall be designed for ground motion in two horizontal directions as per 6.3.2.2 or 6.3.4.2.

7.11.5.3 –

See commentary of clause 6.3.2

7.11.7.12– Deformation**C7.12 – Deformation**

CODE**COMMENTARY**

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 force is a reduced force. 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 shear walls, etc.

7.11.17.12.1– Storey Drift Limitation

The storey drift in any storey due to the minimum specified design lateral force, with partial load factor of 1.0, shall not exceed 0.004 times the storey height.

C7.12.1 – 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

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For the purposes of displacement requirements only (see ~~7.11.1~~ 7.12.1, ~~7.11.2~~ 7.12.2 and ~~7.11.3~~ 7.12.3 only), it is permissible to use seismic force obtained from the computed fundamental period (T) of the building without the lower bound limit on design seismic force specified in **7.8.2**.

There shall be no drift limit for single storey building which has been designed to accommodate storey drift.

~~7.11.2~~ 7.12.2 – Deformation Capability of Non-Seismic Members

For building located in seismic Zones IV and V, it shall be ensured that the structural components, that are not a part of the seismic force resisting system in the direction under consideration, do not lose their vertical load-carrying capacity under the induced moments resulting from storey deformations equal to R times the storey displacements calculated as per ~~7.11.1~~ 7.12.1, where R is specified in Table 7.

NOTE- For instance, consider a flat-slab building in which lateral load resistance is provided by shear walls. Since the lateral load resistance of the slab-column system is small, these are often designed only for the gravity loads, while all the seismic force is resisted by the shear walls. Even though the slabs and columns are not required to share the lateral forces, these deform with rest of the structure under seismic force. The concern is that under such deformations, the slab-column system should not lose its vertical load capacity.

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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 third paragraph allows larger than the specified drift for single-storey building provided it is duly accounted for in the analysis and design.

C7.12.2 – 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 shear 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 shear walls due to compatibility imposed by floor diaphragm (Figure C 21). Moments and shears induced in gravity columns due to the lateral deformations may cause collapse if adequate provisions are not made. ACI 318 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.

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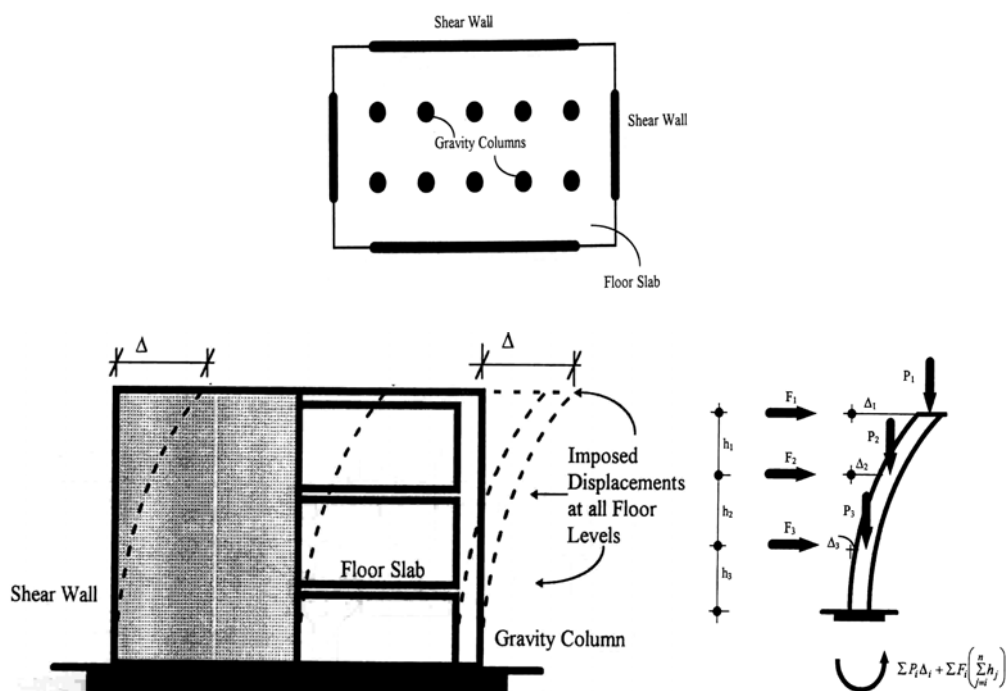


Figure C 21 - Lateral deformation of gravity columns (From Agarwal, 1996)

7.11.37.12.3– Separation between Adjacent Units

Two adjacent buildings, or two adjacent units of the same building with separation joint in between shall be separated by a distance equal to the amount R times the sum of the calculated storey displacements as per 7.11.1 of each of them, to avoid damaging contact when the two units deflect towards each other. When floor levels of two similar adjacent units or buildings are at the same elevation / levels, factor R in this requirement may be replaced by $R/2$.

C7.12.3 – 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*. 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. Pounding effect may be much more serious if floors of one building hit at the mid-height of columns in the other building (Figure C 22 b). Hence, when two units have same floor elevations, the multiplier is reduced from R to $R/2$.

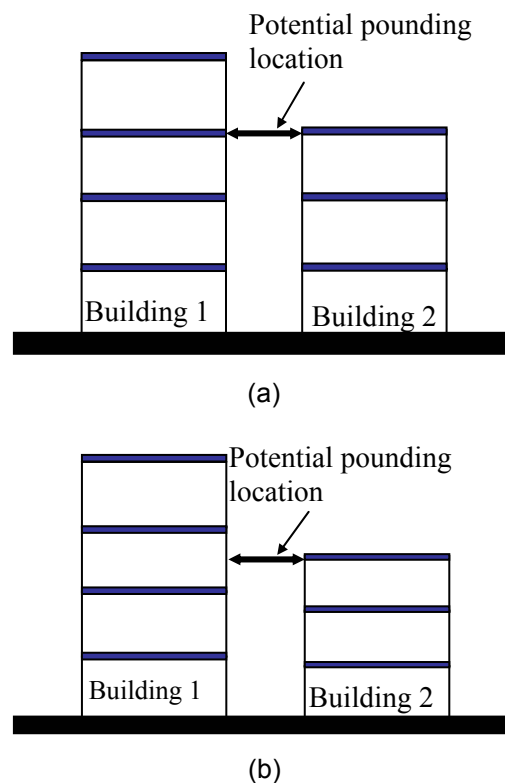
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Figure C 22 - Pounding in situation (b) is far more damaging.

7.13– Nonstructural Elements

7.13.1–General

7.13.1.1-

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

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, of fire and police stations, power stations, communication facilities and water supply. 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:

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1. Gillengerten, J.D., *Design of Nonstructural Systems and Components*, The Seismic Design Handbook (Naeim, F., editor), Kluwer Academic Publishers, Second Edition, 682-721, 2003
2. Villaverde, R., *Seismic Analysis and Design of Nonstructural Elements*, Earth Engineering: from Engineering Seismology to Performance-Based Engineering (Bozorgnia, Y., and Bereto, V.V., editor), CRS Press, 2004.
3. Stratta, J.L., *Manual of Seismic Design*, Pearson Education, First Indian Reprint, 184-216, 2003.
4. FEMA 368, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: Part 1-Provisions*, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C., March 2001.
5. FEMA 369, *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures: Part 2-Commentary*, Building Seismic Safety Council, National Institute of Building Sciences, Washington, D.C., March 2001.
6. IBC 2003, *International Building Code*, International Code Council, USA.
7. Eurocode 8, *Design Provisions for Earthquake Resistance of Structures, Part 1-General Rules, Seismic Action and Rules for Buildings*, prEN 1998-1, European Committee for Standardization, Brussels, 2003.

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 7.10.

7.13.1.3-

For nonstructural elements of great importance or of a particular dangerous nature, the seismic analysis should be based

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on the ~~use of~~ floor response spectra derived from the response of the main structural system. Specialist literature may be referred to for the methods of determining floor response spectrum for various floors/elevations.

7.13.1.4–

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

C 7.13.1.4–

Partial infill of masonry walls between columns may create short-column effect, i.e, reduce the 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 9 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-

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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 9). They must be analyzed for both forms of response, that is, as per provisions 1.3 and 1.4.

7.13.3– Design Seismic Force**7.13.3.1–**

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

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

$$\geq 0.10 W_p$$

Where

Z = Zone factor given in Table 2,

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 10,

R_p = Component response modification factor given in Table 11,

I_p = Importance factor of the nonstructural element given in Table 12, 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 9, 10, 11) 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).

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

Table 9: Response Sensitivity of Nonstructural Components (clause 7.13.2)

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

Table 10: Coefficients for Architectural Components (clause 7.13.3)

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

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

Table 11: Coefficients for Mechanical and Electrical Components (clause 7.13.3)

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

^aA 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 12: Importance Factor (I_p) of Nonstructural Elements (Clause 7.13.3)

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

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7.13.3.2–

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

7.13.3.3–

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

7.13.3.4– Connections

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

7.13.4– Seismic Relative Displacement

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

7.13.4.1–

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

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C7.13.3.2–

No distinction is being made between the horizontal and the vertical vibrations of the ground and of the structure, considering many other approximations involved.

C7.13.3.3–

A vibration isolated component can experience higher seismic accelerations than in the case the same component is rigidly mounted. This is due to the amplification effects of the vibration mounts. The fundamental period of the isolated components can be such that resonance condition with one or more modes of the primary structure is possible. This can result in amplification in lateral force.

C7.13.3.4–

Friction forces induced by gravity should be ignored, because vertical ground motions may reduce the effect of gravity.

C7.13.4–

Seismic relative displacement equations are provided to support the selection and design of cladding, stairwells, piping systems, sprinkler systems, and other components that are connected to the building at multiple levels (clause 7.13.4.1) or to adjacent buildings (clause 7.13.4.2). These equations provide the architect a rational basis for assessing the flexibility or clearances required by components and claddings and their connections to accommodate the expected building movements during earthquake.

C7.13.4.1–

The first equation yields an estimate of actual structural displacements, as determined by elastic analysis, with no structural-response modification factor (R). Second equation is provided in recognition that elastic displacements are not

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

D_p is not required to be taken as greater than

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

where,

δ_{xA} = Deflection at building level x of structure A due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as per Table 7,

δ_{yA} = Deflection at building level y of structure A due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as per Table 7,

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

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

Δ_{aA} = Allowable storey drift for structure A calculated as per 7.12.1, and

h_{sx} = Storey height below level x.

always defined or available at the time the component is designed or procured. This equation allows the use of storey drift limitations.

7.13.4.2-

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

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

D_p is not required to be taken as greater than

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

where,

δ_{yB} = Deflection at building level y of structure B due to design seismic load determined by elastic analysis, and multiplied by response reduction factor (R) of the building as per Table 7,

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Δ_{aB} = Allowable storey drift for structure B
calculated as per 7.12.1.

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.

7.12.7.14 – Miscellaneous**7.12.17.14.1 – Foundations**

Isolated R.C.C. footings without tie beams, or unreinforced strip foundation shall not be permitted in soft soils with $N < 10$ for any seismic zone. The use of foundations vulnerable to significant differential settlement due to ground shaking shall be avoided for structures in seismic Zones III, IV and V. 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 5% of the larger of the column or pile cap load, in addition to the otherwise computed forces. Here, A_h is as per 6.4.2.

C7.14 -- Miscellaneous**C7.14.1 – 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.

7.12.27.14.2 – Cantilever Projections**7.12.2.17.14.2.1 – Vertical Projections**

Tower, tanks, parapets, smoke stacks (chimneys) and other vertical cantilever projections attached to buildings and projecting above the roof, shall be designed and checked for stability for five times the design horizontal seismic coefficient A_h specified in 6.4.2. In the analysis of the building, the weight of these projecting elements will be lumped with the roof weight

C7.14.2 – Cantilever Projections

All projections (vertical and horizontal) are highly vulnerable to damage during earthquakes. Being cantilevers, there is no redundancy and hardly any ductility. Hence, the projections are designed for five times the seismic coefficient.

7.12.2.27.14.2.2 – Horizontal Projections

All horizontal projections like brackets, cornices and balconies shall be designed

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and checked for stability for five times the design vertical coefficient specified in **6.4.5** (that is for $10/3 A_n$).

~~7.12.2.37.14.2.3 –~~

The increased design forces specified in ~~7.12.2.1 7.14.2.1~~ and ~~7.12.2.2 7.14.2.2~~ are only for designing the projecting parts and their connections with the main structures. For the design of the main structure, such increase need not be considered.

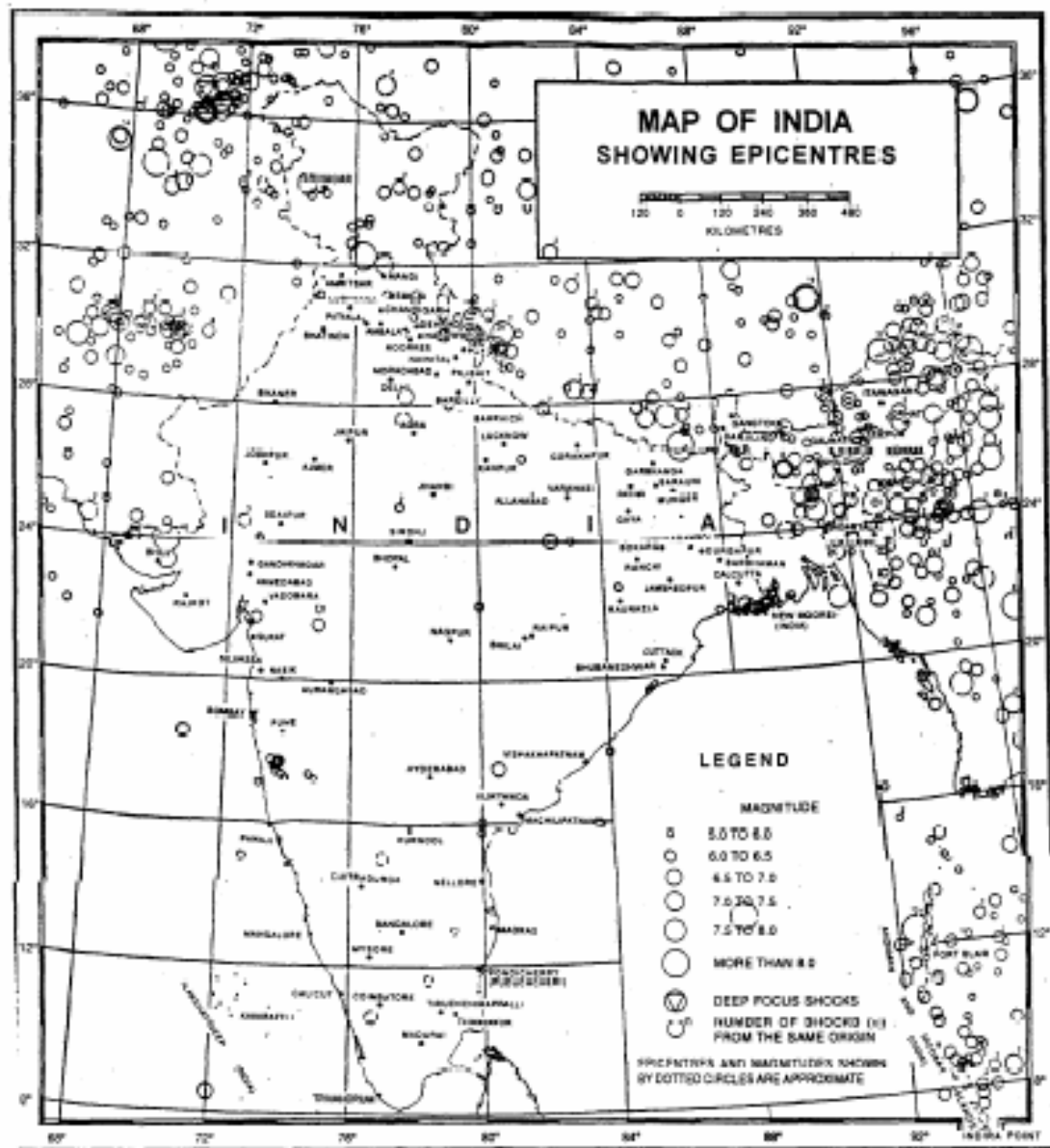
~~7.12.37.14.3– Compound Walls~~

Compound walls shall be designed for the design horizontal coefficient of $Z/2$. ~~A_n with importance factor $I = 1.0$ specified in 6.4.2.~~

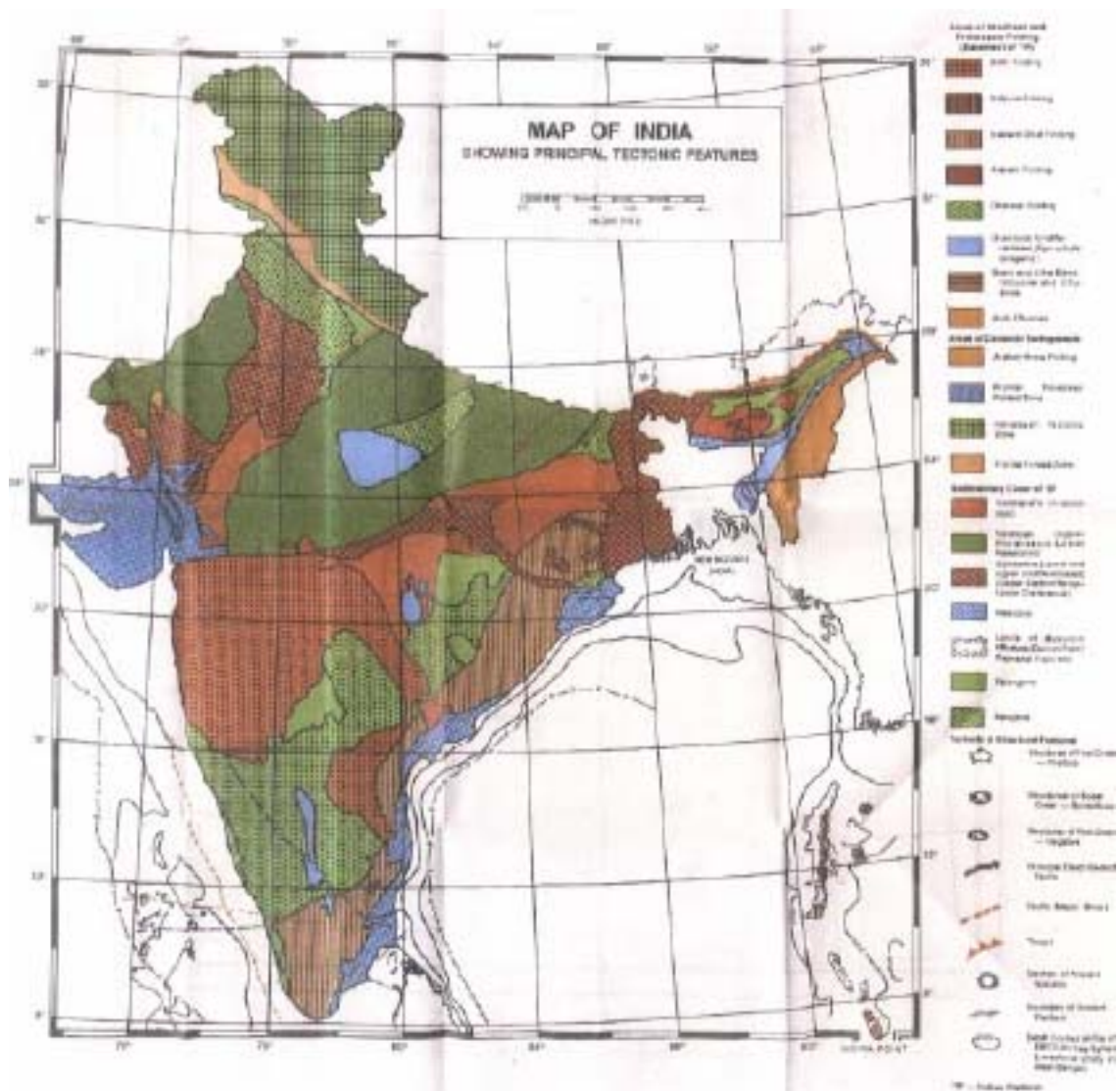
~~7.12.4 7.14.4– Connections between Parts~~

All parts of the building, except between the separation sections, shall be tied together to act as integrated single unit. All connections between different parts, such as beams to columns and columns to their footings, should be made capable of transmitting a force, in all possible directions, of magnitude (Q_i/W_i) times but not less than 0.05 times the weight of the smaller part of the total of dead and imposed load reaction. Frictional resistance shall not be relied upon for fulfilling these requirements.

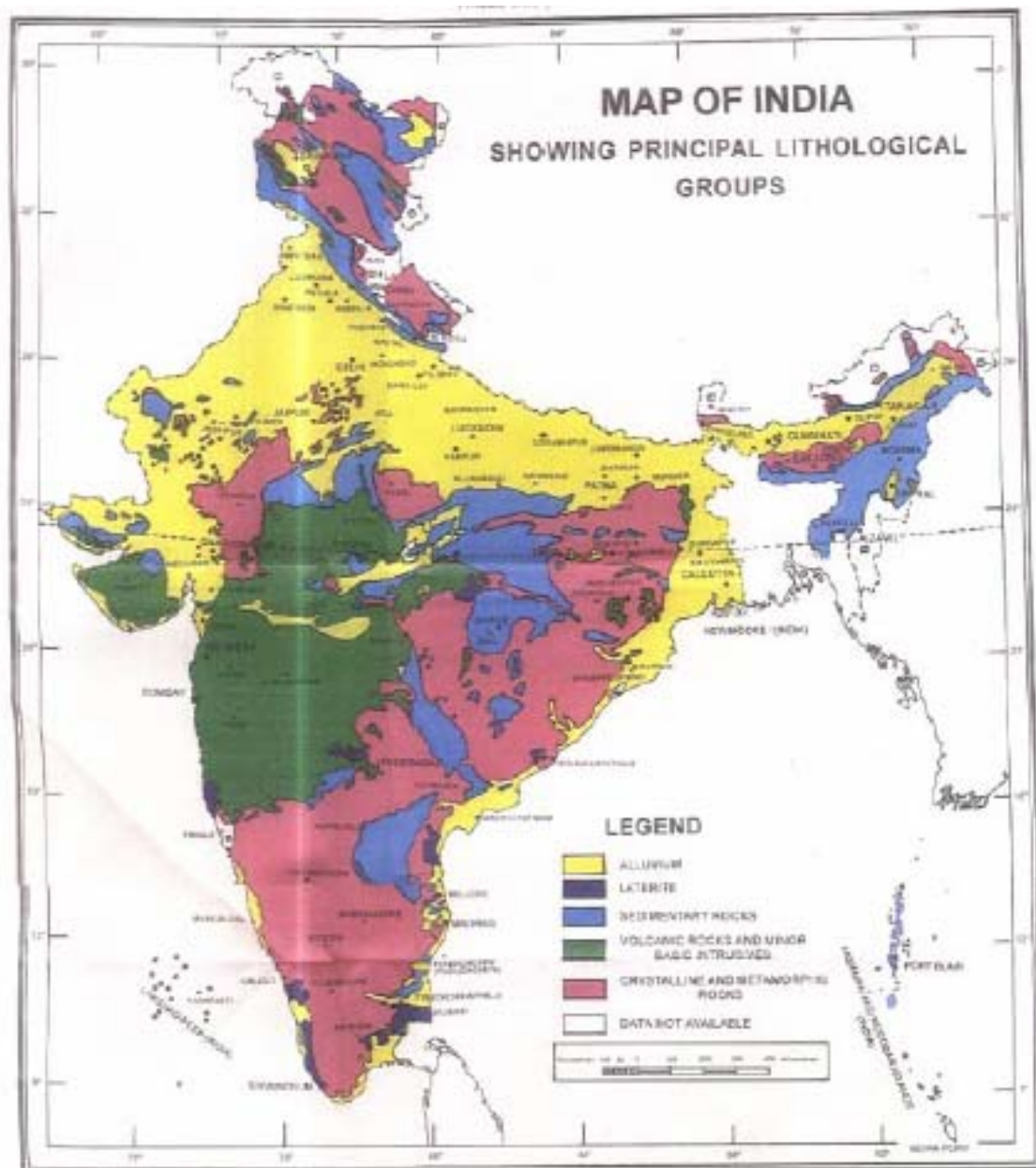
Annex A



Annex B



Annex C



Annex D

(Foreword and Clause 3.15)

~~Comprehensive Intensity Scale (MSK 64 Intensity Scale)~~

~~The scale was discussed generally at the inter-governmental meeting convened by UNESCO in April 1964. Though not finally approved the scale is more comprehensive and describes the intensity of earthquake more precisely. The main definitions used are followings;~~

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.

b) Definition of Quantity

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

c) Classification of Damage to buildings

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

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.

~~de~~) Intensity Scale

I. ~~4~~ Not Noticeable – The intensity of the vibration is below the limits of sensibility; the tremor is detected and recorded by seismograph only.

II. ~~2~~ Scarcely noticeable (very slight) – Vibration is felt only by individual people at rest in houses, especially on upper floors of buildings.

III. ~~3~~ Weak, partially observed only – The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects.

IV. ~~4~~ 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. ~~5~~-Awakening

- i) The earthquake is felt indoors by all, outdoors by many. Many people awake. A few run outdoors. Animals become uneasy. Buildings tremble throughout. Hanging objects swing considerably. Pictures knock against walls or swing out of place. Occasionally pendulum clocks stop. Unstable objects overturn or shift. ~~Open doors~~ 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. ~~6~~-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 ~~few 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 ~~few some~~ buildings of Type A is of Grade 2.
- iii) ~~In few cases, cracks~~ 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. ~~7~~-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. ~~8~~-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. ~~9~~-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. ~~10.~~ 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. ~~11.~~ 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. ~~12.~~ 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 deflected. The intensity of the earthquake requires to be investigated specially.

Annex E

Town	Zone	Zone Factor, Z	Town	Zone	Zone Factor, Z
Agra	III	0.16	Kanchipuram	III	0.16
Ahmedabad	III	0.16	Kanpur	III	0.16
Ajmer	II	0.10	Karwar	III	0.16
Allahabad	II	0.10	Kochi	III	0.16
Almora	IV	0.24	Kohima	V	0.36
Ambala	IV	0.24	Kolkata	III	0.16
Amritsar	IV	0.24	Kota	II	0.10
Asansol	III	0.24	Kurnool	II	0.10
Aurangabad	II	0.10	Lucknow	III	0.16
Bahraich	IV	0.24	Ludhiyana	IV	0.24
Bangalore	II	0.10	Madurai	II	0.10
Barauni	IV	0.24	Mandi	V	0.36
Bareilly	III	0.16	Mangalore	III	0.16
Belgaum	III	0.16	Monghyr	IV	0.24
Bhatinda	III	0.16	Moradabad	IV	0.24
Bhilai	II	0.10	Mumbai	III	0.16
Bhopal	II	0.10	Mysore	II	0.10
Bhubaneswar	III	0.16	Nagpur	II	0.10
Bhuj	V	0.36	Nagarjunasagar	II	0.10
Bijapur	III	0.16	Nainital	IV	0.24
Bikaner	III	0.16	Nasik	III	0.16
Bokaro	III	0.16	Nellore	III	0.16
Bulandshahr	IV	0.24	Osmanabad	III	0.16
Burdwan	III	0.16	Panjim	III	0.16
Calicut	III	0.16	Patiala	III	0.16
Chandigarh	IV	0.24	Patna	IV	0.24
Chennai	III	0.16	Pilibhit	IV	0.24
Chitradurga	II	0.10	Pondicherry	II	0.10
Coimatore	III	0.16	Pune	III	0.16
Cuddalore	II	0.10	Raipur	II	0.10
Cuttack	III	0.16	Rajkot	III	0.16
Darbhanga	V	0.36	Ranchi	II	0.10
Darjeeling	IV	0.24	Roorkee	IV	0.24
Dharwad	III	0.16	Rourkela	II	0.10
Dehra Dun	IV	0.24	Sadiya	V	0.36
Dharampuri	III	0.16	Salem	III	0.16

			Shillong	V	0.36
Delhi	IV	0.24	Simla	IV	0.24
Durgapur	III	0.16	Sironj	II	0.10
Gangtok	IV	0.24	Solapur	III	0.16
Guwahati	V	0.36	Srinagar	V	0.36
Goa	III	0.16	Surat	III	0.16
Gulbarga	II	0.10	Tarapur	III	0.16
Gaya	III	0.16	Tezpur	V	0.36
Gorakhpur	IV	0.24	Thane	III	0.16
Hyderabad	II	0.10	Thanjavur	II	0.10
Imphal	V	0.36	Thiruvananthapuram	III	0.16
Jabalpur	III	0.16	Tiruchirappali	II	0.10
Jaipur	II	0.10	Thiruvannamalai	III	0.16
Jamshedpur	II	0.10	Udaipur	II	0.10
Jhansi	II	0.10	Vadodara	III	0.16
Jodhpur	II	0.10	Varanasi	III	0.16
Jorhat	V	0.36	Vellore	III	0.16
Kakrapara	III	0.16	Vijayawada	III	0.16
Kalapakkam	III	0.16	Vishakhapatnam	II	0.10

Annex F

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, 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_v) and effective vertical stress (σ'_v) for all potentially liquefiable layers within the deposit.

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

$$r_d = 1 - 0.000765z \quad \text{for } z \leq 9.15 \text{ m and}$$

$$r_d = 1 - 0.0267z \quad \text{for } 9.15 < z \leq 23 \text{ m}$$

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

Step 4: Calculate the *critical stress ratio* induced by the design earthquake, CSR_{eq} , as:

$$CSR_{eq} = 0.65(a_{max} / g)r_d(\sigma_v / \sigma'_v)$$

where σ_v and σ'_v are the total and effective vertical stresses, respectively, at depth z , a_{max} is the peak ground acceleration, and g is the acceleration due to gravity.

Step 5: Correct CSR_{eq} for earthquake magnitude (M_w), stress level and for initial static shear using correction factors k_m , k_σ and k_α , respectively, according to:

$$CSR_L = CSR_{7.5} k_M k_\sigma k_\alpha$$

The correction factors are estimated using Figures F-1, F-2 and F-3 (in combination with figure F-4), respectively.

For assessing liquefaction susceptibility using the SPT go to Step 6a, for the CPT go to Step 6b, and the shear wave velocity go to Step 6c.

Step 6a: Evaluate the *standardized SPT blow count* (N_{60}) which is the standard penetration test blow count for a hammer with an efficiency of 60 percent. Specifications of the "standardized" equipment corresponding to an efficiency of 60 percent are given in Table F-1. If

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

nonstandard equipment is used, N_{60} , is obtained from the equation:

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

where C_{60} is the product of various correction factors. Correction factors recommended by various investigators for some common non-standard SPT configurations are provided in Table F-2. For SPT conducted as per IS: 2131-1981, the energy delivered to the drill rod is 60 percent and hence $C_{60} = 1$ is assumed.

Calculate the *normalized standardized SPT blow count*, $(N_1)_{60}$. $(N_1)_{60}$ is the standardized blow count normalized to an effective overburden pressure of 96 kPa in order to eliminate the influence of confining pressure. This is obtained by the following equation:

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

Stress normalization factor C_N is calculated from following expression:

$$C_N = (P_a / \sigma'_v)^{1/2}$$

Subjected to $C_N \leq 2$, where P_a is the atmospheric pressure. The Critical Resistance Ratio (*CRR*) or the resistance of a soil layer against liquefaction is estimated from Figure F-5 depending on the $(N_1)_{60}$ value representative of the deposit.

Step 6b:

Calculate normalized cone tip resistance, $(q_{c1N})_{cs}$, using $(q_{c1N})_{cs} = K_c (P_a / \sigma'_v)^n (q_c / P_a)$

where q_c is the measured cone tip resistance corrected for thin layers, exponent n has a value of 0.5 for sand and 1 for clay, and K_c is the correction factor for grain characteristics estimated as follows.

$K_c = 1.0$ for $I_c \leq 1.64$ and

$$K_c = -0.403I_c^4 + 5.581I_c^3 - 21.63I_c^2 + 33.75I_c - 17.88 \text{ for } I_c > 1.64$$

The soil behavior type index, I_c , is given by

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

where $Q = [(q_c - \sigma_v) / P_a] (P_a / \sigma'_v)^n$, $F = f / (q_c - \sigma_v) \times 100$, f is the measured sleeve friction and n has the same values as described earlier. Assess susceptibility of a soil to liquefaction using Figure F-6.

Although soils with $I_c > 2.6$ are deemed non-liquefiable, such deposits may soften and deform during earthquakes. General guidance is not available to deal with such possibilities. Softening and deformability of deposits with $I_c > 2.6$ should thus be treated on a material specific basis.

Step 6c:

Calculate normalized shear wave velocity, V_{s1} , for clean sands using: $V_{s1} = V_s \times (P_a / \sigma'_v)^{0.25}$. Assess liquefaction susceptibility of clean sands using Figure F-7.

Step 7: Calculate the factor of safety against initial liquefaction, FS_{liq} , as:

$$FS_{liq} = CSR_L / CRR$$

where CSR_L is as estimated in Step 5 and CRR is from Step 6a, 6b or 6c. When the design

ground motion is conservative, earthquake-related permanent ground deformation is generally small if $FS_{liq} \geq 1.2$.

Table F-1: Recommended “Standardized” SPT Equipment.

Element	Standard Specification
Sampler	Standard split-spoon sampler with: (a) Outside diameter, O.D. = 51 mm, and Inside Diameter, I.D. = 35 mm (constant – i.e., no room for liners in the barrel)
Drill Rods	A or AW-type for depths less than 15.2 m; N- or NW-type for greater depths
Hammer	Standard (safety) hammer with: (a) weight = 63.5 kg; (b) drop = 762 mm (delivers 60% of theoretical free fall energy)
Rope	Two wraps of rope around the pulley
Borehole	100- to 130-mm diameter rotary borehole with bentonite mud for borehole stability (hollow stem augers where SPT is taken through the stem)
Drill Bit	Upward deflection of drilling mud (tricone or baffled drag bit)
Blow Count Rate	30 to 40 blows per minute
Penetration Resistant Count	Measured over range of 150 to 460 mm of penetration into the ground

Table F-2: Correction Factors for Non-Standard SPT Procedures and Equipment.

Correction for	Correction Factor
Nonstandard Hammer Type (<i>DH</i> = doughnut hammer; <i>ER</i> = energy ratio)	$C_{HT} = 0.75$ for <i>DH</i> with rope and ulley $C_{HT} = 1.33$ for <i>DH</i> with trip/auto and <i>ER</i> = 80
Nonstandard Hammer Weight or Height of fall (<i>H</i> = height of fall in mm; <i>W</i> = hammer weight in kg)	$C_{HW} = \frac{H.W}{63.5 \times 762}$
Nonstandard Sampler Setup (standard samples with room for liners, but used without liners)	$C_{SS} = 1.10$ for loose sand $C_{SS} = 1.20$ for dense sand
Nonstandard Sampler Setup (standard samples with room for liners, but liners are used)	$C_{SS} = 0.90$ for loose sand $C_{SS} = 0.80$ for dense sand
Short Rod Length	$C_{RL} = 0.75$ for rod length 0-3 m
Nonstandard Borehole Diameter	$C_{BD} = 1.05$ for 150 mm borehole diameter $C_{BD} = 1.15$ for 200 mm borehole diameter

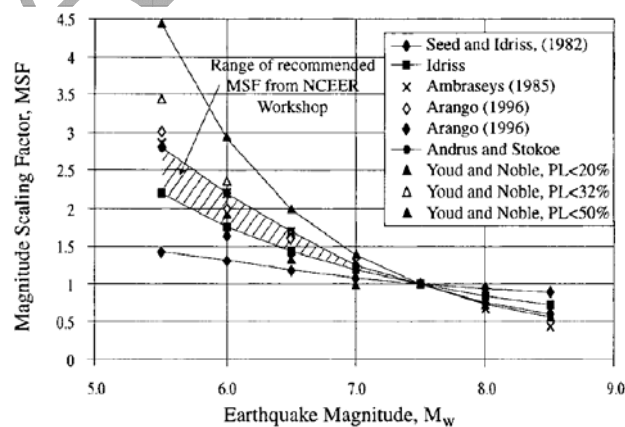
Notes : N = Uncorrected SPT blow count.

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

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

C_N = Correction factor for overburden pressure

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

**Figure F-1: Magnitude Correction factor**

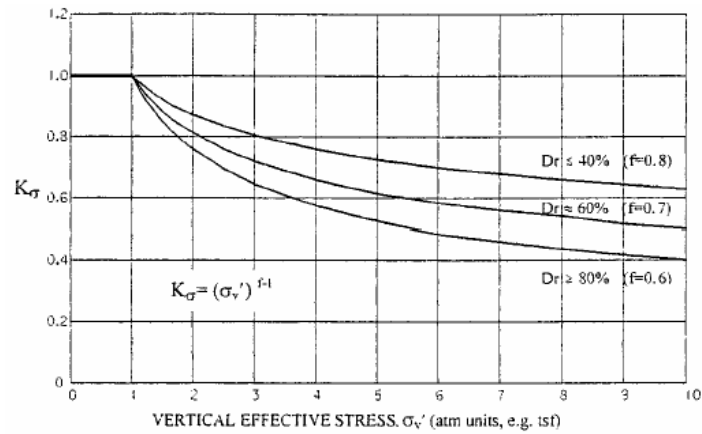


Figure F-2: Stress correction factor

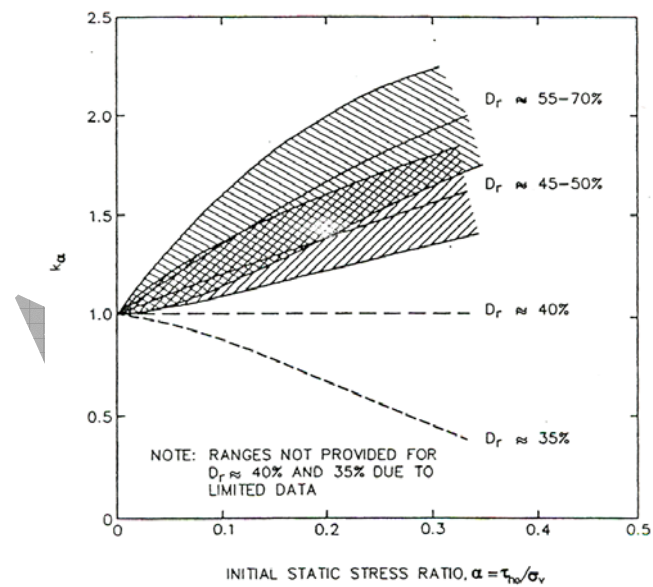


Figure F-3: Correction for initial static shear (Note: Initial static shear for an embankment may be estimated from Figure F-4)

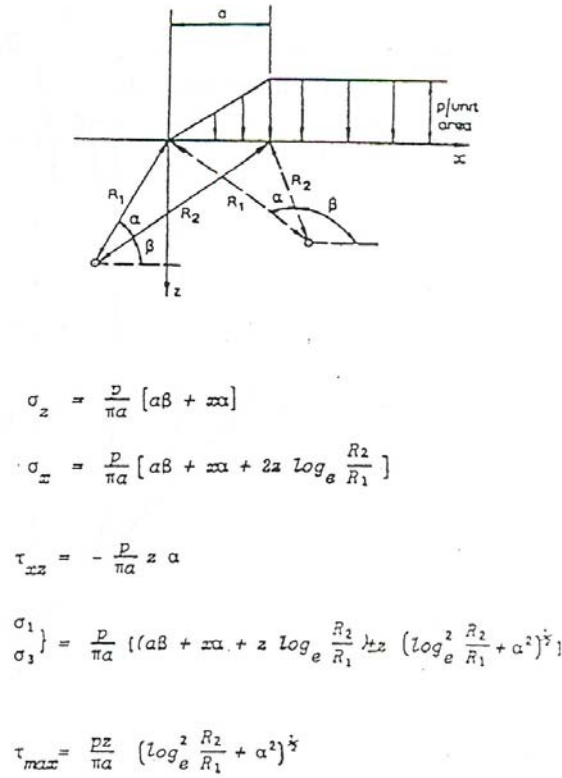
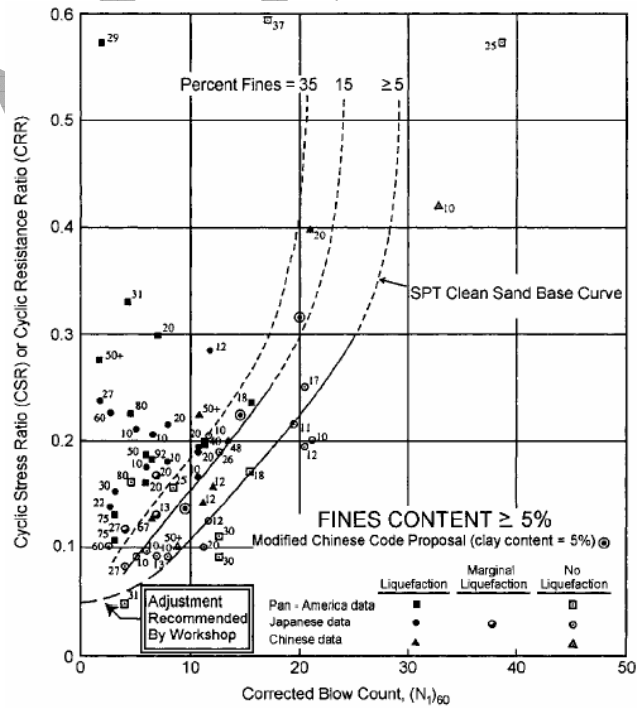


Figure F-4: Initial static shear under an embankment

Figure F-5: Relationship between CRR and $(N_1)_{60}$ for sand for M_w , 7.5 earthquakes

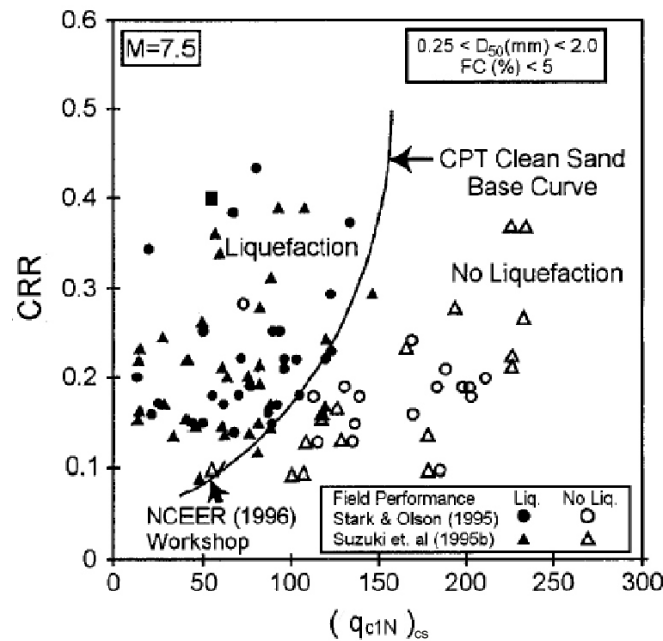


Figure F-6: Relationship between CRR and $(q_{c1N})_{cs}$ for M_w 7.5 earthquakes

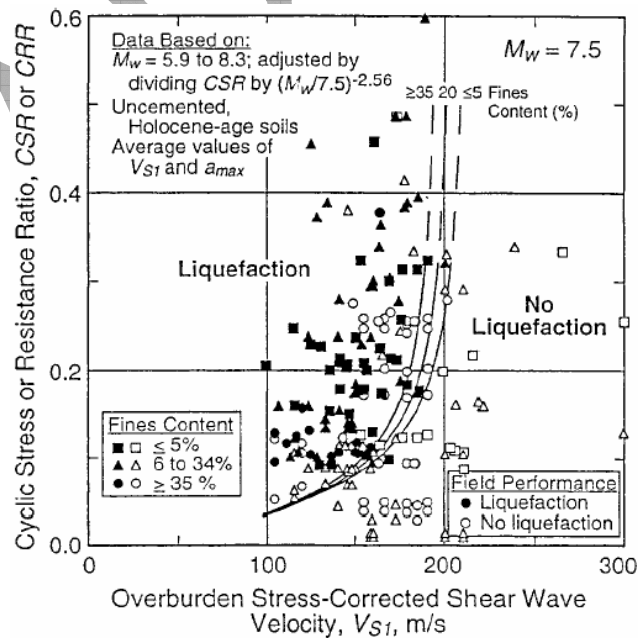


Figure F-7: Relationship between CRR and V_{s1} for M_w 7.5 earthquakes

References for Commentary

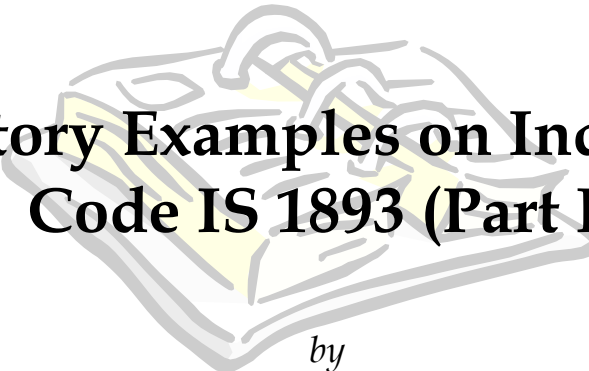
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Explanatory Examples on Indian Seismic Code IS 1893 (Part I)

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- The solved examples included in this document are based on a draft code being developed under IITK-GSDMA Project on Building Codes. The draft code is available at <http://www.nicee.org/IITK-GSDMA/IITK-GSDMA.htm> (document number IITK-GSDMA-EQ05-V3.0).
- This document has been developed through the IITK-GSDMA Project on Building Codes.
- The views and opinions expressed are those of the authors and not necessarily of the GSDMA, the World Bank, IIT Kanpur, or the Bureau of Indian Standards.
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Example 1 – Calculation of Design Seismic Force by Static Analysis Method

Problem Statement:

Consider a four-storey reinforced concrete office building shown in Fig. 1.1. The building is located in Shillong (seismic zone V). The soil conditions are medium stiff and the entire building is supported on a raft foundation. The R. C. frames are infilled with brick-masonry. The lumped weight due to dead loads is 12 kN/m² on floors and 10 kN/m² on the roof. The floors are to cater for a live load of 4 kN/m² on floors and 1.5 kN/m² on the roof. Determine design seismic load on the structure as per new code.

[Problem adopted from Jain S.K, “A Proposed Draft for IS:1893 Provisions on Seismic Design of Buildings; Part II: Commentary and Examples”, Journal of Structural Engineering, Vol.22, No.2, July 1995, pp.73-90]

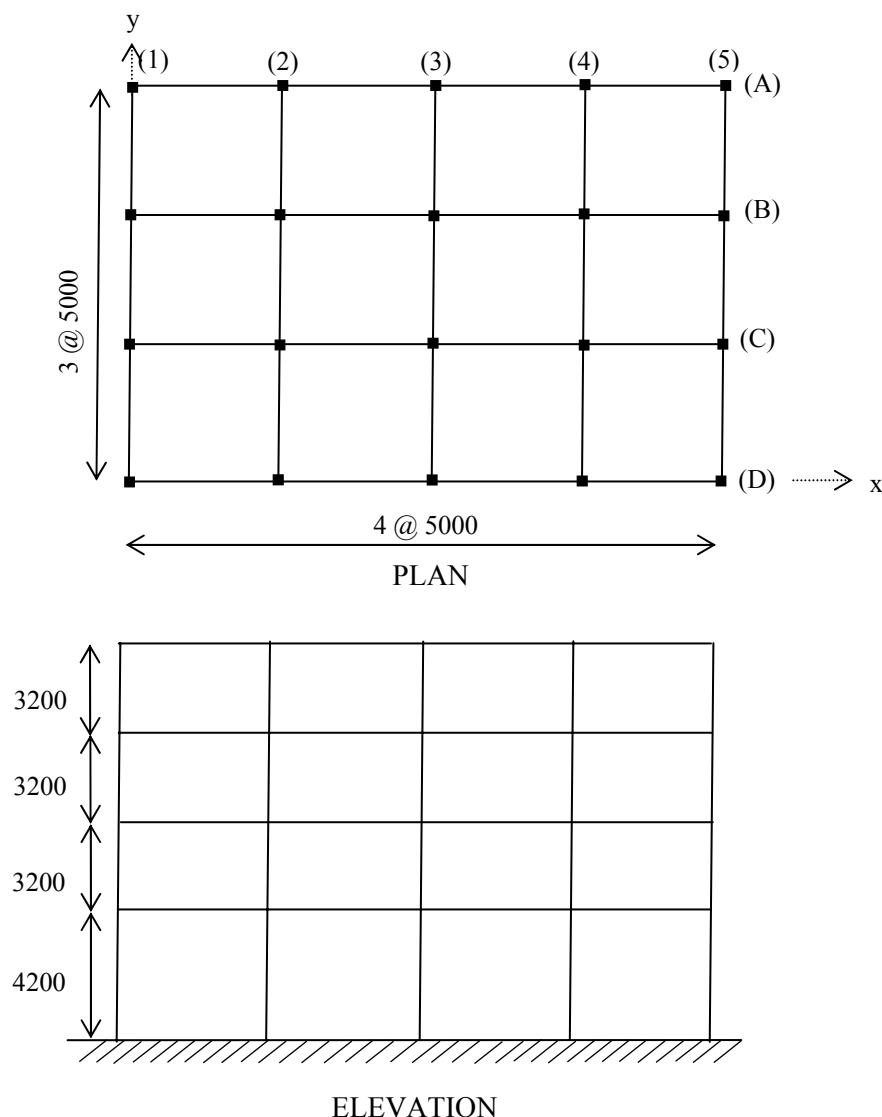


Figure 1.1 – Building configuration

Solution:**Design Parameters:**

For seismic zone V, the zone factor Z is 0.36 (Table 2 of IS: 1893). Being an office building, the importance factor, I , is 1.0 (Table 6 of IS: 1893). Building is required to be provided with moment resisting frames detailed as per IS: 13920-1993. Hence, the response reduction factor, R , is 5.

(Table 7 of IS: 1893 Part 1)

Seismic Weights:

The floor area is $15 \times 20 = 300$ sq. m. Since the live load class is 4kN/sq.m, only 50% of the live load is lumped at the floors. At roof, no live load is to be lumped. Hence, the total seismic weight on the floors and the roof is:

Floors:

$$W_1 = W_2 = W_3 = 300 \times (12 + 0.5 \times 4) = 4,200 \text{ kN}$$

Roof:

$$W_4 = 300 \times 10 = 3,000 \text{ kN}$$

(clause 7.3.1, Table 8 of IS: 1893 Part 1)

Total Seismic weight of the structure,

$$W = \sum W_i = 3 \times 4,200 + 3,000 = 15,600 \text{ kN}$$

Fundamental Period:

Lateral load resistance is provided by moment resisting frames infilled with brick masonry panels. Hence, approximate fundamental natural period:

(Clause 7.6.2. of IS: 1893 Part 1)

EL in X-Direction:

$$T = 0.09h / \sqrt{d}$$

$$= 0.09(13.8) / \sqrt{20} = 0.28 \text{ sec}$$

The building is located on Type II (medium soil).

From Fig. 2 of IS: 1893, for $T = 0.28$ sec, $S_a/g =$

2.5

$$A_h = \frac{ZI}{2R} \frac{S_a}{g} = \frac{0.36 \times 1.0}{2 \times 5} \times 2.5 = 0.09$$

(Clause 6.4.2 of IS: 1893 Part 1)

Design base shear

$$V_B = A_h W = 0.09 \times 15,600 = 1,440 \text{ kN}$$

(Clause 7.5.3 of IS: 1893 Part 1)

Force Distribution with Building Height:

The design base shear is to be distributed with height as per clause 7.7.1. Table 1.1 gives the calculations. Fig. 1.2(a) shows the design seismic force in X-direction for the entire building.

EL in Y-Direction:

$$T = 0.09h / \sqrt{d} = 0.09(13.8) / \sqrt{15} = 0.32 \text{ sec}$$

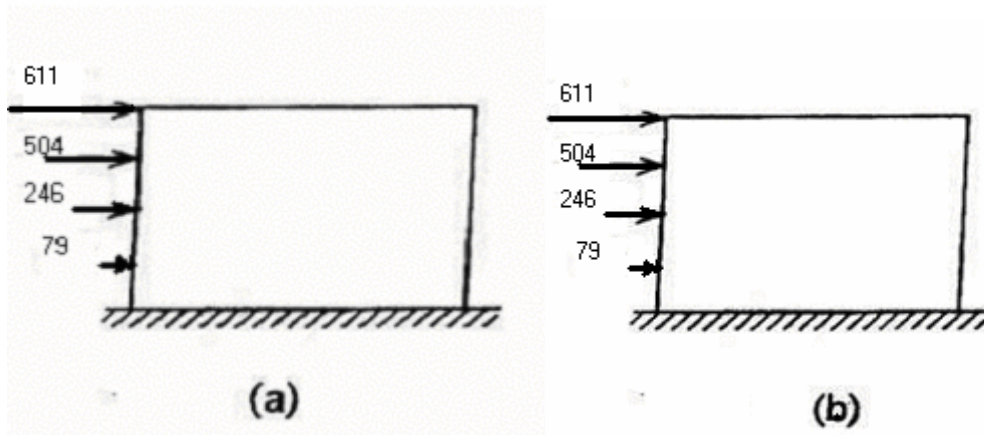
$$\frac{S_a}{g} = 2.5;$$

$$A_h = 0.09$$

Therefore, for this building the design seismic force in Y-direction is same as that in the X-direction. Fig. 1.2(b) shows the design seismic force on the building in the Y-direction.

Table 1.1 – Lateral Load Distribution with Height by the Static Method

Storey Level	W_i (kN)	h_i (m)	$W_i h_i^2 \times (1000)$	$\frac{W_i h_i^2}{\sum W_i h_i^2}$	Lateral Force at i^{th} Level for EL in direction (kN)	
					X	Y
4	3,000	13.8	571.3	0.424	611	611
3	4,200	10.6	471.9	0.350	504	504
2	4,200	7.4	230.0	0.171	246	246
1	4,200	4.2	74.1	0.055	79	79
Σ			1,347.3	1,000	1,440	1,440

**Figure 1.2 -- Design seismic force on the building for (a) X-direction, and (b) Y-direction.**

Example 2 – Calculation of Design Seismic Force by Dynamic Analysis Method

Problem Statement:

For the building of Example 1, the dynamic properties (natural periods, and mode shapes) for vibration in the X-direction have been obtained by carrying out a free vibration analysis (Table 2.1). Obtain the design seismic force in the X-direction by the dynamic analysis method outlined in cl. 7.8.4.5 and distribute it with building height.

Table 2.1 – Free Vibration Properties of the building for vibration in the X-Direction

Natural Period (sec)	Mode 1	Mode 2	Mode 3
	0.860	0.265	0.145
	Mode Shape		
Roof	1.000	1.000	1.000
3 rd Floor	0.904	0.216	-0.831
2 nd Floor	0.716	-0.701	-0.574
1 st Floor	0.441	-0.921	1.016

[Problem adopted from, Jain S.K, “A Proposed Draft for IS: 1893 Provisions on Seismic Design of Buildings; Part II: Commentary and Examples”, Journal of Structural Engineering, Vol.22, No.2, July 1995, pp.73-90]

Solution:

Table 2.2 -- Calculation of modal mass and modal participation factor (clause 7.8.4.5)

Storey Level i	Weight W_i (kN)	Mode 1			Mode 2			Mode 3		
4	3,000	1.000	3,000	3,000	1.000	3,000	3,000	1.000	3,000	3,000
3	4,200	0.904	3,797	3,432	0.216	907	196	-0.831	-3,490	2,900
2	4,200	0.716	3,007	2,153	-0.701	-2,944	2,064	-0.574	-2,411	1,384
1	4,200	0.441	1,852	817	-0.921	-3,868	3,563	1.016	4,267	4,335
Σ	15,600		11,656	9,402		-2,905	8,822		1,366	11,620
$M_k = \frac{[\sum w_i \phi_{ik}]^2}{g \sum w_i \phi_{ik}^2}$		$\frac{11,656^2}{9,402g} = \frac{14,450kN}{g} = 14,45,000 \text{ kg}$			$\frac{2,905^2}{8,822g} = \frac{957kN}{g} = 95,700 \text{ kg}$			$\frac{1,366^2}{11,620g} = \frac{161kN}{g} = 16,100 \text{ kg}$		
% of Total weight		92.6%			6.1%			1.0%		
$P_k = \frac{\sum w_i \phi_{ik}}{\sum w_i \phi_{ik}^2}$		$\frac{11,656}{9,402} = 1.240$			$\frac{-2,905}{8,822} = -0.329$			$\frac{1,366}{11,620} = 0.118$		

It is seen that the first mode excites 92.6% of the total mass. Hence, in this case, codal requirements on number of modes to be considered such that at least 90% of the total mass is excited, will be satisfied by considering the first mode of

vibration only. However, for illustration, solution to this example considers the first three modes of vibration.

The lateral load Q_{ik} acting at i^{th} floor in the k^{th} mode is

$$Q_{ik} = A_{hk} \phi_{ik} P_k W_i$$

(clause 7.8.4.5 c of IS: 1893 Part 1)

The value of A_{hk} for different modes is obtained from clause 6.4.2.

Mode 1:

$$T_1 = 0.860 \text{ sec};$$

$$(S_a / g) = \frac{1.0}{0.86} = 1.16;$$

$$\begin{aligned} A_{h1} &= \frac{ZI}{2R} (S_a / g) \\ &= \frac{0.36 \times 1}{2 \times 5} \times (1.16) \\ &= 0.0418 \end{aligned}$$

$$Q_{i1} = 0.0418 \times 1.240 \times \phi_{i1} \times W_i$$

Mode 2:

$$T_2 = 0.265 \text{ sec};$$

$$(S_a / g) = 2.5;$$

$$\begin{aligned} A_{h2} &= \frac{ZI}{2R} (S_a / g) \\ &= \frac{0.36 \times 1}{2 \times 5} \times (2.5) \\ &= 0.09 \end{aligned}$$

$$Q_{i2} = 0.09 \times (-0.329) \times \phi_{i2} \times W_i$$

Mode 3:

$$T_3 = 0.145 \text{ sec};$$

$$(S_a / g) = 2.5;$$

$$\begin{aligned} A_{h3} &= \frac{ZI}{2R} (S_a / g) \\ &= \frac{0.36 \times 1}{2 \times 5} \times (2.5) \\ &= 0.09 \end{aligned}$$

$$Q_{i3} = 0.09 \times (0.118) \times \phi_{i3} \times W_i$$

Table 2.3 summarizes the calculation of lateral load at different floors in each mode.

Table 2.3 – Lateral load calculation by modal analysis method (earthquake in X-direction)

Floor Level i	Weight W_i (kN)	Mode 1			Mode 2			Mode 3		
		ϕ_{i1}	Q_{i1}	V_{i1}	ϕ_{i2}	Q_{i2}	V_{i2}	ϕ_{i3}	Q_{i3}	V_{i3}
4	3,000	1.000	155.5	155.5	1.000	-88.8	-88.8	1.000	31.9	31.9
3	4,200	0.904	196.8	352.3	0.216	-26.8	-115.6	-0.831	-37.1	-5.2
2	4,200	0.716	155.9	508.2	-0.701	87.2	-28.4	-0.574	-25.6	-30.8
1	4,200	0.441	96.0	604.2	-0.921	114.6	86.2	1.016	45.4	14.6

Since all of the modes are well separated (clause 3.2), the contribution of different modes is combined by the SRSS (square root of the sum of the square) method

$$V_4 = [(155.5)^2 + (88.8)^2 + (31.9)^2]^{1/2} = 182 \text{ kN}$$

$$V_3 = [(352.3)^2 + (115.6)^2 + (5.2)^2]^{1/2} = 371 \text{ kN}$$

$$V_2 = [(508.2)^2 + (28.4)^2 + (30.8)^2]^{1/2} = 510 \text{ kN}$$

$$V_1 = [(604.2)^2 + (86.2)^2 + (14.6)^2]^{1/2} = 610 \text{ kN}$$

(Clause 7.8.4.4a of IS: 1893 Part 1)

The externally applied design loads are then obtained as:

$$Q_4 = V_4 = 182 \text{ kN}$$

$$Q_3 = V_3 - V_4 = 371 - 182 = 189 \text{ kN}$$

$$Q_2 = V_2 - V_3 = 510 - 371 = 139 \text{ kN}$$

$$Q_1 = V_1 - V_2 = 610 - 510 = 100 \text{ kN}$$

(Clause 7.8.4.5f of IS: 1893 Part 1)

Clause 7.8.2 requires that the base shear obtained by dynamic analysis ($V_B = 610 \text{ kN}$) be compared with that obtained from empirical fundamental period as per Clause 7.6. If V_B is less than that from empirical value, the response quantities are to be scaled up.

We may interpret “base shear calculated using a fundamental period as per 7.6” in two ways:

1. We calculate base shear as per Cl. 7.5.3. This was done in the previous example for the same building and we found the base shear as 1,404 kN. Now, dynamic analysis gives us base shear of 610 kN which is lower. Hence, all the response quantities are to be scaled up in the ratio $(1,404/610 = 2.30)$. Thus, the seismic forces obtained above by dynamic analysis should be scaled up as follows:

$$Q_4 = 182 \times 2.30 = 419 \text{ kN}$$

$$Q_3 = 189 \times 2.30 = 435 \text{ kN}$$

$$Q_2 = 139 \times 2.30 = 320 \text{ kN}$$

$$Q_1 = 100 \times 2.30 = 230 \text{ kN}$$

2. We may also interpret this clause to mean that we redo the dynamic analysis but replace the fundamental time period value by T_a ($= 0.28 \text{ sec}$). In that case, for mode 1:

$$T_1 = 0.28 \text{ sec};$$

$$(S_a / g) = 2.5,$$

$$A_{h1} = \frac{ZI}{2R} (S_a / g) \\ = 0.09$$

$$\text{Modal mass times } A_{h1} \\ = 14,450 \times 0.09 \\ = 1,300 \text{ kN}$$

Base shear in modes 2 and 3 is as calculated earlier: Now, base shear in first mode of vibration = 1300 kN, 86.2 kN and 14.6 kN, respectively.

Total base shear by SRSS

$$= \sqrt{1300^2 + 86.2^2 + 14.6^2}$$

$$= 1,303 \text{ kN}$$

Notice that most of the base shear is contributed by first mode only. In this interpretation of Cl 7.8.2, we need to scale up the values of response quantities in the ratio $(1,303/610 = 2.14)$. For instance, the external seismic forces at floor levels will now be:

$$Q_4 = 182 \times 2.14 = 389 \text{ kN}$$

$$Q_3 = 189 \times 2.14 = 404 \text{ kN}$$

$$Q_2 = 139 \times 2.14 = 297 \text{ kN}$$

$$Q_1 = 100 \times 2.14 = 214 \text{ kN}$$

Clearly, the second interpretation gives about 10% lower forces. We could make either interpretation. Herein we will proceed with the values from the second interpretation and compare the design values with those obtained in Example 1 as per static analysis:

Table 2.4 – Base shear at different storeys

Floor Level i	Q (static)	Q (dynamic, scaled)	Storey Shear V (static)	Storey Shear V (dynamic, scaled)	Storey Moment, M (Static)	Storey Moment, M (Dynamic)
4	611 kN	389 kN	611 kN	389 kN	1,907 kNm	1,245 kNm
3	504 kN	404 kN	1,115 kN	793 kN	5,386 kNm	3,782 kNm
2	297 kN	297 kN	1,412 kN	1,090 kN	9,632 kNm	7,270 kNm
1	79 kN	214 kN	1,491 kN	1,304 kN	15,530 kNm	12,750 kNm

Notice that even though the base shear by the static and the dynamic analyses are comparable, there is considerable difference in the lateral load distribution with building height, and therein lies the advantage of dynamic analysis. For instance, the storey moments are significantly affected by change in load distribution.

Example 3 – Location of Centre of Mass

Problem Statement:

Locate centre of mass of a building having non-uniform distribution of mass as shown in the figure 3.1

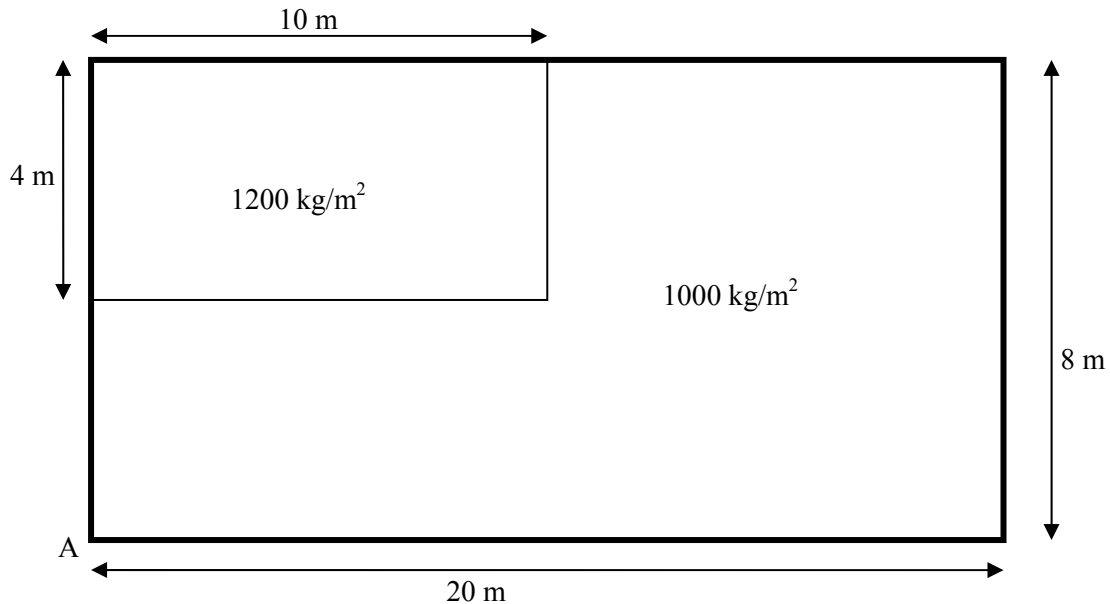


Figure 3.1 –Plan

Solution:

Let us divide the roof slab into three rectangular parts as shown in figure 2.1

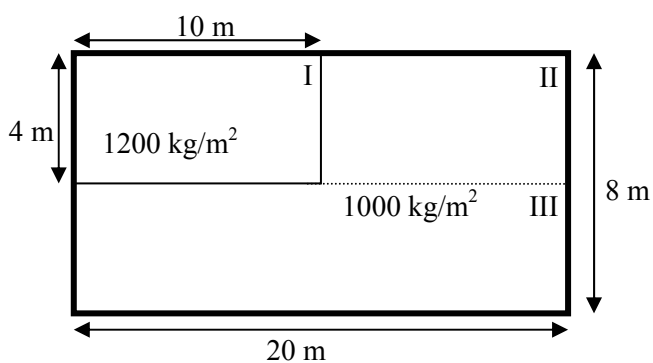


Figure 3.2

Mass of part I is 1200 kg/m^2 , while that of the other two parts is 1000 kg/m^2 .

Let origin be at point A, and the coordinates of the centre of mass be at (X, Y)

$$X = \frac{(10 \times 4 \times 1200) \times 5 + (10 \times 4 \times 1000) \times 15 + (20 \times 4 \times 1000) \times 10}{(10 \times 4 \times 1200) + (10 \times 4 \times 1000) + (20 \times 4 \times 1000)}$$

$$= 9.76 \text{ m}$$

$$Y = \frac{(10 \times 4 \times 1200) \times 6 + (10 \times 4 \times 1000) \times 6 + (20 \times 4 \times 1000) \times 2}{(10 \times 4 \times 1200) + (10 \times 4 \times 1000) + (20 \times 4 \times 1000)}$$

$$= 4.1 \text{ m}$$

Hence, coordinates of centre of mass are (9.76, 4.1)

Example 4 – Location of Centre of Stiffness

Problem Statement:

The plan of a simple one storey building is shown in figure 3.1. All columns and beams are same. Obtain its centre of stiffness.

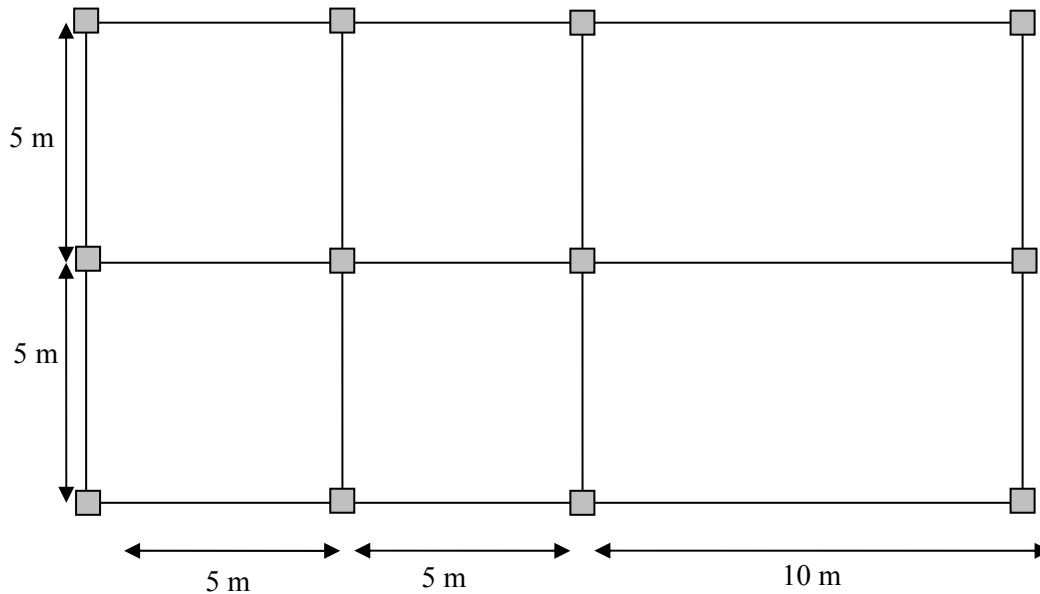


Figure 4.1 –Plan

Solution:

In the X-direction there are three identical frames located at uniform spacing. Hence, the y-coordinate of centre of stiffness is located symmetrically, i.e., at 5.0 m from the left bottom corner.

In the Y-direction, there are four identical frames having equal lateral stiffness. However, the spacing is not uniform. Let the lateral stiffness of each transverse frame be k , and coordinating of center of stiffness be (X, Y) .

$$X = \frac{k \times 0 + k \times 5 + k \times 10 + k \times 20}{k + k + k + k} = 8.75 \text{ m}$$

Hence, coordinates of centre of stiffness are (8.75, 5.0).

Example 5 –Lateral Force Distribution as per Torsion Provisions of IS 1893-2002 (Part 1)

Problem Statement:

Consider a simple one-storey building having two shear walls in each direction. It has some gravity columns that are not shown. All four walls are in M25 grade concrete, 200 thick and 4 m long. Storey height is 4.5 m. Floor consists of cast-in-situ reinforced concrete. Design shear force on the building is 100 kN in either direction.

Compute design lateral forces on different shear walls using the torsion provisions of 2002 edition of IS 1893 (Part 1).

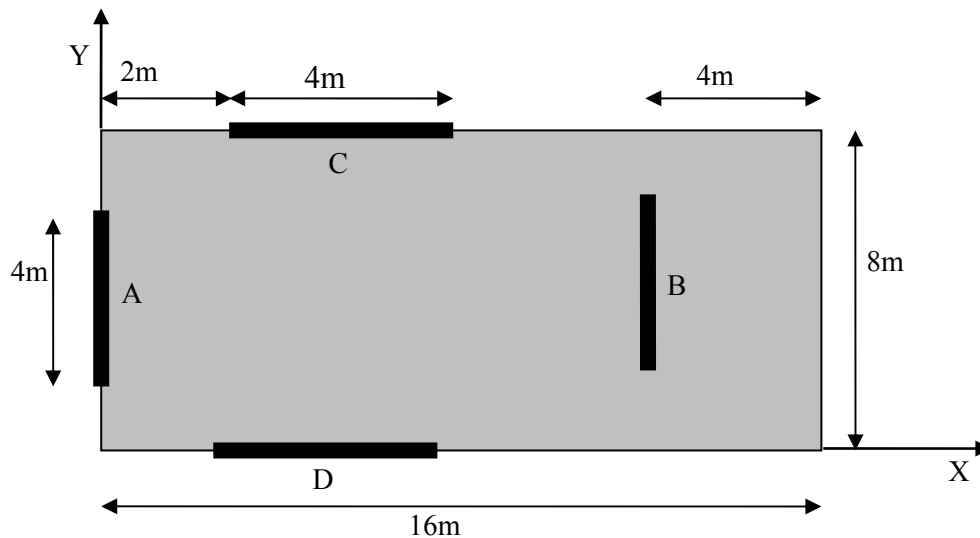


Figure 5.1 – Plan

Solution:

Grade of concrete: M25

$$E = 5000\sqrt{25} = 25000 \text{ N/mm}^2$$

Storey height $h = 4500 \text{ mm}$

Thickness of wall $t = 200 \text{ mm}$

Length of walls $L = 4000 \text{ mm}$

All walls are same, and hence, spaces have same lateral stiffness, k .

Centre of mass (CM) will be the geometric centre of the floor slab, i.e., (8.0, 4.0).

Centre of rigidity (CR) will be at (6.0, 4.0).

EQ Force in X-direction:

Because of symmetry in this direction, calculated eccentricity = 0.0 m

Design eccentricity:

$$e_d = 1.5 \times 0.0 + 0.05 \times 8 = 0.4, \\ \text{and}$$

$$e_d = 0.0 - 0.05 \times 8 = -0.4$$

(Clause 7.9.2 of IS 1893:2002)

Lateral forces in the walls due to translation:

$$F_{CT} = \frac{K_C}{K_C + K_D} F = 50.0 \text{ kN}$$

$$F_{DT} = \frac{K_D}{K_C + K_D} F = 50.0 \text{ kN}$$

Lateral forces in the walls due to torsional moment:

$$F_{iT} = \frac{K_i r_i}{\sum_{i=A,B,C,D} K_i r_i^2} (F e_d)$$

where r_i is the distance of the shear wall from CR.

All the walls have same stiffness, $K_A = K_B = K_C = K_D = k$, and

$$r_A = -6.0 \text{ m}$$

$$r_B = -6.0 \text{ m}$$

$$r_C = 4.0 \text{ m}$$

$$r_D = -4.0 \text{ m,}$$

$$\text{and } e_d = \pm 0.4 \text{ m}$$

Therefore,

$$F_{AR} = \frac{r_A k}{(r_A^2 + r_B^2 + r_C^2 + r_D^2)k} (F e_d)$$

$$= \pm 2.31 \text{ kN}$$

Similarly,

$$F_{BR} = \pm 2.31 \text{ kN}$$

$$F_{CR} = \pm 1.54 \text{ kN}$$

$$F_{DR} = \pm 1.54 \text{ kN}$$

Total lateral forces in the walls due to seismic load in X direction:

$$F_A = 2.31 \text{ kN}$$

$$F_B = 2.31 \text{ kN}$$

$$F_C = \text{Max } (50 \pm 1.54) = 51.54 \text{ kN}$$

$$F_D = \text{Max } (50 \pm 1.54) = 51.54 \text{ kN}$$

EQ Force in Y-direction:

Calculated eccentricity = 2.0 m

Design eccentricity:

$$e_d = 1.5 \times 2.0 + 0.05 \times 16 = 3.8 \text{ m}$$

$$\text{or } = 2.0 - 0.05 \times 16 = 1.2 \text{ m}$$

Lateral forces in the walls due to translation:

$$F_{AT} = \frac{K_A}{K_A + K_B} F = 50.0 \text{ kN}$$

$$F_{BT} = \frac{K_B}{K_A + K_B} F = 50.0 \text{ kN}$$

Lateral force in the walls due to torsional moment: when $e_d = 3.8 \text{ m}$

$$F_{AR} = \frac{r_A k}{(r_A^2 + r_B^2 + r_C^2 + r_D^2)k} (F e_d) = -$$

21.92 kN

Similarly,

$$F_{BR} = 21.92 \text{ kN}$$

$$F_{CR} = -14.62 \text{ kN}$$

$$F_{DR} = 14.62 \text{ kN}$$

Total lateral forces in the walls:

$$F_A = 50 - 21.92 = 28.08 \text{ kN}$$

$$F_B = 50 + 20.77 = 71.92 \text{ kN}$$

$$F_C = -14.62 \text{ kN}$$

$$F_D = 14.62 \text{ kN}$$

Similarly, when $e_d = 1.2 \text{ m}$, then the total lateral forces in the walls will be,

$$F_A = 50 - 6.93 = 43.07 \text{ kN}$$

$$F_B = 50 + 6.93 = 56.93 \text{ kN}$$

$$F_C = -4.62 \text{ kN}$$

$$F_D = 4.62 \text{ kN}$$

Maximum forces in walls due to seismic load in Y direction:

$$F_A = \text{Max } (28.08, 43.07) = 43.07 \text{ kN;}$$

$$F_B = \text{Max } (71.92, 56.93) = 71.92 \text{ kN;}$$

$$F_C = \text{Max } (14.62, 4.62) = 14.62 \text{ kN;}$$

$$F_D = \text{Max } (14.62, 4.62) = 14.62 \text{ kN;}$$

Combining the forces obtained from seismic loading in X and Y directions:

$$F_A = 43.07 \text{ kN}$$

$$F_B = 71.92 \text{ kN}$$

$$F_C = 51.54 \text{ kN}$$

$$F_D = 51.54 \text{ kN.}$$

However, note that clause 7.9.1 also states that "However, negative torsional shear shall be neglected". Hence, wall A should be designed for not less than 50 kN.

Example 6 – Lateral Force Distribution as per New Torsion Provisions

Problem Statement:

For the building of example 5, compute design lateral forces on different shear walls using the torsion provisions of revised draft code IS 1893 (part 1), i.e., IITK-GSDMA-EQ05-V2.0.

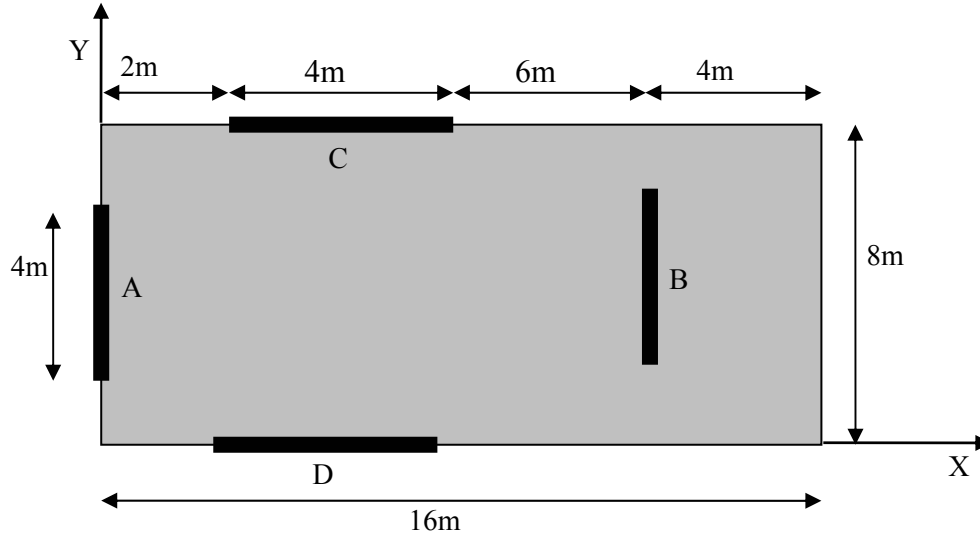


Figure 6.1 – Plan

Solution:

Grade of concrete: M25

$$E = 5000\sqrt{25} = 25000 \text{ N/mm}^2$$

Storey height $h = 4500 \text{ mm}$

Thickness of wall $t = 200 \text{ mm}$

Length of walls $L = 4000 \text{ mm}$

All walls are same, and hence, same lateral stiffness, k .

Centre of mass (CM) will be the geometric centre of the floor slab, i.e., (8.0, 4.0).

Centre of rigidity (CR) will be at (6.0, 4.0).

EQ Force in X-direction:

Because of symmetry in this direction, calculated eccentricity = 0.0 m

Design eccentricity, $e_d = 0.0 \pm 0.1 \times 8 = \pm 0.8$
(clause 7.9.2 of Draft IS 1893: (Part1))

Lateral forces in the walls due to translation:

$$F_{CT} = \frac{K_C}{K_C + K_D} F = 50.0 \text{ kN}$$

$$F_{DT} = \frac{K_D}{K_C + K_D} F = 50.0 \text{ kN}$$

Lateral forces in the walls due to torsional moment:

$$F_{iR} = \frac{K_i r_i}{\sum_{i=A,B,C,D} K_i r_i^2} (F e_d)$$

where r_i is the distance of the shear wall from CR

All the walls have same stiffness, $K_A = K_B = K_C = K_D = k$

$$r_A = -6.0 \text{ m}$$

$$r_B = -6.0 \text{ m}$$

$$r_C = 4.0 \text{ m}$$

$$r_D = -4.0 \text{ m}$$

$$F_{AR} = \frac{r_A k}{(r_A^2 + r_B^2 + r_C^2 + r_D^2) k} (F e_d)$$

$$= -4.62 \text{ kN}$$

Similarly,

$$F_{BR} = 4.62 \text{ kN}$$

$$F_{CR} = 3.08 \text{ kN}$$

$$F_{DR} = -3.08 \text{ kN}$$

Total lateral forces in the walls:

$$F_A = 4.62 \text{ kN}$$

$$F_B = -4.62 \text{ kN}$$

$$F_C = 50 + 3.08 = 53.08 \text{ kN}$$

$$F_D = 50 - 3.08 = 46.92 \text{ kN}$$

Similarly, when $e_d = -0.8$ m, then the lateral forces in the walls will be,

$$F_A = -4.62 \text{ kN}$$

$$F_B = 4.62 \text{ kN}$$

$$F_C = 50 - 3.08 = 46.92 \text{ kN}$$

$$F_D = 50 + 3.08 = 53.08 \text{ kN}$$

Design lateral forces in walls C and D are:

$$F_C = F_D = 53.05 \text{ kN}$$

EQ Force in Y-direction:

Calculated eccentricity = 2.0 m

Design eccentricity,

$$e_d = 2.0 + 0.1 \times 16 = 3.6 \text{ m}$$

or

$$e_d = 2.0 - 0.1 \times 16 = 0.4 \text{ m}$$

Lateral forces in the walls due to translation:

$$F_{AT} = \frac{K_A}{K_A + K_B} F = 50.0 \text{ kN}$$

$$F_{BT} = \frac{K_B}{K_A + K_B} F = 50.0 \text{ kN}$$

Lateral force in the walls due to torsional moment: when $e_d = 3.6$ m

$$F_{AR} = \frac{r_A k}{(r_A^2 + r_B^2 + r_C^2 + r_D^2) k} (F e_d) =$$

$$20.77 \text{ kN}$$

Similarly,

$$F_{BR} = 20.77 \text{ kN}$$

$$F_{CR} = 13.85 \text{ kN}$$

$$F_{DR} = -13.8 \text{ kN}$$

Total lateral forces in the walls:

$$F_A = 50 - 20.77 = 29.23 \text{ kN}$$

$$F_B = 50 + 20.77 = 70.77 \text{ kN}$$

$$F_C = 13.85 \text{ kN}$$

$$F_D = -13.85 \text{ kN}$$

Similarly, when $e_d = 0.4$ m, then the total lateral forces in the walls will be,

$$F_A = 50 - 2.31 = 47.69 \text{ kN}$$

$$F_B = 50 + 2.31 = 53.31 \text{ kN}$$

$$F_C = 1.54 \text{ kN}$$

$$F_D = -1.54 \text{ kN}$$

Maximum forces in walls A and B

$$F_A = 47.69 \text{ kN}, F_B = 70.77 \text{ kN}$$

Design lateral forces in all the walls are as follows:

$$F_A = 47.69 \text{ kN}$$

$$F_B = 70.77 \text{ kN}$$

$$F_C = 53.05 \text{ kN}$$

$$F_D = 53.05 \text{ kN}$$

Example 7 – Design for Anchorage of an Equipment

Problem Statement:

A 100 kN equipment (Figure 7.1) is to be installed on the roof of a five storey building in Simla (seismic zone IV). It is attached by four anchored bolts, one at each corner of the equipment, embedded in a concrete slab. Floor to floor height of the building is 3.0 m, except the ground storey which is 4.2 m. Determine the shear and tension demands on the anchored bolts during earthquake shaking.

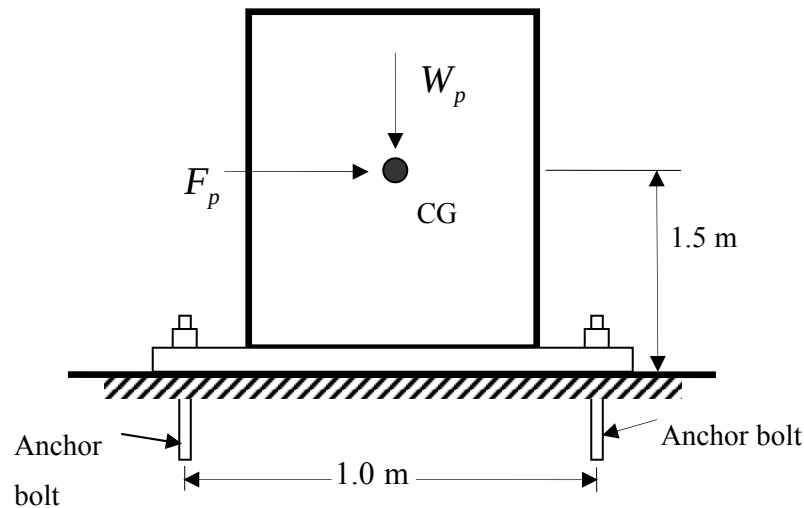


Figure 7.1– Equipment installed at roof

Solution:

Zone factor, $Z = 0.24$ (for zone IV, Table 2 of IS 1893),

Height of point of attachment of the equipment above the foundation of the building, $x = (4.2 + 3.0 \times 4) \text{ m} = 16.2 \text{ m}$,

Height of the building, $h = 16.2 \text{ m}$,

Amplification factor of the equipment, $a_p = 1$ (rigid component, Table 11),

Response modification factor $R_p = 2.5$ (Table 11),

Importance factor $I_p = 1$ (not life safety component, Table 12),

Weight of the equipment, $W_p = 100 \text{ kN}$

The design seismic force

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

$$= \frac{0.24}{2} \left(1 + \frac{16.2}{16.2} \right) \frac{1.0}{2.5} (1) (100) \text{ kN}$$

$$= 9.6 \text{ kN} < 0.1 W_p = 10.0 \text{ kN}$$

Hence, design seismic force, for the equipment

$$F_p = 10.0 \text{ kN.}$$

The anchorage of equipment with the building must be designed for twice of this force (Clause 7.13.3.4 of draft IS 1893)

$$\begin{aligned}\text{Shear per anchor bolt, } V &= 2F_p/4 \\ &= 2 \times 10.0/4 \text{ kN} \\ &= 5.0 \text{ kN}\end{aligned}$$

The overturning moment is

$$\begin{aligned}M_{ot} &= 2.0 \times (10.0 \text{ kN}) \times (1.5 \text{ m}) \\ &= 30.0 \text{ kN-m}\end{aligned}$$

The overturning moment is resisted by two anchor bolts on either side. Hence, tension per anchor bolt from overturning is

$$\begin{aligned}F_t &= \frac{(30.0)}{(1.0)(2)} \text{ kN} \\ &= 15.0 \text{ kN}\end{aligned}$$

Example 8 – Anchorage Design for an Equipment Supported on Vibration Isolator

Problem Statement:

A 100 kN electrical generator of a emergency power supply system is to be installed on the fourth floor of a 6-storey hospital building in Guwahati (zone V). It is to be mounted on four flexible vibration isolators, one at each corner of the unit, to damp the vibrations generated during the operation. Floor to floor height of the building is 3.0 m. except the ground storey which is 4.2 m. Determine the shear and tension demands on the isolators during earthquake shaking.

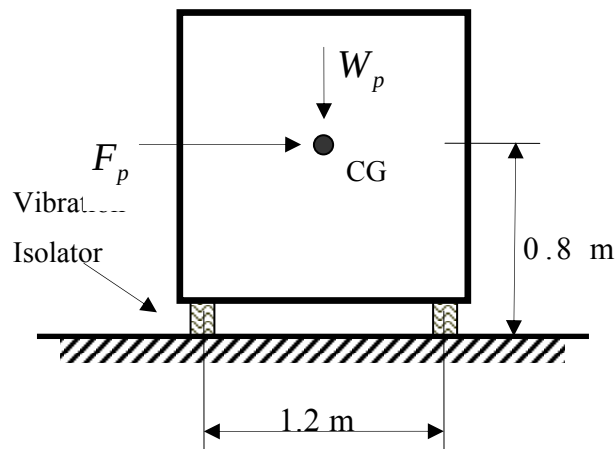


Figure 8.1 – Electrical generator installed on the floor

Solution:

Zone factor, $Z = 0.36$ (for zone V, Table 2 of IS 1893),

Height of point of attachment of the generator above the foundation of the building,

$$x = (4.2 + 3.0 \times 3) \text{ m} \\ = 13.2 \text{ m},$$

Height of the building,

$$h = (4.2 + 3.0 \times 5) \text{ m} \\ = 19.2 \text{ m},$$

Amplification factor of the generator, $a_p = 2.5$ (flexible component, Table 11),

Response modification factor $R_p = 2.5$ (vibration isolator, Table 11),

Importance factor $I_p = 1.5$ (life safety component, Table 12),

Weight of the generator, $W_p = 100 \text{ kN}$

The design lateral force on the generator,

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

$$= \frac{0.36}{2} \left(1 + \frac{13.2}{19.2} \right) \frac{2.5}{2.5} (1.5)(100) \text{ kN}$$

$$= 45.6 \text{ kN}$$

$$0.1W_p = 10.0 \text{ kN}$$

Since the generator is mounted on flexible vibration isolator, the design force is doubled i.e.,

$$F_p = 2 \times 45.6 \text{ kN}$$

$$= 91.2 \text{ kN}$$

Shear force resisted by each isolator,

$$V = F_p/4$$

$$= 22.8 \text{ kN}$$

The overturning moment,

$$M_{ot} = (91.2 \text{ kN}) \times (0.8 \text{ m})$$

$$= 73.0 \text{ kN-m}$$

The overturning moment (M_{ot}) is resisted by two vibration isolators on either side. Therefore, tension or compression on each isolator,

$$F_t = \frac{(73.0)}{(1.2)(2)} \text{ kN}$$

$$= 30.4 \text{ kN}$$

Example 9 – Design of a Large Sign Board on a Building

Problem Statement:

A neon sign board is attached to a 5-storey building in Ahmedabad (seismic zone III). It is attached by two anchors at a height 12.0 m and 8.0 m. From the elastic analysis under design seismic load, it is found that the deflections of upper and lower attachments of the sign board are 35.0 mm and 25.0 mm, respectively. Find the design relative displacement.

Solution:

Since sign board is a displacement sensitive nonstructural element, it should be designed for seismic relative displacement.

Height of level x to which upper connection point is attached, $h_x = 12.0$ m

Height of level y to which lower connection point is attached, $h_y = 8.0$ m

Deflection at building level x of structure A due to design seismic load determined by elastic analysis = 35.0 mm

Deflection at building level y of structure A due to design seismic load determined by elastic analysis = 25.0 mm

Response reduction factor of the building $R = 5$ (special RC moment resisting frame, Table 7)

$$\begin{aligned}\delta_{xA} &= 5 \times 35 \\ &= 175.0 \text{ mm}\end{aligned}$$

$$\begin{aligned}\delta_{yA} &= 5 \times 25 \\ &= 125.0 \text{ mm}\end{aligned}$$

$$\begin{aligned}\text{(i) } D_p &= \delta_{xA} - \delta_{yA} \\ &= (175.0 - 125.0) \text{ mm} \\ &= 50.0 \text{ mm}\end{aligned}$$

Design the connections of neon board to accommodate a relative motion of 50 mm.

(ii) Alternatively, assuming that the analysis of building is not possible to assess deflections under seismic loads, one may use the drift limits (this presumes that the building complies with seismic code).

Maximum interstorey drift allowance as per clause 7.11.1 is IS : 1893 is 0.004 times the storey height, i.e.,

$$\frac{\Delta_{aA}}{h_{sx}} = 0.004$$

$$\begin{aligned}D_p &= R(h_x - h_y) \frac{\Delta_{aA}}{h_{sx}} \\ &= 5 (12000.0 - 8000.0)(0.004) \text{ mm} \\ &= 80.0 \text{ mm}\end{aligned}$$

The neon board will be designed to accommodate a relative motion of 80 mm.

Example: 10 Liquefaction Analysis using SPT data

Problem Statement:

The measured SPT resistance and results of sieve analysis for a site in Zone IV are indicated in Table 10.1. The water table is at 6m below ground level. Determine the extent to which liquefaction is expected for 7.5 magnitude earthquake. Estimate the liquefaction potential and resulting settlement expected at this location.

Table 10.1: Result of the Standard penetration Test and Sieve Analysis

Depth (m)	N_{60}	Soil Classification	Percentage fine
0.75	9	Poorly Graded Sand and Silty Sand (SP-SM)	11
3.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	16
6.75	13	Poorly Graded Sand and Silty Sand (SP-SM)	12
9.75	18	Poorly Graded Sand and Silty Sand (SP-SM)	8
12.75	17	Poorly Graded Sand and Silty Sand (SP-SM)	8
15.75	15	Poorly Graded Sand and Silty Sand (SP-SM)	7
18.75	26	Poorly Graded Sand and Silty Sand (SP-SM)	6

Solution:

Site Characterization:

This site consists of loose to dense poorly graded sand to silty sand (SP-SM). The SPT values ranges from 9 to 26. The site is located in zone IV. The peak horizontal ground acceleration value for the site will be taken as 0.24g corresponding to zone factor $Z = 0.24$

Liquefaction Potential of Underlying Soil

Step by step calculation for the depth of 12.75m is given below. Detailed calculations for all the depths are given in Table 10.2. This table provides the factor of safety against liquefaction (FS_{liq}), maximum depth of liquefaction below the ground surface, and the vertical settlement of the soil due to liquefaction (Δ_v).

$$\frac{a_{max}}{g} = 0.24, M_w = 7.5,$$

$$\gamma_{sat} = 18.5 \text{ kN/m}^3, \gamma_w = 9.8 \text{ kN/m}^3$$

Depth of water level below G.L. = 6.00m

Depth at which liquefaction potential is to be

evaluated = 12.75m

Initial stresses:

$$\sigma_v = 12.75 \times 18.5 = 235.9 \text{ kPa}$$

$$u_0 = (12.75 - 6.00) \times 9.8 = 66.2 \text{ kPa}$$

$$\begin{aligned} \sigma'_v &= (\sigma_v - u_0) = 235.9 - 66.2 \\ &= 169.7 \text{ kPa} \end{aligned}$$

Stress reduction factor:

$$r_d = 1 - 0.015z = 1 - 0.015 \times 12.75 = 0.81$$

Critical stress ratio induced by earthquake:

$$a_{max} = 0.24g, M_w = 7.5$$

$$CSR_{eq} = 0.65 \times (a_{max} / g) \times r_d \times (\sigma_v / \sigma'_v)$$

$$\begin{aligned} CSR_{eq} &= 0.65 \times (0.24) \times 0.81 \times (235.9 / 169.7) \\ &= 0.18 \end{aligned}$$

Correction for SPT (N) value for overburden pressure:

$$(N)_{60} = C_N \times N_{60}$$

$$C_N = 9.79 \left(1 / \sigma'_v \right)^{1/2}$$

$$C_N = 9.79 (1/169.7)^{1/2} = 0.75$$

$$(N)_{60} = 0.75 \times 17 = 13$$

Critical stress ratio resisting liquefaction:

For $(N)_{60} = 13$, fines content of 8%

$$CSR_{7.5} = 0.14 \text{ (Figure F-2)}$$

Corrected Critical Stress Ratio Resisting Liquefaction:

$$CSR_L = CSR_{7.5} k_m k_\alpha k_\sigma$$

k_m = Correction factor for earthquake magnitude other than 7.5 (Figure F-4)

$$= 1.00 \text{ for } M_w = 7.5$$

k_α = Correction factor for initial driving static shear

(Figure F-6)

$$= 1.00, \text{ since no initial static shear}$$

k_σ = Correction factor for stress level larger than 96 kPa (Figure F-5)

$$= 0.88$$

$$CSR_L = 0.14 \times 1 \times 1 \times 0.88 = 0.12$$

Factor of safety against liquefaction:

$$FS_L = CSR_L / CSR_{eq} = 0.12 / 0.18 = 0.67$$

Percentage volumetric strain (% ϵ)

$$\text{For } CSR_{eq} = CSR_{eq} / (k_m k_\alpha k_\sigma)$$

$$= 0.18 / (1 \times 1 \times 0.88) = 0.21$$

$$(N_1)_{60} = 13$$

$$\% \epsilon = 2.10 \text{ (from Figure F-8)}$$

Liquefaction induced vertical settlement (ΔV):

(ΔV) = volumetric strain x thickness of liquefiable level

$$= 2.1 \times 3.0 / 100 = 0.063m = 63mm$$

Summary:

Analysis shows that the strata between depths 6m and 19.5m are liable to liquefy. The maximum settlement of the soil due to liquefaction is estimated as 315mm (Table 10.2)

Table 10.2: Liquefaction Analysis: Water Level 6.00 m below GL (Units: Tons and Meters)

Depth	%Fine	σ_v (kPa)	σ'_v (kPa)	N_{60}	C_N	$(N)_{60}$	r_d	CSR_{eq}	CSR_{eq1}	$CSR_{7.5}$	CSR_L	FS_L	% ϵ	ΔV
0.75	11.00	13.9	13.9	9.00	2.00	18	0.99	0.15	0.14	0.22	0.25	1.67	-	-
3.75	16.00	69.4	69.4	17.00	1.18	20	0.94	0.15	0.14	0.32	0.34	2.27	-	-
6.75	12.00	124.9	117.5	13.00	0.90	12	0.90	0.15	0.15	0.13	0.13	0.86	2.30	0.069
9.75	8.00	180.4	143.6	18.00	0.82	15	0.85	0.17	0.18	0.16	0.15	0.88	1.90	0.057
12.75	8.00	235.9	169.7	17.00	0.75	13	0.81	0.18	0.20	0.14	0.12	0.67	2.10	0.063
15.75	7.00	291.4	195.8	15.00	0.70	10	0.76	0.18	0.21	0.11	0.09	0.50	2.50	0.075
18.75	6.00	346.9	221.9	26.00	0.66	17	0.72	0.18	0.22	0.18	0.15	0.83	1.70	0.051
Total Δ														0.315

Example: 11 Liquefaction Analysis using CPT data

Problem Statement:

Prepare a plot of factors of safety against liquefaction versus depth. The results of the cone penetration test (CPT) of 20m thick layer in Zone V are indicated in Table 11.1. Assume the water table to be at a depth of 2.35 m, the unit weight of the soil to be 18 kN/m^3 and the magnitude of 7.5.

Table 11.1: Result of the Cone penetration Test

Depth (m)	$\overline{q_c}$	$\overline{f_s}$	Depth (m)	$\overline{q_c}$	$\overline{f_s}$	Depth (m)	$\overline{q_c}$	$\overline{f_s}$
0.50	144.31	0.652	7.50	45.46	0.132	14.50	46.60	0.161
1.00	95.49	0.602	8.00	39.39	0.135	15.00	46.77	0.155
1.50	39.28	0.281	8.50	36.68	0.099	15.50	47.58	0.184
2.00	20.62	0.219	9.00	45.30	0.129	16.00	41.99	0.130
2.50	150.93	1.027	9.50	51.05	0.185	16.50	48.94	0.329
3.00	55.50	0.595	10.00	46.39	0.193	17.00	56.69	0.184
3.50	10.74	0.359	10.50	58.05	0.248	17.50	112.90	0.392
4.00	9.11	0.144	11.00	48.94	0.159	18.00	104.49	0.346
4.50	33.69	0.297	11.50	63.75	0.218	18.50	77.75	0.256
5.00	70.69	0.357	12.00	53.93	0.193	19.00	91.58	0.282
5.50	49.70	0.235	12.50	53.60	0.231	19.50	74.16	0.217
6.00	51.43	0.233	13.00	62.39	0.275	20.00	115.02	0.375
6.50	64.94	0.291	13.50	54.58	0.208			
7.00	57.24	0.181	14.00	52.08	0.173			

Solution:

Liquefaction Potential of Underlying Soil

Step by step calculation for the depth of 4.5m is given below. Detailed calculations are given in Table 11.2. This table provides the factor of safety against liquefaction (FS_{liq}).

The site is located in zone V. The peak horizontal ground acceleration value for the site will be taken as $0.36g$ corresponding to zone factor $Z = 0.36$

$$a_{max}/g = 0.36, M_w = 7.5,$$

$$\gamma_{sat} = 18 \text{ kN/m}^3, \gamma_w = 9.8 \text{ kN/m}^3$$

Depth of water level below G.L. = 2.35m

Depth at which liquefaction potential is to be evaluated = 4.5m

Initial stresses:

$$\sigma_v = 4.5 \times 18 = 81.00 \text{ kPa}$$

$$u_0 = (4.5 - 2.35) \times 9.8 = 21.07 \text{ kPa}$$

$$\sigma'_v = (\sigma_v - u_0) = 81 - 21.07 = 59.93 \text{ kPa}$$

Stress reduction factor:

$$r_d = 1 - 0.000765 z$$

$$= 1 - 0.000765 \times 4.5 = 0.997$$

Critical stress ratio induced by earthquake:

$$CSR_{eq} = 0.65 \times (a_{maz} / g) \times r_d \times (\sigma_v / \sigma'_v)$$

$$CSR_{eq} = 0.65 \times (0.36) \times 0.997 \times (81 / 59.93) \\ = 0.32$$

Corrected Critical Stress Ratio Resisting Liquefaction:

$$CSR_L = CSR_{eq} k_m k_\alpha k_\sigma$$

k_m = Correction factor for earthquake magnitude other than 7.5
(Figure F-4)

$$= 1.00 \text{ for } M_w = 7.5$$

k_α = Correction factor for initial driving static shear

(Figure F-6)

$$= 1.00, \text{ since no initial static shear}$$

k_σ = Correction factor for stress level larger than 96 kPa
(Figure F-5)

$$= 1.00$$

$$CSR_L = 0.32 \times 1 \times 1 \times 1 = 0.32$$

Correction factor for grain characteristics:

$$K_c = 1.0 \quad \text{for } I_c \leq 1.64 \text{ and}$$

$$K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2$$

$$\vdots + 33.75 I_c - 17.88 \quad \text{for } I_c > 1.64$$

The soil behavior type index, I_c , is given by

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

$$I_c = \sqrt{(3.47 - \log 42.19)^2 + (1.22 + \log 0.903)^2} \\ = 2.19$$

Where,

$$F = f / (q_c - \sigma_v) \times 100$$

$$F = [29.7 / (3369 - 81)] \times 100 = 0.903 \text{ and}$$

$$Q = [(q_c - \sigma_v) / P_a] (P_a / \sigma'_v)^n$$

$$Q = [(3369 - 81) / 101.35] \times (101.35 / 59.93)^{0.5} \\ = 42.19$$

$$K_c = -0.403(2.19)^4 + 5.581(2.19)^3 - 21.63(2.19)^2 \\ \vdots + 33.75(2.19) - 17.88 = 1.64$$

Normalized Cone Tip Resistance:

$$(q_{c1N})_{cs} = K_c (P_a / \sigma'_v)^n (q_c / P_a)$$

$$(q_{c1N})_{cs} = 1.64 (101.35 / 59.93)^{0.5} (3369 / 101.35) \\ = 70.77$$

Factor of safety against liquefaction:

$$\text{For } (q_{c1N})_{cs} = 70.77,$$

$$CRR = 0.11 \text{ (Figure F-6)}$$

$$FS_{liq} = CRR / CSR_L$$

$$FS_{liq} = 0.11 / 0.32 = 0.34$$

Summary:

Analysis shows that the strata between depths 0-1m are liable to liquefy under earthquake shaking corresponding to peak ground acceleration of 0.36g. The plot for depth verses factor of safety is shown in Figure 11.1

Table 11.2: Liquefaction Analysis: Water Level 2.35 m below GL (Units: kN and Meters)

Depth	σ_v	σ'_v	r_d	q_c (kPa)	f_s (kPa)	CSR_{eq}	CSR_L	F	Q	Ic	Kc	(qc1N)cs	CRR	FS_{liq}
0.50	9.00	9.00	1.00	14431	65.20	0.23	0.23	0.45	241.91	1.40	1.00	242.06	100.00	434.78
1.00	18.00	18.00	1.00	9549	60.20	0.23	0.23	0.63	159.87	1.63	1.00	160.17	100.00	434.78
1.50	27.00	27.00	1.00	3928	28.10	0.23	0.23	0.72	65.43	1.97	1.27	83.53	0.13	0.57
2.00	36.00	36.00	1.00	2062	21.90	0.23	0.23	1.08	33.54	2.31	1.99	68.04	0.11	0.47
2.50	45.00	43.53	1.00	15093	102.70	0.24	0.24	0.68	226.55	1.53	1.00	227.23	100.00	416.67
3.00	54.00	47.63	1.00	5550	59.50	0.26	0.26	1.08	79.10	2.01	1.31	105.02	0.19	0.73
3.50	63.00	51.73	1.00	1074	35.90	0.28	0.28	3.55	13.96	2.92	5.92	87.81	0.14	0.50
4.00	72.00	55.83	1.00	911	14.40	0.30	0.30	1.72	11.15	2.83	5.01	60.64	0.10	0.33
4.50	81.00	59.93	1.00	3369	29.70	0.32	0.32	0.90	42.19	2.19	1.64	70.77	0.11	0.34
5.00	90.00	64.03	1.00	7069	35.70	0.33	0.33	0.51	86.63	1.79	1.10	96.60	0.16	0.48
5.50	99.00	68.13	1.00	4970	23.50	0.34	0.34	0.48	58.62	1.93	1.22	72.68	0.12	0.35
6.00	108.00	72.23	1.00	5143	23.30	0.35	0.35	0.46	58.85	1.92	1.21	72.45	0.12	0.34
6.50	117.00	76.33	1.00	6494	29.10	0.36	0.36	0.46	72.50	1.83	1.13	83.61	0.13	0.36
7.00	126.00	80.43	0.99	5724	18.10	0.36	0.36	0.32	62.00	1.83	1.13	71.56	0.11	0.31
7.50	135.00	84.53	0.99	4546	13.20	0.37	0.37	0.30	47.66	1.92	1.21	59.46	0.10	0.27
8.00	144.00	88.63	0.99	3939	13.50	0.38	0.38	0.36	40.04	2.02	1.33	55.18	0.10	0.26
8.50	153.00	92.73	0.99	3668	9.90	0.38	0.38	0.28	36.26	2.02	1.33	50.45	0.09	0.24
9.00	162.00	96.83	0.99	4530	12.90	0.39	0.39	0.30	44.09	1.95	1.24	56.79	0.10	0.26
9.50	171.00	100.93	0.75	5105	18.50	0.30	0.30	0.37	48.78	1.95	1.24	62.62	0.10	0.33
10.00	180.00	105.03	0.73	4639	19.30	0.29	0.29	0.43	43.22	2.02	1.33	59.94	0.10	0.34
10.50	189.00	109.13	0.72	5805	24.80	0.29	0.29	0.44	53.40	1.95	1.23	68.16	0.11	0.38
11.00	198.00	113.23	0.71	4894	15.90	0.29	0.29	0.34	43.84	1.98	1.27	58.01	0.10	0.34
11.50	207.00	117.33	0.69	6375	21.80	0.29	0.29	0.35	56.56	1.88	1.17	68.51	0.11	0.38
12.00	216.00	121.43	0.68	5393	19.30	0.28	0.28	0.37	46.67	1.97	1.26	61.23	0.10	0.36
12.50	225.00	125.53	0.67	5360	23.10	0.28	0.28	0.45	45.53	2.01	1.31	62.48	0.10	0.36
13.00	234.00	129.63	0.65	6239	27.50	0.28	0.28	0.46	52.39	1.96	1.25	68.09	0.11	0.39
13.50	243.00	133.73	0.64	5458	20.80	0.27	0.27	0.40	44.79	2.00	1.29	60.67	0.10	0.37
14.00	252.00	137.83	0.63	5208	17.30	0.27	0.27	0.35	41.93	2.00	1.30	57.21	0.10	0.37
14.50	261.00	141.93	0.61	4660	16.10	0.26	0.26	0.37	36.68	2.06	1.39	53.90	0.09	0.35
15.00	270.00	146.03	0.60	4677	15.50	0.26	0.26	0.35	36.23	2.06	1.38	53.24	0.09	0.35
15.50	279.00	150.13	0.59	4758	18.40	0.25	0.25	0.41	36.31	2.08	1.43	55.02	0.10	0.40
16.00	288.00	154.23	0.57	4199	13.00	0.25	0.25	0.33	31.28	2.11	1.47	49.44	0.09	0.36
16.50	297.00	158.33	0.56	4894	32.90	0.25	0.25	0.72	36.29	2.19	1.65	63.63	0.10	0.40
17.00	306.00	162.43	0.55	5669	18.40	0.24	0.24	0.34	41.80	2.00	1.30	57.28	0.10	0.42
17.50	315.00	166.53	0.53	11290	39.20	0.24	0.24	0.36	84.48	1.73	1.06	91.71	0.15	0.63
18.00	324.00	170.63	0.52	10449	34.60	0.23	0.23	0.34	76.99	1.75	1.07	85.35	0.14	0.61
18.50	333.00	174.73	0.51	7775	25.60	0.23	0.23	0.34	55.92	1.88	1.17	68.46	0.11	0.48
19.00	342.00	178.83	0.49	9158	28.20	0.22	0.22	0.32	65.48	1.81	1.11	75.57	0.12	0.55
19.50	351.00	182.93	0.48	7416	21.70	0.22	0.22	0.31	51.89	1.89	1.18	64.35	0.10	0.45
20.00	360.00	187.03	0.47	11502	37.50	0.21	0.21	0.34	80.93	1.73	1.06	88.47	0.14	0.67

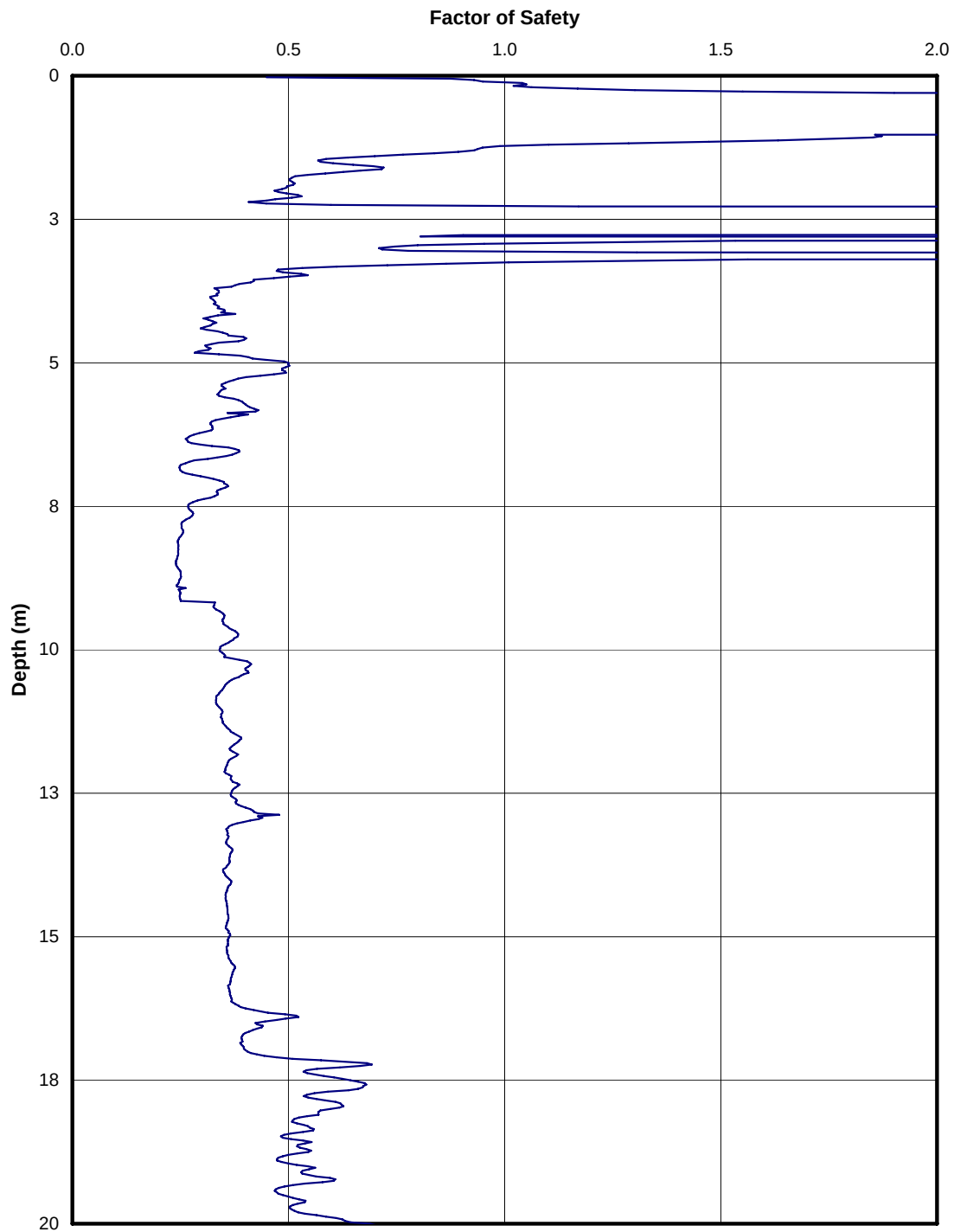


Figure 11.1: Factor of Safety against Liquefaction