Guidebook
on Earthquake Resistant Design and Construction

Building Materials & Technology Promotion Council
Ministry of Housing & Urban Poverty Alleviation
Government of India
New Delhi

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FOREWORD

As a fallout of 2001 Bhuj Earthquake, disaster mitigation & management has been given top priority by Govt. of India. Some of the notable contributions have been enactment of Disaster Management Act in 2005, National Disaster Management Policy in 2009 and now National Disaster management plan in 2016. Also, globally, through Sendai Framework for Disaster Risk Reduction 2015-2030, all countries are committed to enhance disaster resilience and adopt best practices from all over the globe. Also, Sustainable Development Goals 2015-30, lays down emphasis on making cities safe, resilient & sustainable and recently held COP 21 Paris agreement speaks about the issue of increase disaster risks from the adverse impact of climate change. Out of several hydro-metrological & geo hazards, Earthquake has been one of the most lethal hazard taking away human lives in matter of seconds. As per estimates, the loss of lives per year due to earthquake is more than 15000 globally. It is a known fact that the main catalyst in triggering loss of human lives has been our built environment in particular our buildings. There are gaps as regards construction practices in vogue, the knowledge about earthquake resistant construction, the design principles, the associated codes and whole gamut of other parameters such as capacities, education, implementation & awareness. BMTPC in its endeavor to have earthquake resilient India has been in forefront and brings out simple to comprehend guidelines, manuals, illustrations which can help not only professionals as well as common people so as to understand the complexities of earthquake resistant design & construction. Some of the milestones of BMTPC has been Do's & Don'ts, Earthquake TIPS, Vulnerability Atlas of India and district wise Earthquake Hazard Maps for the entire country. Nevertheless, a lot still is desired and to be achieved to reach out to the common people and spread awareness so that there are no deaths during future events. Taking this forward, BMTPC joined hands with Department of Earthquake Engineering (erstwhile School of Earthquake Engineering), IIT, Roorkee to prepare a document which in common parlance though diagrammatic representations explains the earthquake design & construction principles keeping in mind the common man.

It is indeed a matter of great gratification for me to present this Guidebook on the World Habit Day 2016 with a theme 'Housing at the Centre'. The manual has three sections and covers all aspects of earthquake engineering ranging from non-engineered construction to RCC, multistoried buildings, thumb rules to classical design & construction principles and also seismic evaluation and retrofit. The meticulous job done by Dr. Pankaj Agarwal, Professor, Department of Earthquake Engineering is laudable as it is difficult to translate complex engineering into simple comprehensible language. I sincerely hope that document will serve as a useful resource document to dispel the myths of earthquake engineering and help citizens of India to build better & resilient houses which will cause no loss to human lives.

Let us transform India by building safe resilient houses

7th Day of September, 2016 — Dr. Shailesh Kr. Agrawal
New Delhi — Executive Director, BMTPC

PREFACE

The **"Guidebook on Earthquake Resistant Design and Construction"** has been prepared for Building Materials and Promotion Technology Council (BMPTC) under a project. The purpose of preparation of the guidebook is that a common man who has no idea/exposure to earthquake engineering can make his house seismically safe. However, it is very difficult for the author of the guidebook to entirely eliminate all the intricacies of earthquake engineering and present the matter in a fashion understandable to all. Here an attempt has been made so that the owner of a house/building can ensure the safety of his house from the earthquake point of view. Three sections in the guidebook have been prepared to fill this objective.

The first Section entitled **"How to make a dream house earthquake safe"** is a guideline to construct a masonry house consisting of brick, block, stone, adobe etc. with mortar. Such type of construction is called load bearing walls construction. Most of the Indian population is living in this type of constructions. Past earthquakes in India reveal that most of the causalities have occurred only due to collapse of these constructions. Therefore, it is necessary to reduce earthquake disaster in a common man's house making it seismically safe.

The second Section entitled **"How to ensure the seismic safety of multistory reinforced concrete buildings"** encompasses buildings in which people live in flats i.e. frame construction with beam, columns, shear wall etc. This type of construction is safer than the first type of construction if it is made on the basis of BIS code recommendations. Two BIS codes namely IS 1893 and IS 13920 are generally used for the earthquake resistant construction of multi-storey framed buildings. These codes are not easily to understand for our design professional and architects since they are accustomed to the conventional design but not earthquake resistant design. The main reason is that, earthquake engineering in the past was available only at a few places and most of the engineering colleges either did not include earthquake engineering in their curricula or at the most one or two optional courses. Therefore, the graduates of civil engineering either had no idea about earthquake engineering or only a little bit which was not sufficient for a design professional. However, after the Bhuj earthquake on January 26, 2001 there have been number of technical programmes or schemes launched by different ministries with the help of technical institutions on earthquake engineering education in the form of short term courses.

The final and third Section entitled **"How to Analyze, Design, Evaluate and Retrofit Multistoried RC Framed Construction for Earthquake forces"** is useful for those professionals who have interest in seismic analysis and design of multi-storey reinforced concrete buildings. This guidebook will help to analyze the structure for earthquake loads as per IS 1893 and design the same with the help of IS 13920. A number of practical examples have been given so that one can easily understand the clauses of these codes.

This guidebook not only cover the details of earthquake safe construction but also provide direction to evaluate the seismic capacity and vulnerabilities of a house/ building for future earthquake. If the house/building is found seismically deficient, a number of techniques have been suggested to increase/upgrade their seismic capacity for the future earthquake. These techniques are called retrofitting measures. The guidebook also suggest measures to repair and retrofit an earthquake damaged house/building. In short, the guidebook have covered all the details to make earthquake safe construction (new or existing) of a house or multistory reinforced concrete buildings.

The format of the guidebook has also proved to be another challenging task as to how each constructional detail is to be explained; to what extent and in what manner so that it can implement in the field and at the same time the study of guidebook may not be boring. Therefore, the guidebook have been prepared in the form of question – answer format to maintain the reader's interest. These questions are almost independent so that the reader may fulfill his query by selecting the particular question or part. The author has provided the minimum required information alongwith the possible diagram/sketches.

I hope this guidebook will fulfill the requirement and serve the desired purpose. To prepare the guidebook a bulk of literature available at institute's library as well as internet has been utilized and proper credit is given to best of my knowledge. Up-to-date and advanced information is included in preparation of the guidebook. A selected list of main documents used in the preparation has been given at the end. The author has also invited constructive/positive comments so that the guidebook may further be improved and be successful in their objectives. Finally I am thankful to BMTPC for providing the help and particularly Dr. Shailesh Kumar Agarwal, Executive Director for inspiring to prepare such guidebook.

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Section

I

How

to

Make A Dream House

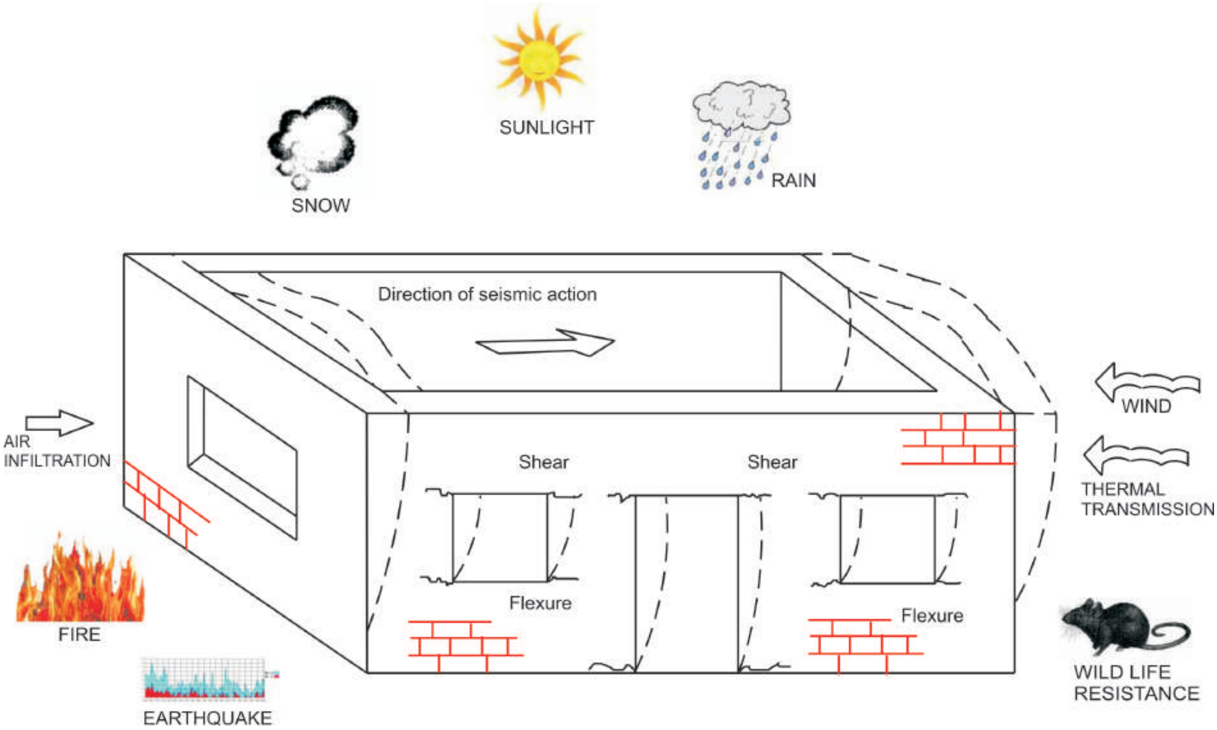
Earthquake Resistant?

Chapter 1

A House and its Behavior during Earthquake

1.1 A House?

A house is living place that protects from the ill effects of environmental forces and weather extremes such as heat, cold, rains, stormy winds, loud sounds and vibrations providing a comfortable atmosphere for living. It also provides privacy and security including limiting entry by unwanted elements. But imagine! What will happen if the safety of the house comes in trouble in the aftermath of natural forces like earthquake?



Concept of a house

1.2 How a house is built?

If a house is observed while being built one may notice a number of stages and operations which are carried out to construct it such as

Digging of a trench for the foundation - to ensure that the house may neither settle, slide nor overturn under forces like wind, earthquakes, etc.

Laying of foundation on a soil stratum - to transmit the loads (horizontal and vertical) acting on the house to the ground

Laying the plinth – to provide a uniform horizontal leveled surface to construct the super-structure which is visible. *The part below the plinth level is called sub-structure while above the plinth level it is called superstructures.*

Laying of damp-proof-course (dpc) - to restrain penetration of moisture from the ground to the superstructures

Construction of external walls – to protect the house from the ill effects of natural and environmental forces minimizing interference by others and acting as a medium for transferring loads to foundation. *The walls also act as load bearing walls for vertical loads and shear walls to resist lateral loads arising due to earthquake, wind, blast etc.*

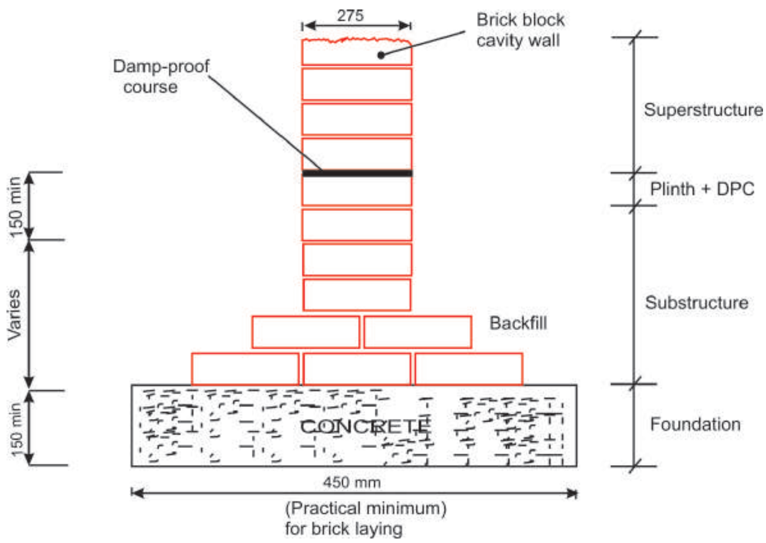
Construction of internal walls – to provide internal management of the house and rest with minimum interference by member themselves.

Laying of roof – to provide a shelter from natural forces, to unite all the external and internal walls under the vertical and lateral loads and to provide a horizontal surface for the upper storey. It is also called **diaphragm** as it plays vital role in transferring the lateral loads to the foundation

Laying of floor – to provide a smooth horizontal surface for movement that divides the house in vertical segments called stories

Construction of parapet wall- to protect from falling of the object or a person from the roof

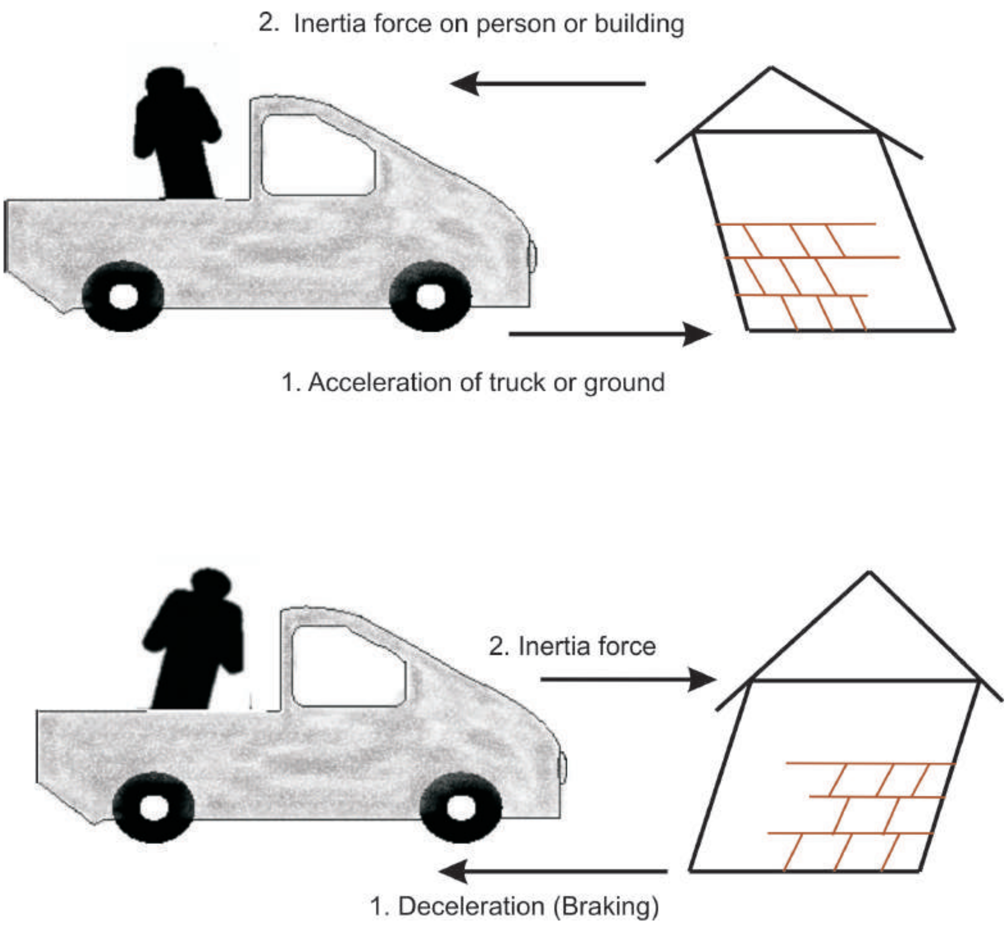
Different types of structural layout may be possible for a house but basically it is primary like a box whose upper and lower sides are roof and floor respectively and vertical sides are wall. These sequences will continue for the next storey also, except substructure part. This type of construction in itself is very robust to resist the ***particularly vertical load.***



Sequence for a construction of a house

1.3 How does an earthquake affect the house?

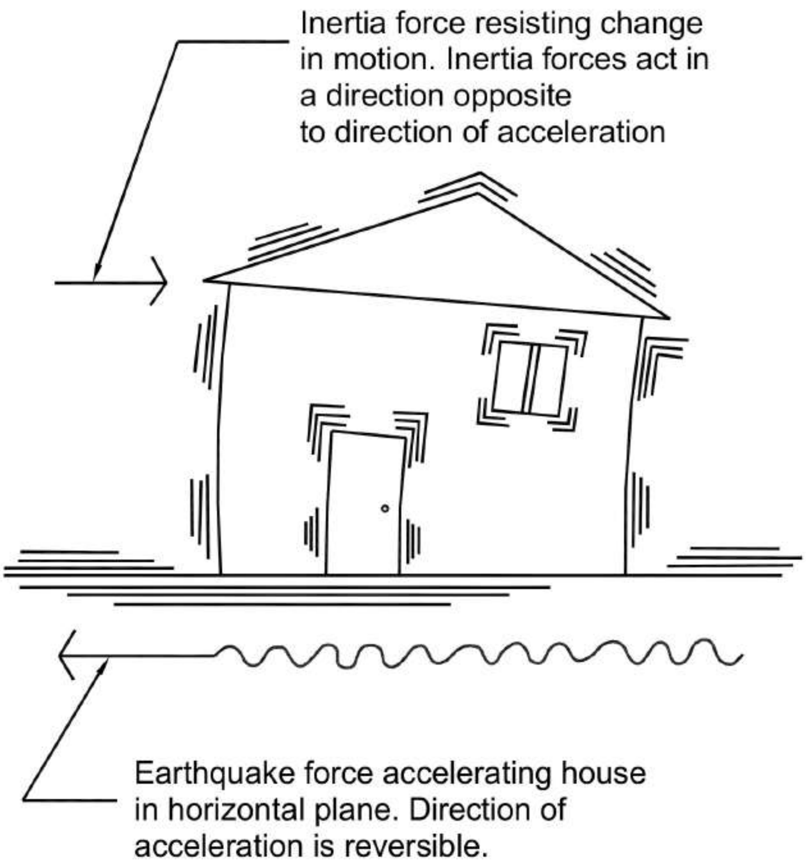
First of all, know about the earthquake and its consequences. The phenomenon of an earthquake can be simply explained like sudden throwing of a piece of stone in a large calm pond of water. As the water body behaves consequently, similarly the earth surface or ground behaves after an earthquake. Technically, an earthquake is a sudden tremor or movement of earth's crust originating naturally at or below the surface, transmitting the seismic waves in all directions horizontally and vertically. This leads to shaking of ground which in turn imparts horizontal forces due to inertia of the house which is initially at rest. It is just as we experience inertia forces in daily life e.g. moving or traveling in a car or bus; as driver suddenly applies brakes we move forward and as he accelerates we move backwards. Inertia force is instantaneously felt in a direction opposite to the direction of base acceleration.



A house during earthquake

1.4 Why does a house damage during an earthquake?

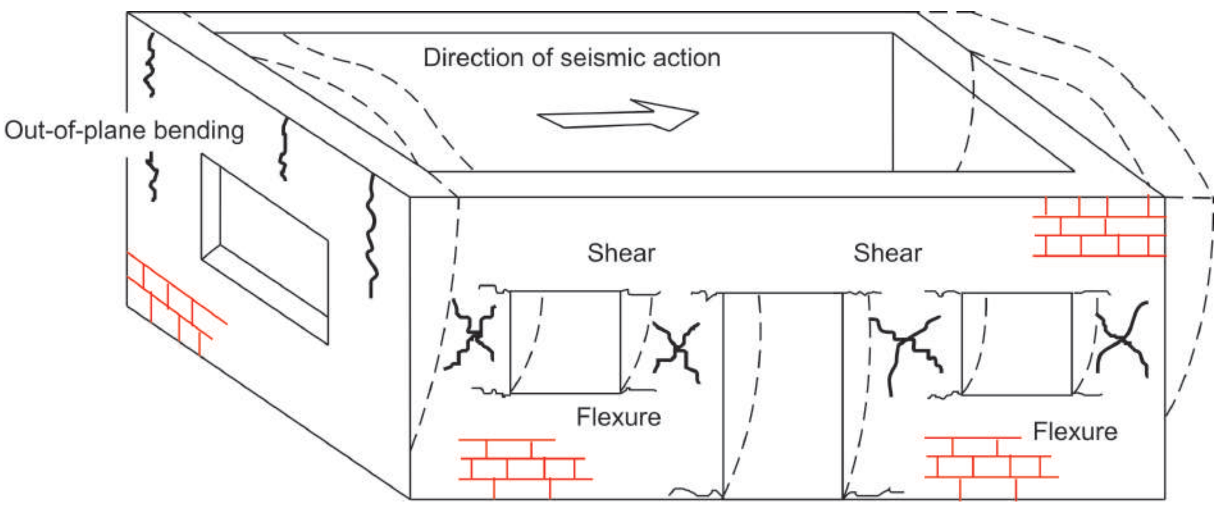
A house is generally made to resist vertical loads such as weight of the building itself, weight of persons and environmental loads (snow, rains etc.). Fortunately all the forces often act in vertical direction except wind or earthquake. Wind forces are not so intense to affect the stability of a house unless constructed in wind prone areas. But earthquake forces, generally proportional to the weight of house are called inertia forces act in horizontal directions. It is very much like our natural resistance for vertical loads since a gravitational force always acts on us vertically downwards but if someone suddenly pulls or pushes us we generally destabilize and may fall. Similarly, a house resists gravity loads but can not endure horizontal loads because lack of resistance to such loads, which is mainly responsible for damages or collapses in earthquake.



Deformation and damage of a house during earthquake[45]

1.5 What type of failure may occur in a house?

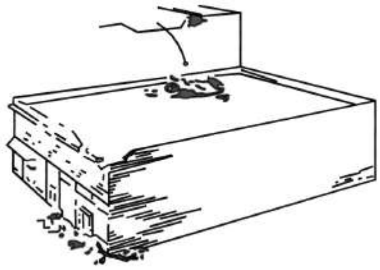
The observation of a house plan may reveal the presence of walls in two orthogonal directions. When the ground beneath the structure moves, motion is induced in all directions. To understand simply lets resolve the base motion in three directions; two along the plan of house in which the walls of the house are located and one in vertical direction i.e. along the height of the house. The house normally resists vertical components due to its natural resistance to gravity loads except in epicentral region where induced vertical accelerations could be large. *The vertical component of motion is generally weaker than horizontal components and is usually less than 67% of the horizontal ones.* All the walls parallel to one horizontal components of earthquake motion behave as *in-plane walls* or *shear wall* and perpendicular walls simultaneously resisting out-of-plane bending by flexure behave as flexure walls. The same explanation holds for the motion in orthogonal direction.



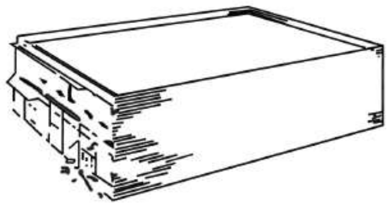
Typical failure of a house during earthquake[23]

1.6 What are the possible ways of damage in a house during earthquake?

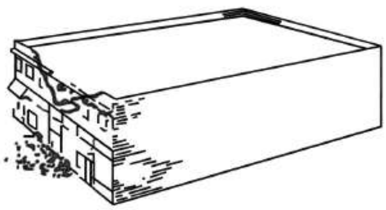
The structure of a house may be different depending upon type of construction, sites, structural topology which varies in different regions but the damage by an earthquake may be identified uniformly. The most common type of damage noticed in past earthquakes may be categorised as; Out-of-plane failure of a wall; Damage around openings, In-plane wall failure; Excessive diaphragm failure; Wall- diaphragm tie failure; Parapet failure; Non-parapet failure hazard such as balconies, overhangs etc.



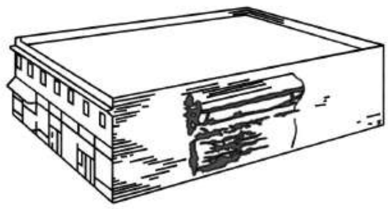
Parapet failure



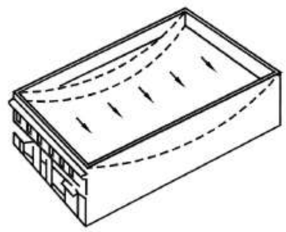
Nonparapet falling hazards



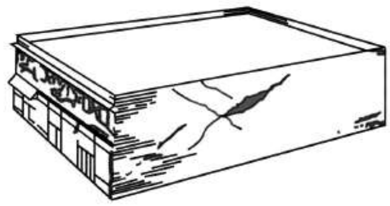
Wall-diaphragm tie failure



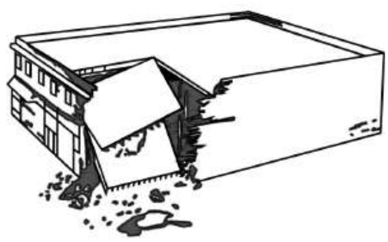
Wall failure in bending between diaphragm



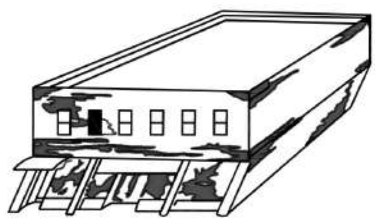
Excessive diaphragm deflection



Inplance wall failure



Roof and / or floor collapse



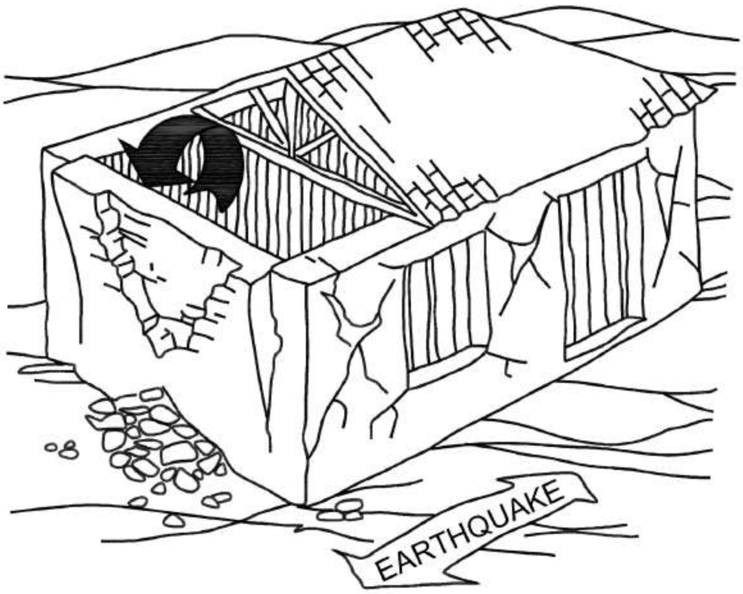
Soft story or other configuration induced failure

Typical failure mode of a house during earhquake[45]

1.7 Why are the masonry houses more vulnerable to earthquake forces?

Most of the houses are constructed with locally available materials such as brick, stone, block, adobe etc. which are termed as masonry construction. In contrast to other materials; masonry has wider acceptability which depends upon locally available materials, climatic and functional requirements, technical knowledge and traditional practices. But such materials make a house stiff and heavy with low seismic resistance due to limited tensile and shear strength. These constructions do not posses any ductility (ability to deform in inelastic state) and ability for energy dissipations. Therefore, these constructions are much vulnerable to earthquake forces since the earthquake resistant capacity of a house can be simply expressed as - the product of resistance and ductility and in case of masonry construction both the quantities have a lower value as compared to other contemporary materials; the product of both the terms further decrease the earthquake resistance capacity.

Masonry walls being made from discrete units lack structural integrity which is another cause of weakness against lateral loads. Moreover, wall and floor masses create large inertia forces and with the absence of bond among the walls, walls and floor/roof result in the loss of box action and each wall of house behave independently in out-of-plane action. As we know the capacity of a wall against out-of-plane action is much less to its in-plane capacity therefore the walls of the houses may fall outward and consequently the roof may collapse.



Seismic behaviour of a building
with poor connection between walls

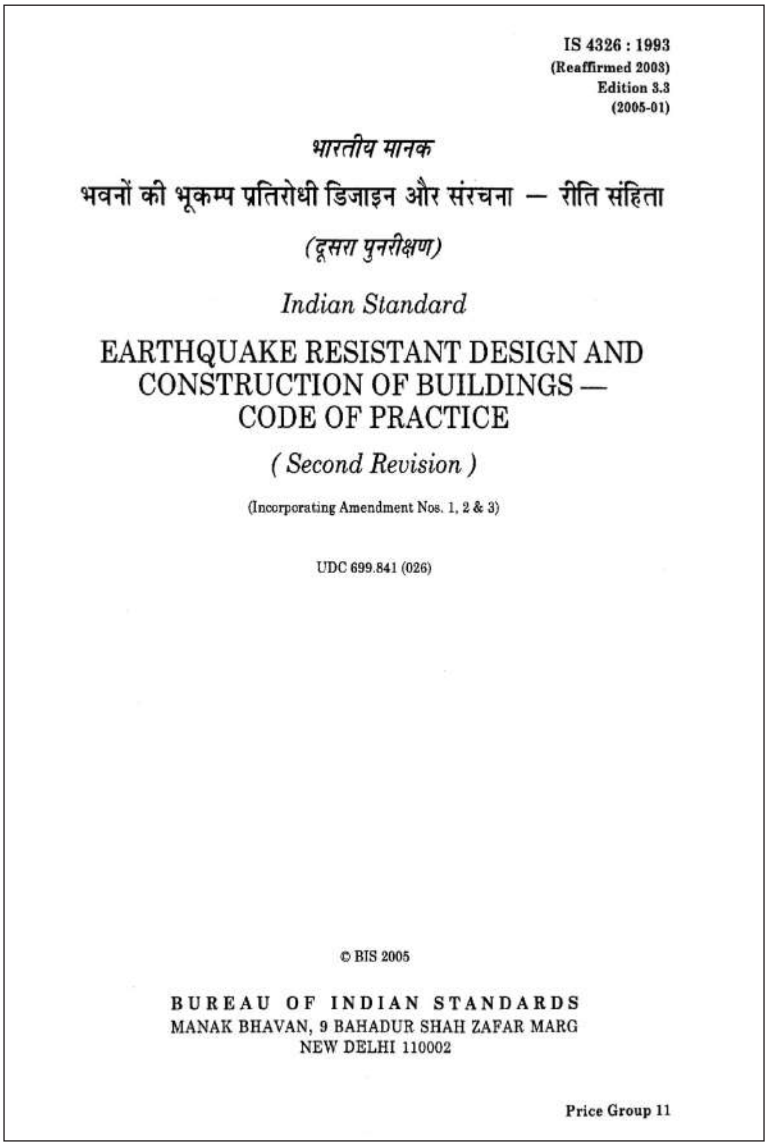
***Failure of a masonry house*[24]**

Chapter 2

Possible Ways to Increase the Earthquake Resistant Capacity of a House

2.1 Can a house resist earthquake?

Why not! It is possible to make an earthquake resistant house. Some damages may occur but it will not collapse. Just as a child is to be saved from various fatal disease by following vaccination program regularly right from the beginning; similarly to save a house from earthquake, a number of provisions and precautions are to be taken since its inception – planning, construction and post construction. These provisions are called earthquake resistant measures even available in BIS code of practices. First of all it is necessary to know how these provisions work to save the damage in a house during earthquake.



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Indian Standard

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(Second Revision)

(Incorporating Amendment Nos. 1, 2 & 3)

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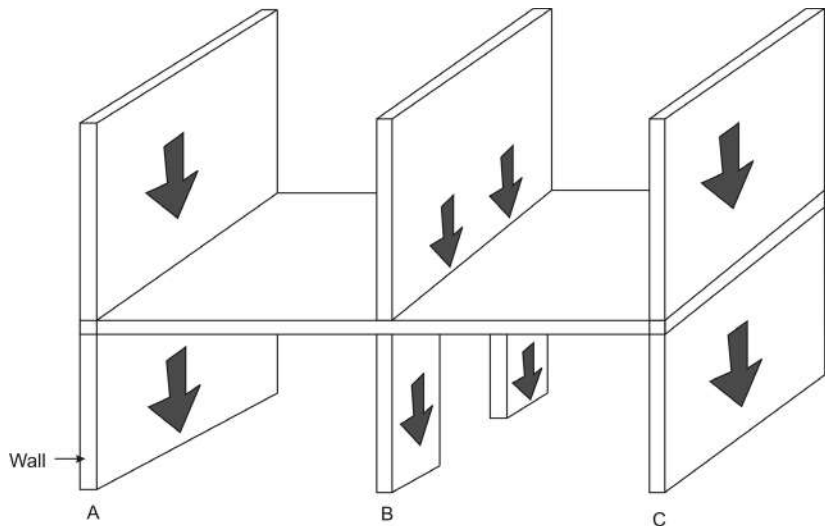
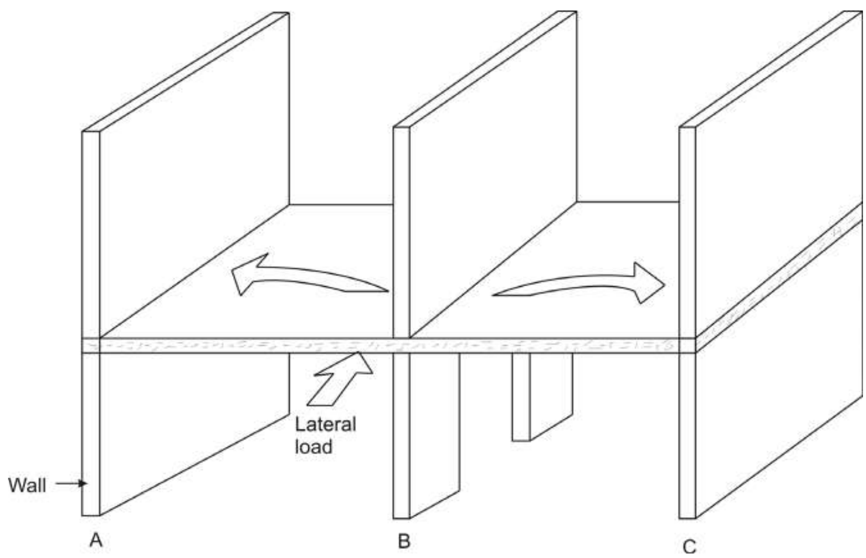
BIS codes on earthquake resistance measures in masonry houses

2.2 How does a house resist earthquake loads acting on it?

A house is constructed on certain design principles irrespective of its vertical height which vary from single to four stories. The seismically ideal condition of a house is that vertical load is transferred to the foundation through the load bearing walls while the lateral load is carried by the composite box structure. The seismic efficiency of load bearing structures largely depends upon the lateral load resistance of the walls of a house. The walls spanning vertically between horizontal diaphragms (floors) face lateral loads caused by earthquake. The floor acts as rigid diaphragm transferring the loads to in-plane walls which in turn transmit the forces to the ground. Such structures are quite stout having short span concrete slab system supported by numerous walls running in both the principal directions of a house. The efficacy of the system relies on the capability of the individual masonry elements sustaining it to share loads along with their mutual connection to transmit forces.



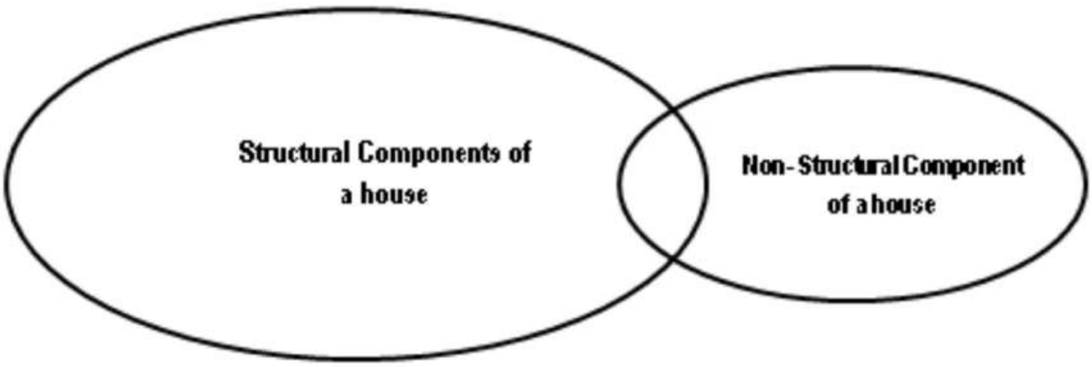
Vertical load resisting mechanism in a house



Lateral load resisting mechanism in a house[7]

2.3 What are the main components of a house to resist the earthquake?

A house is a composite unit or assembly of certain components namely structural components and non-structural components. The combined performance of individual system enables it to resist vertical and lateral loads. The structural components transfer the horizontal and vertical loads comprising of roof, walls, connection of roof and walls, connection of wall to wall, floors and foundation and its ground stratum. Non-structural components provide utility services to the occupants such as balconies, projections, parapet walls, staircases, overhead tanks, chimneys etc. But the failure of a non-structural component also brings about severe damage hence proper connection between non-structural components and structures also matters.

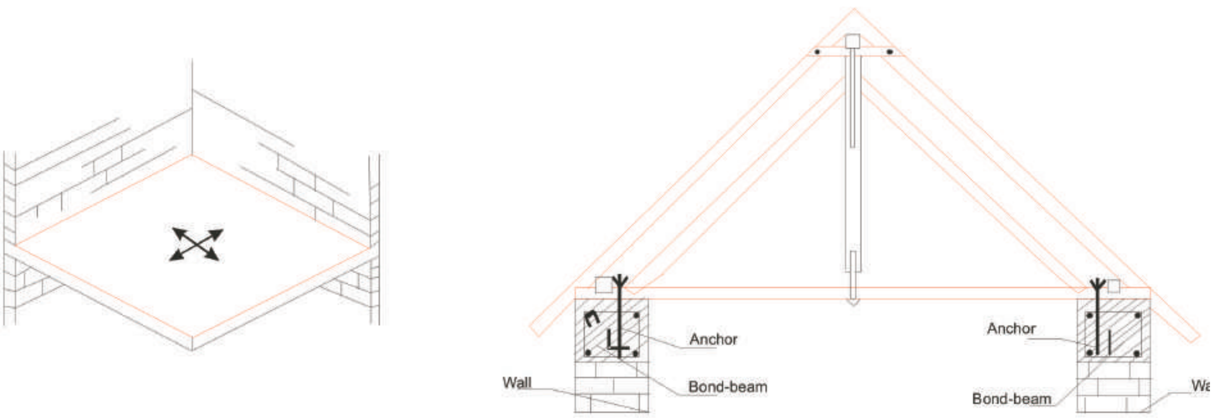


Structural and Non-structural components of a house should be connected to each other

2.4 How floor and roof systems behave during earthquake?

Floor systems in a house carry out dual functions i.e. supporting the vertical loads and distributing lateral loads to shear walls (in-plane walls). Walls perpendicular to the assumed ground motion must span vertically between the floor diaphragms so that the inertial effect of half of the wall height may be transferred to floor diaphragm already bearing its dead loads. Generally house roofs are of two types (i) RCC or RB type also called rigid slab; i.e. a 75 mm thick concrete slab provides a rigid diaphragm action, effectively suitable for irregular floor plan or odd span (ii) wooden slab, tiled roof etc. i.e. flexible roof system which is more vulnerable to earthquake forces due to incapability of transferring inertial effect to the shear walls and has torsional forces. Failure of rigid diaphragm is very rare during earthquake while the flexible diaphragm fail during earthquake.

Therefore, the first way to protect a house from earthquake is that it may have a rigid type of slab such as RCC or RB with a minimum of 75mm thickness. In case of flexible type of roof like gable roof it should be properly braced in both the directions so that a rigid action can be achieved and should be adequately anchored into the R.C. bond beam. The following additional requirements should also be considered (i) each floor should be situated in a single plan avoiding sharp dis-levelment (ii) rigid behavior of horizontal diaphragm should not be altered by the presence of discontinuities such as staircase, large opening zones should be reinforced by bond beams (iii) Two way slabs are preferred in place of one way slab (iv) to reduce seismic loads, light roof structural system and roof cover (tiles) are preferred.

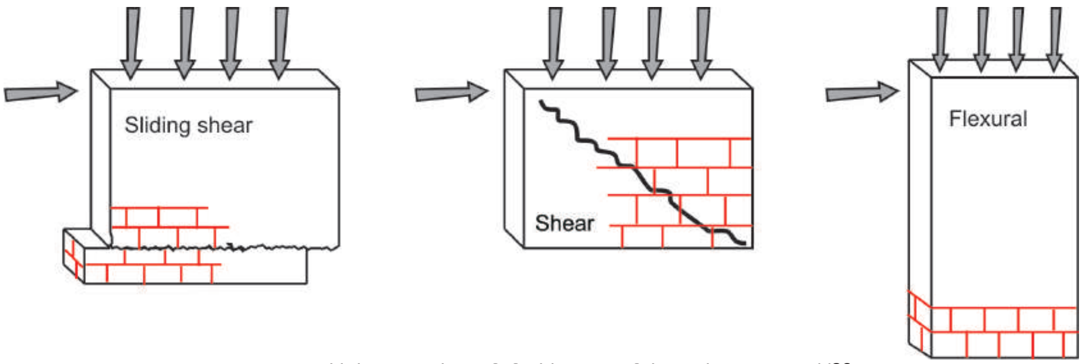


Two way rigid slab or adequately braced in both direction gable roof is preferable for transferring of earthquake forces[23]

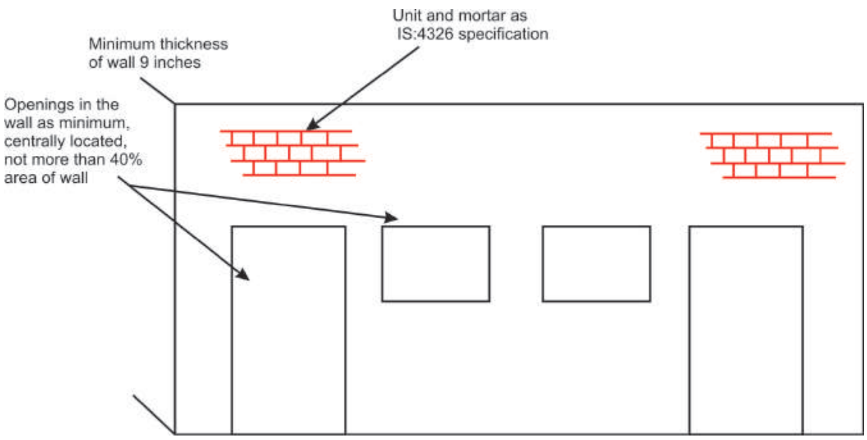
2.5 How do wall systems behave during earthquake?

The in-plane walls, to which the inertial forces are, transferred, may have three modes of failures under varying loading conditions namely vertical and horizontal mainly due to earthquake (a) sliding or tensile failure at the heel of wall (b) diagonal shear failure (c) flexure failure at the toe of wall. Toe failure occurs due to crushing under bi-axial compressive stress; failure at the heel occurs when vertical loads are lower to lateral loads which develop tensile stress normal to the bed joint with consequent horizontal cracks. Failure in the center of walls is described as diagonal shear failure, and infact takes place in the bed and header joints under a combination of principal tensile and compressive stress with subsequent sliding along the joint. The shear strength of the wall increases with increase in vertical loads. The stress distribution within a shear wall is complex and depends upon the geometry of wall, the nature of the load application, the presence of opening etc. Unless the major openings or discontinuities are present none of the above failure modes can normally cause complete collapse of the wall. If the in-plane shear capacity of the wall is adequate resisting the combined effects of inertial forces, all the inertial forces will be transferred to the foundation.

Therefore, the second way to protect a house from the earthquake is that its walls should have sufficient in-plane strength which largely depends upon the unit and mortar strength, thickness of wall and even the vertical load acting on the wall. Unit and mortar as recommended by the code but the minimum thickness of wall must be 9" (225mm) and for vertical load of all the walls of the upper stories in one vertical plane.



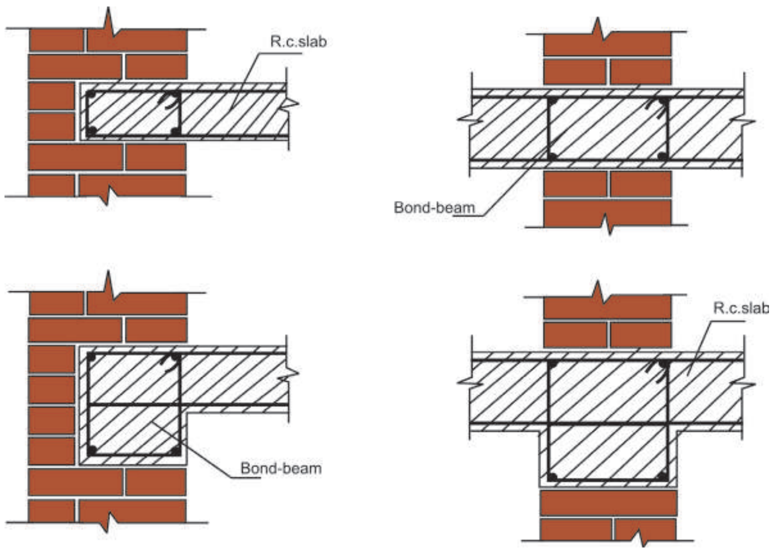
***Possible mode of failure of in-plane wall*[23]**



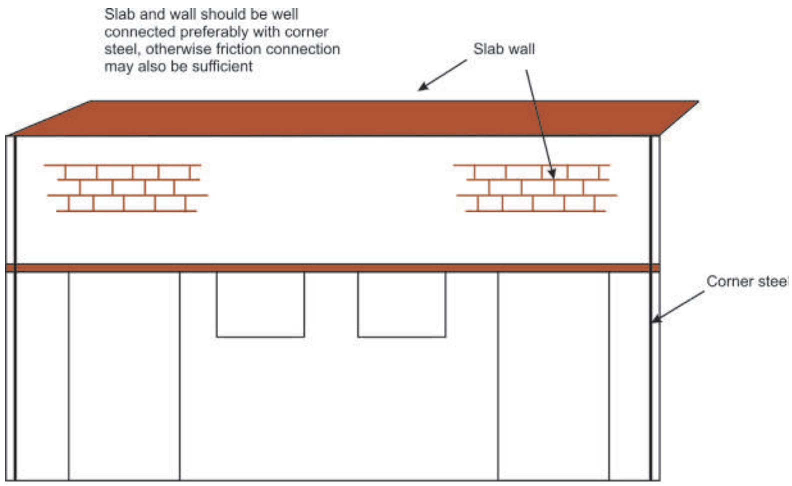
Possible way to prevent in-plane wall failure

2.6 How important is the roof/floor wall connection for resisting earthquake?

In a house, the connection between floors and walls is vital link since is has two main functions and unfortunately both are opposite to each other. At one time it provides lateral support to the wall to resist the earthquake loads under the in-plane action as well as out-of-plane. On the other hand the connection must allow progressive movements which occur in between the two elements due to the effect of temperature, concrete shrinkage and masonry growth or shrinkage. If the first objective is achieved by providing a monolithic connection in between the floor and wall then it is difficult to achieve the second objective i.e. a free movement at the interface of floor and wall that might create cracking at the interface. Therefore, the ideal condition for the connection of floor and wall in a house from seismic point of view is monolithic but practically one can rely on friction but in that case masonry house may face some potential serviceability problems.



Bond Beams in masonry buildings [23]

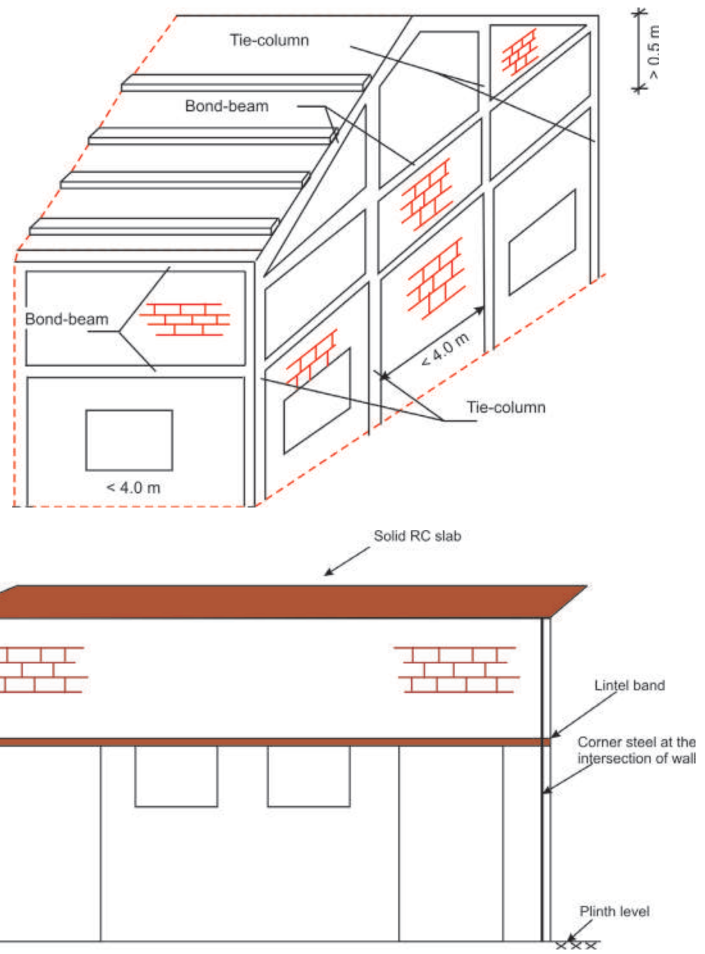


Floor and walls should be connected to each other either by bond beam or vertical tie member

2.7 How wall to wall connection is important for resisting earthquake forces?

Wall to wall connection is another important link to improve the earthquake safety of a house. This connection provides an integrated action among all the walls of a house that will be very effective for resisting the earthquake forces. In common house construction, the wall to wall connection is provided in the form of toothing i.e. two orthogonal walls are connected by creating a half brick gap in alternate courses. Past experience reveals that this form of connection does not behave satisfactorily during the earthquake. Since this connection is neither capable to integrate all the walls nor does it unite the corners where this connection is made.

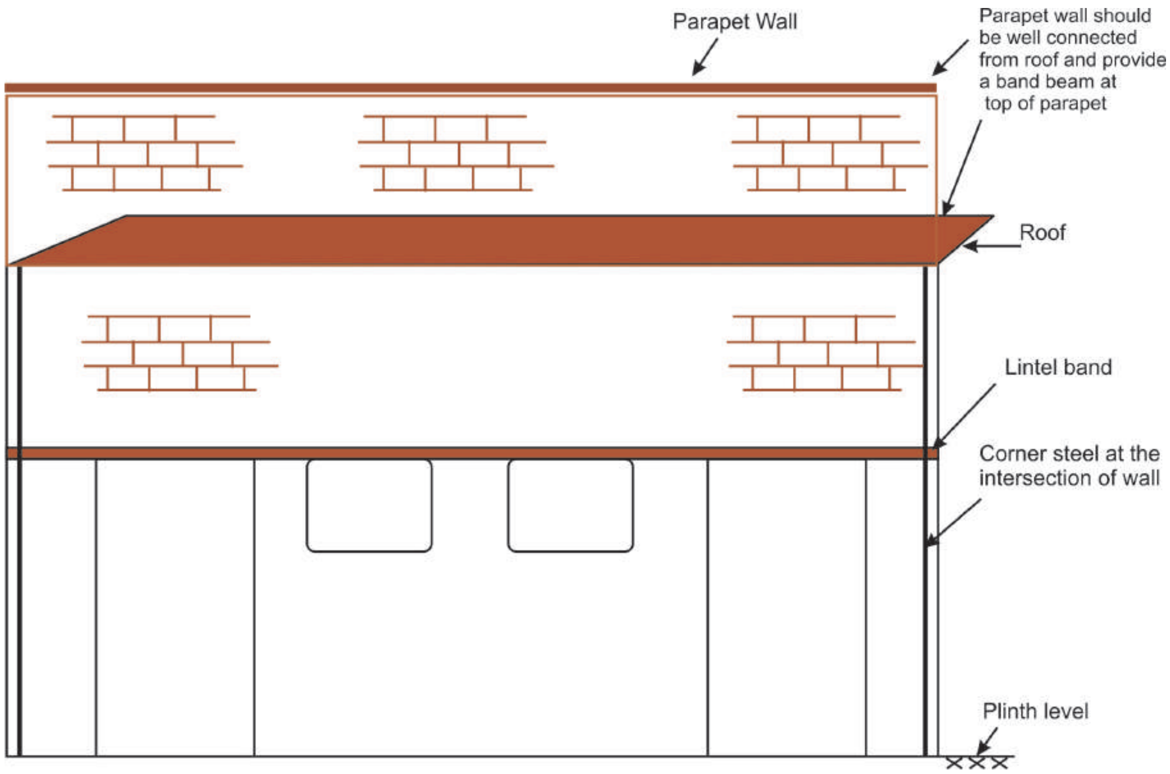
Therefore, the third way to protect a house from the earthquake is that all the walls act in an integrated manner and the corners (L, T, + etc) of the walls must be properly reinforced so that their chances to split out are as minimum as possible. This can be achieved by providing the horizontal bands along the height of house running in all the walls either externally or internally as well as by providing vertical steels at all junctions of wall. The details of these features are given in the BIS codes. Sometimes dowels in place of horizontal bands may also be another cost effective alternative solution to integrate the walls and strength of the junctions of walls but of course less effective.



All exterior and interior wall must be integrated by proving horizontal band and vertical tie members (tie corner, vertical steel)

2.8 How each non-structural component is affected by an earthquake?

A parapet wall and other free standing elements are the most common type of non-structural systems which have little resistance to lateral loads due to low flexural strength. They rely on gravity for stability. Some form of flushing of the base exuberate the situation. Being at or near the top of structure where the effects are magnified by the dynamic response, this element is easily damaged by bending or overturning. Failure of parapet may be fatal when it overturns outside, crushing people or damages parked or moving vehicles. It is desirable to avoid such elements or to support them by local reinforcement. Alternatively un-reinforced parapets can be designed to span horizontally between returns or piers which themselves can designate to provide overall stability. Some times a bond beam occurring at roof or floor level may be used at the top of the parapet wall.



Parapet wall and other nonstructural element should be properly connected to main structural system

Chapter 3

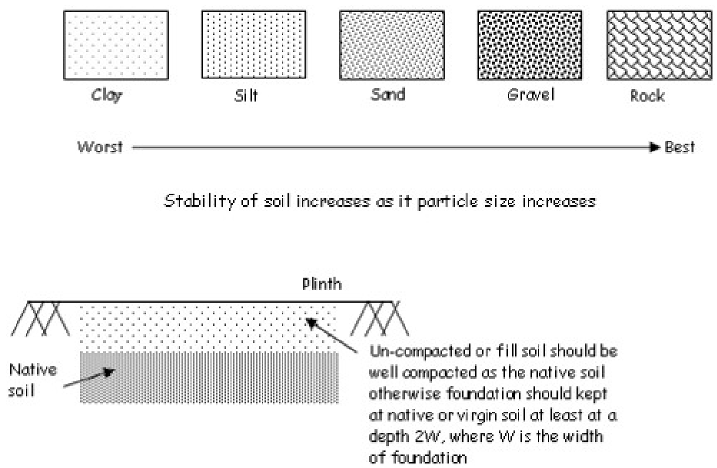
Planning Consideration of a House to improve the Seismic Behavior

3.1 How far construction site affects house stability during earthquake?

The stability of a house in either case much depends on the type of soil at the construction site. The term soil describes the top layers of material constituting the earths crust in varying proportions including three basic materials air, water and solid particle. The major classification of soil is by particle size of solid elements namely either fine grained i.e. silts and clay or coarse grained i.e. gravel and sand. A simple test to identify soil is to disperse a quantity of soil into a container of water and note the rate of sedimentation. Sand and gravel settle quickly within about 30 seconds. Silt settles within 15 to 60 minutes while clay remains suspended for several hours. As a rule the stability of soil depends on the increase in particle size. Sand and gravel soils are the best for construction. Silt is coarse than clay but finer than sand hence is stable, cohesive in nature, tend to compress, deform and creep under constant load particularly vulnerable to volumetric changes induced by moisture variations. The clay is a culprit to foundation stability. If the soil is clayey a number of precautions are required.

Rock is also a soil strata generally assumed as good site for construction. But this hypothesis is not always true because bearing qualities of rock are dependent on factors such as presence of bedding plane, faults, joints, weathering, cementation of constituents etc. resulting in foundation failure particularly in earthquakes. Therefore, the stability of rock may also be ensured before the construction especially in case of earthquake prone regions. The foundation should not be laid on stepped rock unless in a hilly terrain where a special precaution will be required.

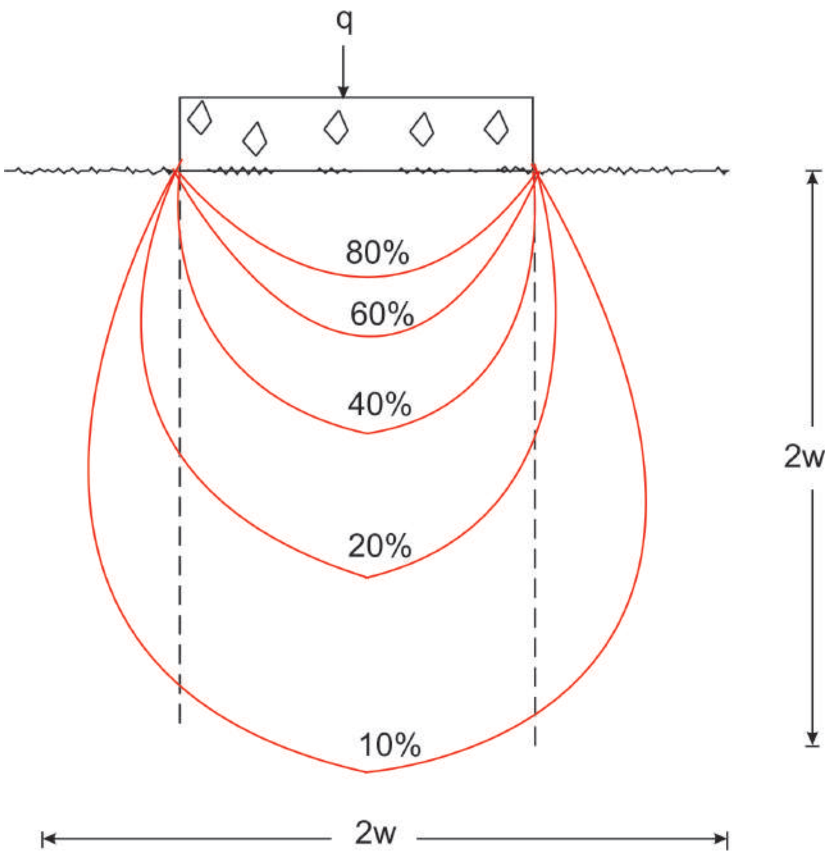
If the construction site is not leveled and required filling, compaction of soil is a must upto the similar densities and nature as the native and virgin soil. In case of silt and clay, soil compaction is more severe. There are some sites where soil condition is very poor which is not able to resist loads such as peat, organic fills, loose filled soils, shifting soil at base of hills. Sometimes the sites are prone to landslide and liquefaction and must be avoided. Saturated cohesionless soils are prime to liquefaction besides a category of silty soils may also liquefy during earthquakes.



Suitability of soil at construction site

3.2 How much depth of soil can affect the stability of a house during earthquake?

After the selection of site, the depth of a good soil is the next issue for seismic efficiency of a house. As a thumb rule for a common house construction which is having a strip foundation with a width W whose load may be Q, requires the depth double the width for diminishing the load effect. At a depth of W/2 the load will remain only 0.8 Q and will keep on reducing; at a depth equal to the width it will come to just 0.5 Q and at a depth 2W it will be about 0.1 Q or even less. This suggests that the depth of good soil should be about 2 W.



Depth and width required for a good soil at the construction site[5]

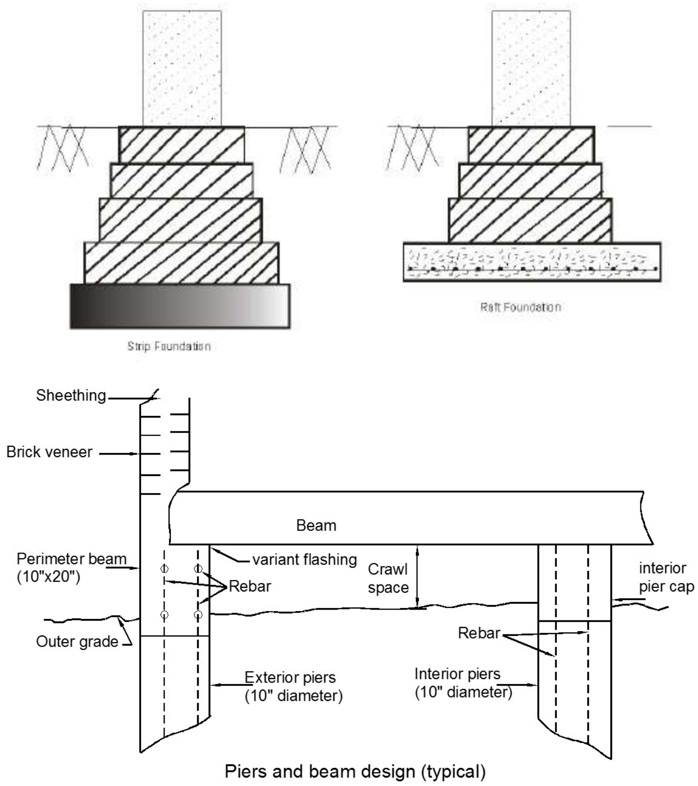
3.3 Which type of foundation is suitable for a house?

Foundations provide the supporting link between the house and the ground, transferring the vertical and horizontal loads to the latter. Hence foundation must be on firm stratum with sufficient resistance for sliding or overturning of a house during earthquake. To perform satisfactorily the foundation must withstand ground movement limiting distortion of the house to tolerable levels.

The usual concept of foundation for low rise buildings is to build walls either directly on to the bottom or on to a thin layer of rubble compacted in the bottom of trench. Later strapping and corbelling of the brick work at the base of wall is to introduce to form footing. For better construction the bottom of the trench is needed to be lined up with a layer of weak un-reinforced concrete. The design width of footing should not only satisfy allowable stress in the foundation material but should also be varied so that neither excessive settlement nor differential settlement may occur.

When the bearing capacity of the soil is insufficient for strip footing the area of footing can increase. A mat/raft covering the whole building reducing soil stresses to an acceptable level. The raft/mat must be strong and stiff which distributes foundation loads and helps to reduce the distortion in the brickwork. Such foundation is expansive for masonry building.

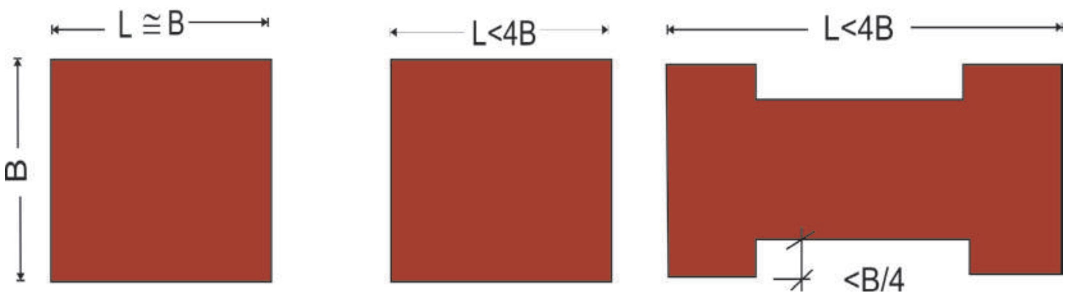
Another option to support the strip footing is by piles or by piers. In these cases, the wall and footing can be connected to become a composite beam spanning between the piles or piers with no support assumed from the soil beneath it.



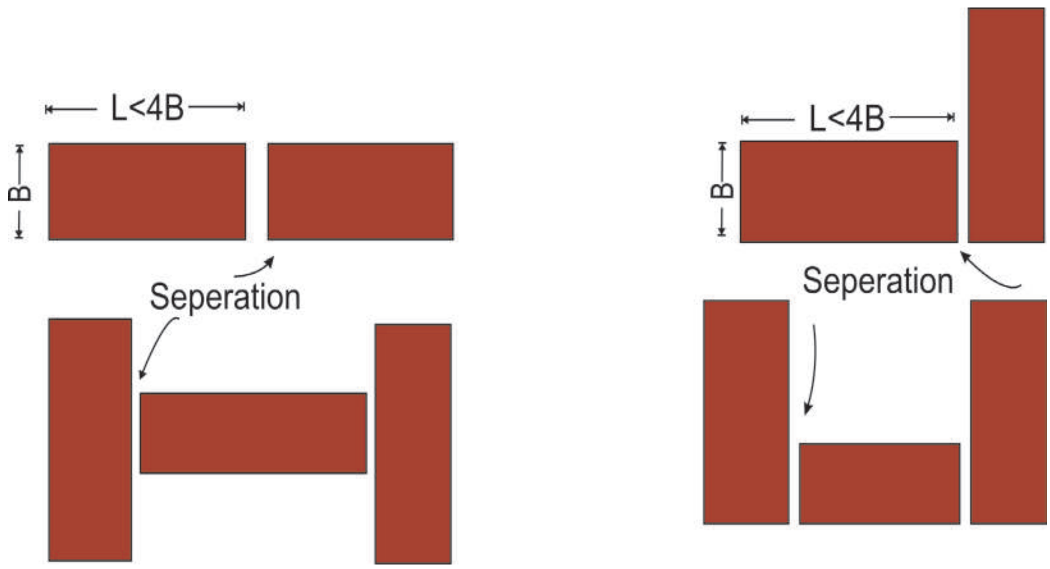
Different type of foundation used in houses depending upon the condition of site[5]

3.4 What is the ideal plan of a house for resisting earthquake?

The building plan of individual house depends upon parameters such as shape, size, and orientation of the site, relation to neighboring building, internal communication route, natural lighting requirements and the function for which the building is designed. The square and rectangular building plan performs better than those with many projections. The total dimensions of the projection, re-entrant corners of recesses in one direction should not exceed 25% of the overall dimensions of the building in the corresponding directions. In case of long rectangular buildings it is desirable to limit the length of a single part up to four times its width. The plan must be symmetrical along both the axis which will prevent possible torsional vibrations causing unexpected behavior. For unsymmetrical plan such as L, T, + shaped, a separate joint can be provided to separate the building into different parts to maintain symmetry and regularity. To prevent hammering effects between the adjacent parts, the separation width should not be less than 30mm with an additional 10mm for each storey with building height exceeding 9m. The seismic joint can be filled or covered with flexible and water proof materials.



Examples of regular configuration of masonry buildings in plan

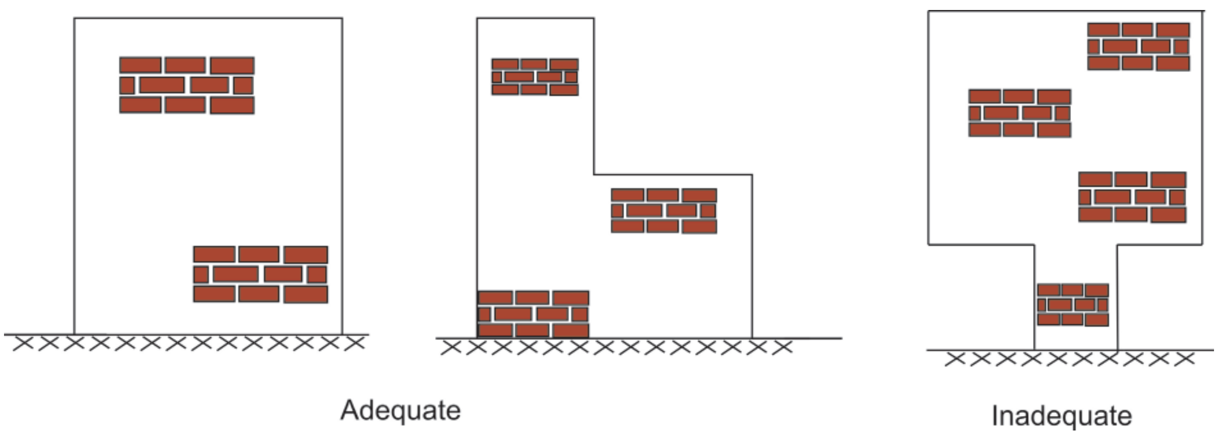


Irregular masonry buildings should be separated in regular sections[23]

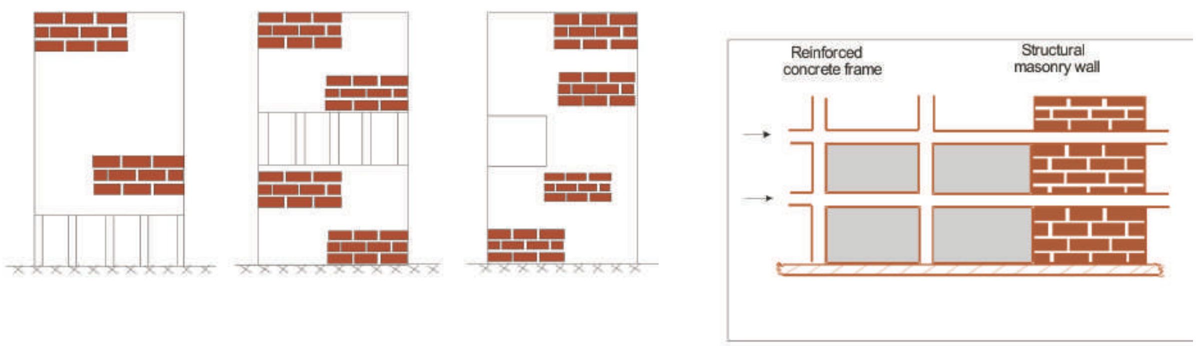
3.5 What may be the ideal elevation for a house for resisting earthquake?

The seismic capacity of a house decreases with increasing height i.e. increasing in number of stories. Partially reinforced masonry or masonry with all earthquake resistant measures may allow upto 3 to 4 stories adequately serviced by the staircase. Masonry houses more than 4 storeys require special emphasis on design and construction and the use of reinforced masonry for the structural system. The main requirements for finalizing the vertical planning are symmetry, regular elevation, and uniform distribution of resisting elements, stiffness, and masses along the height. Avoiding concentration of masses at upper stories, sudden changes in stiffness caused by changing the dimension in plan, distribution and type of structural elements may reduce possible damage. When significant changes in elevation occur a vertical isolation joint can be inserted to separate buildings to prevent them from hitting each other under earthquake loads.

Mix structural system such as combination of masonry structural walls at one storey and RC frame structural system to adjacent storey are also to be avoided or should be reinforced with vertical and horizontal reinforcement. RC columns should be part of framed system connected to walls to ensure distribution of loads among the walls and columns.



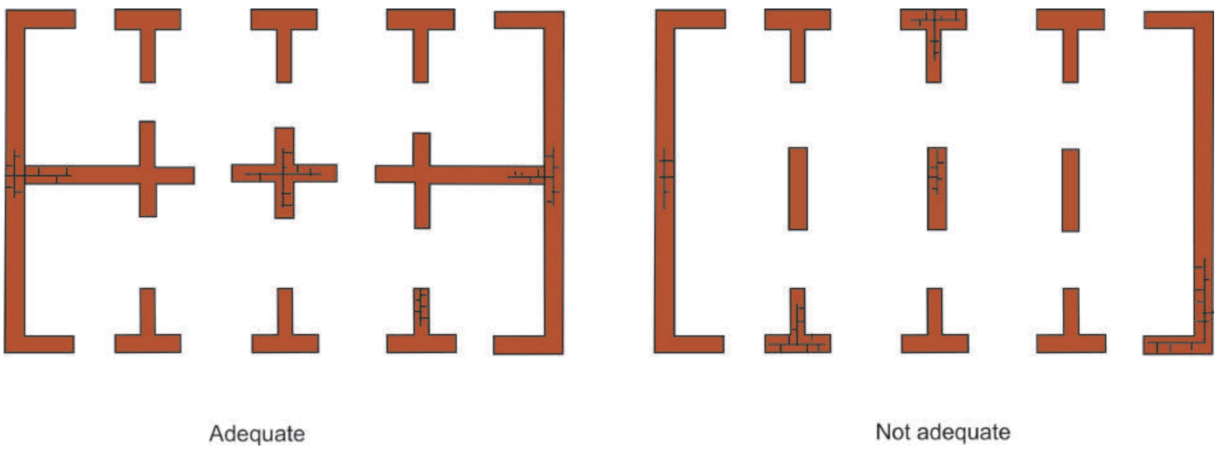
Principles for building configuration in elevation

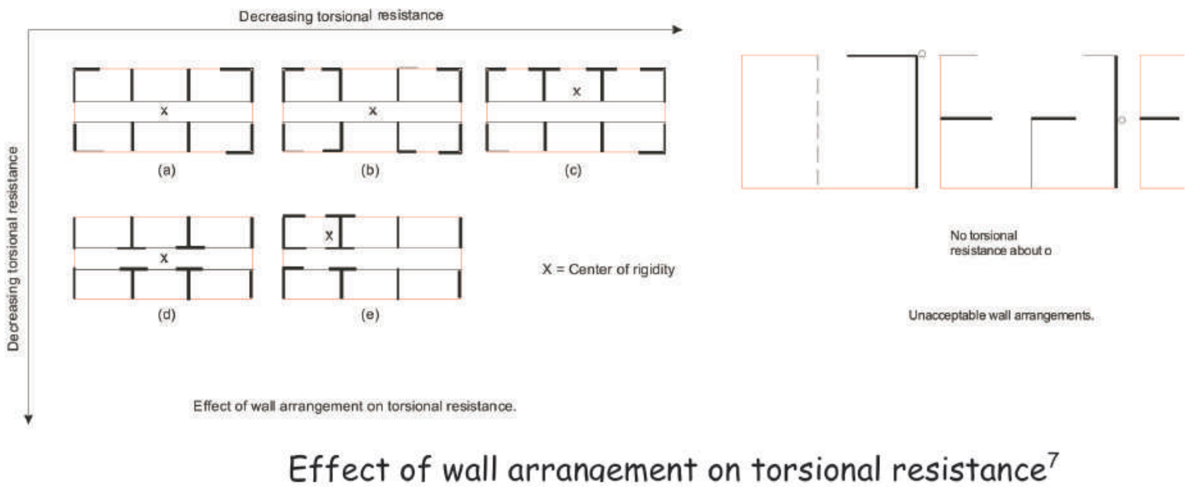


Mixed structural system should be avoided[23]

3.6 What type of wall layout is desirable from earthquake point of view?

Earthquake motion is three dimensional phenomenons. It is unpredictable as to which will be the main direction of motion. Hence the resisting elements should be designed to resist the seismic excitation in both principal directions otherwise the consequences may be serious. The building structure is approximately symmetrical along each principal axis regarding lateral stiffness and mass distribution, but it is not subjected to significant torsion. A sufficient number of structural walls with approximately the same cross-sectional area and stiffness should be provided in each direction. The building with regular structural layout, with walls properly connected together at floor levels has often performed well. Therefore, the structural systems must be simple and regular consisting of load bearing walls and cross walls that do not change their position and shape along the height of building, evenly distributed in both directions with gravity and seismic loads transmitted in a clear and undisturbed way from element to element and the induced seismic energy dissipating uniformly over the entire structure. In the absence of uniform distribution concentrations of stress occur resulting a heavy damage and collapse of the structure.

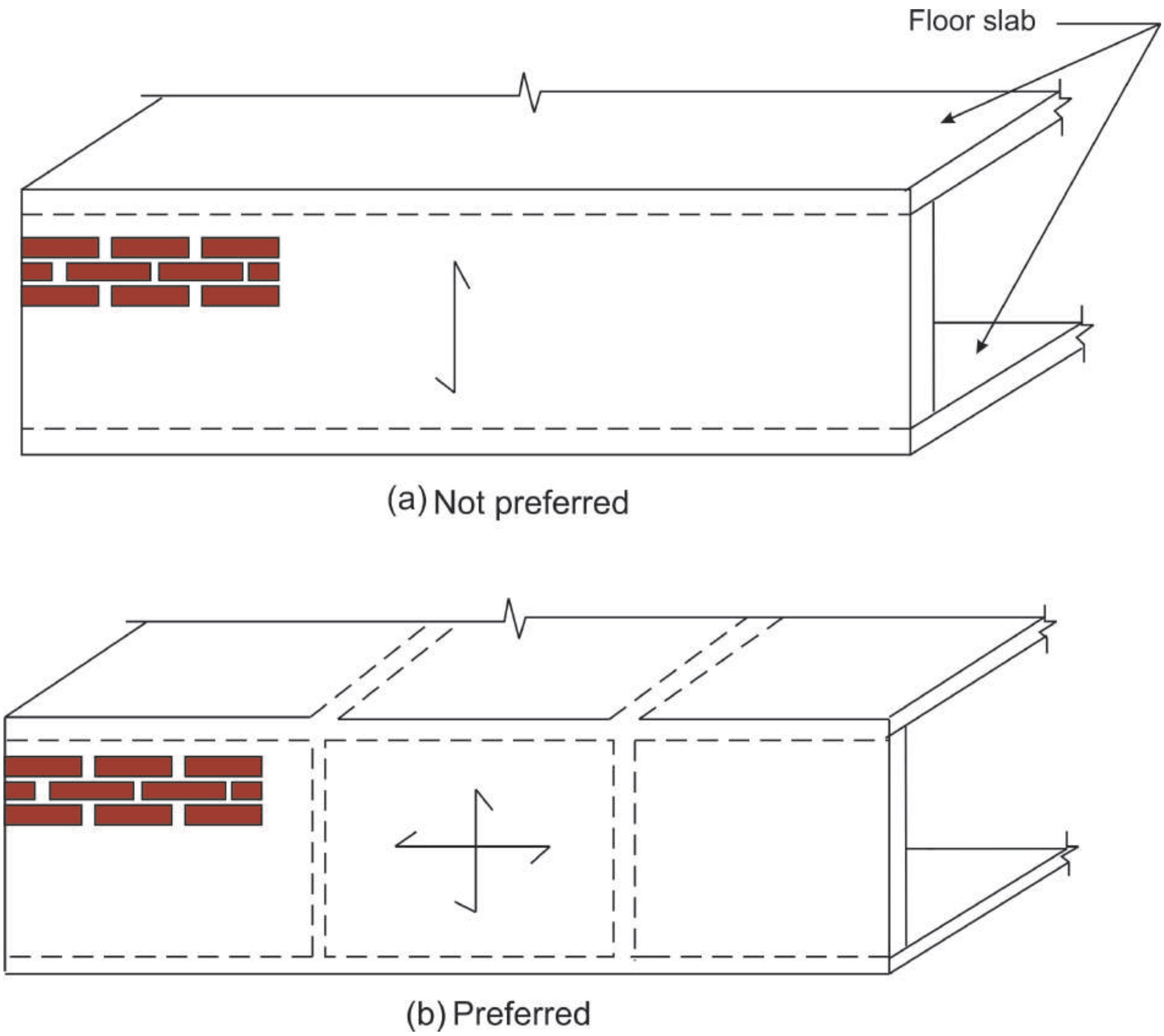




Effect of wall arrangement on torsional resistance[7]

3.7 Are some considerations needed in the planning of exterior walls?

Apart from height limit the most significant impact of the code are the requirements associated with tying and detailing which is already discussed in previous section. The designer must ensure that the walls should span vertically between the floors and horizontally between cross walls behaving in two way flexure. Non-load bearing walls and free standing elements must be supported.



Lateral load resistance of walls (a) not preferred (b) preferred [23]

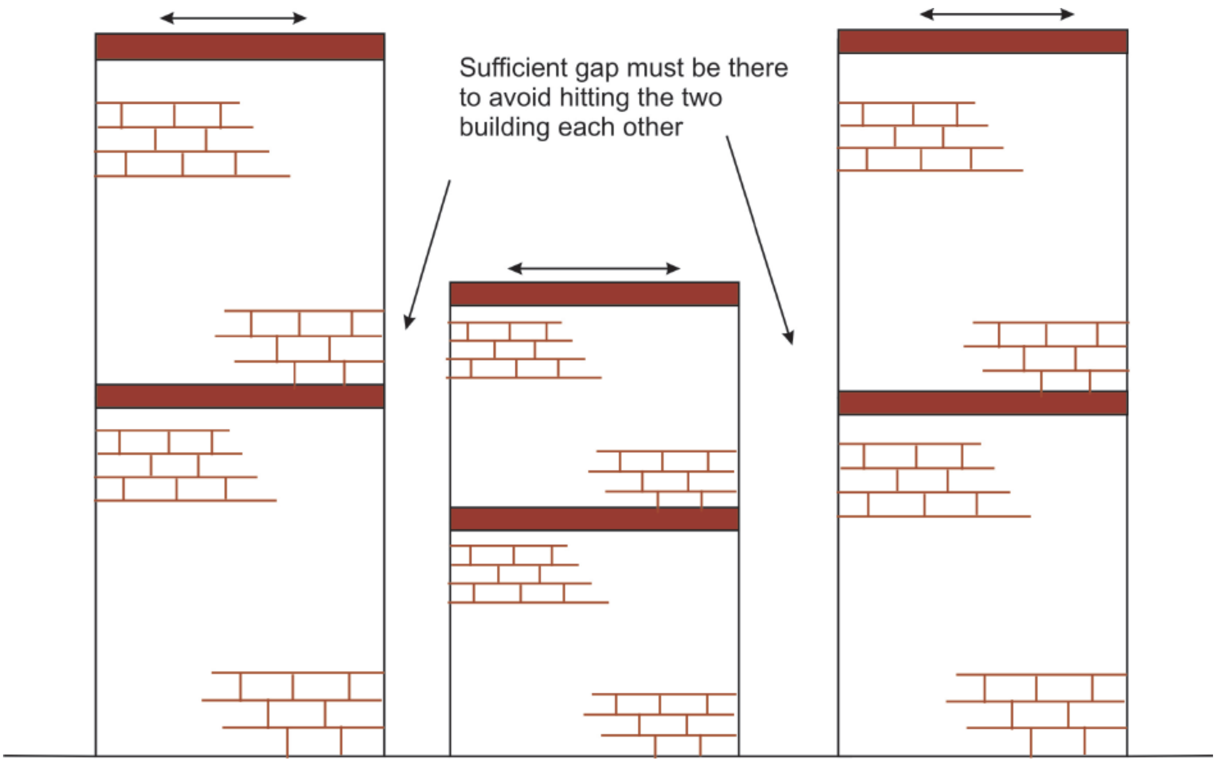
3.8 Are some considerations needed in the planning of openings in exterior walls?

In-plane walls subjected to lateral loading, concentration of the stresses first occur at the openings provoking large strain resulting in cracking masonry elements and degradation in stiffness. The openings in the walls for windows and doors affect lateral stiffness and reduce seismic resistance. The effect depends upon the size and location of the opening. In case of 3 or 4 storied houses, shear forces increase from top to bottom; hence small opening in lower stories is only recommended. Large openings may be provided with increasing number of stories. Staggering openings opposed to align openings produce better coupling for vertical section.

Placement of windows and its effect on wall flexibility[7]

3.9 Is a gap required between nearby houses to avoid hitting against each other during earthquake?

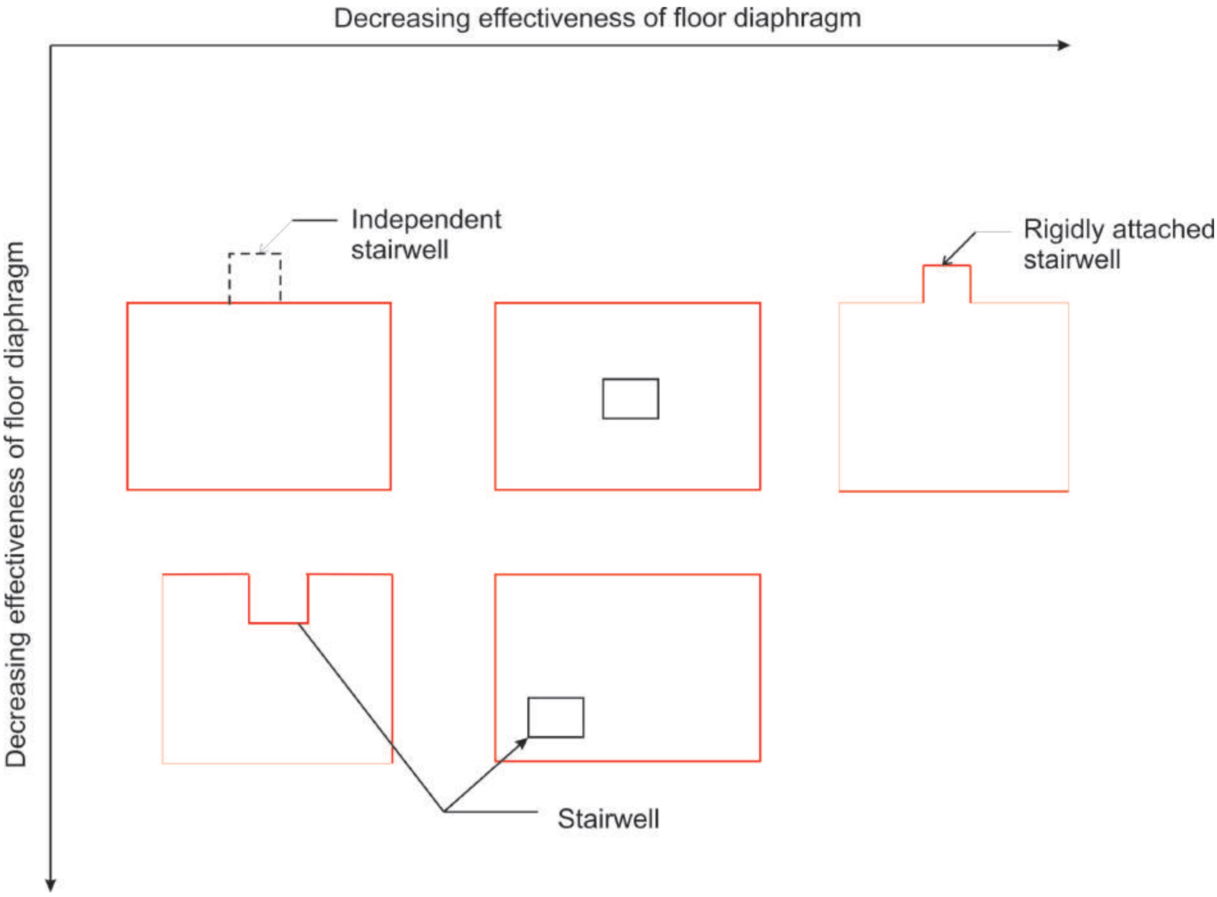
A very common phenomenon is that a building or a house is closed i.e. wall to wall contact on the three sides of the other house. In such a situation, an earthquake can cause a house which is not sufficiently separated to pound against each other severely by heavy hammering. Adequate clearance must be provided so that each can move laterally without interference or contact. Rocking or settling of the foundation and actual deflection of the structure itself, separation at the top of the shorter structure should at least be equal to the sum of the total deflection. For a typical masonry, wall construction less than 25m in height an arbitrary rule of thumb for such a gap is 50mm for the first 5m of height above the ground plus 10mm for each additional 3.3m in height is required.



Typical way to avoid pounding/hammering failure

3.10 What additional measures are to be considered in roof and floor systems of a house for resisting earthquake?

Generally roof and floors do not take part in resisting earthquake loads. Therefore, they are designed on the basis of vertical loads but must be strong enough to transfer a force which means adequate in-plane rigidity. The cast in-situ two way slab is the ideal solution for roofs and floors. Rigid floor action is necessary to distribute forces to individual shear wall. They should be firmly connected by steel ties or RC rings or RC bond beams. The corresponding locations of openings due to staircase and irregularities in the floor, affect the effectiveness of the floor as a diaphragm hence the staircase must be separated from the plan or should be symmetrically placed in the plan.



Influence of placement of stair on effectiveness of floor diaphragms[7]

Chapter 4

Effect of Workmanship and Construction Practices on the Seismic Performance of a Masonry House

4.1 Do the workmanship and construction practice affect the seismic performance of a house

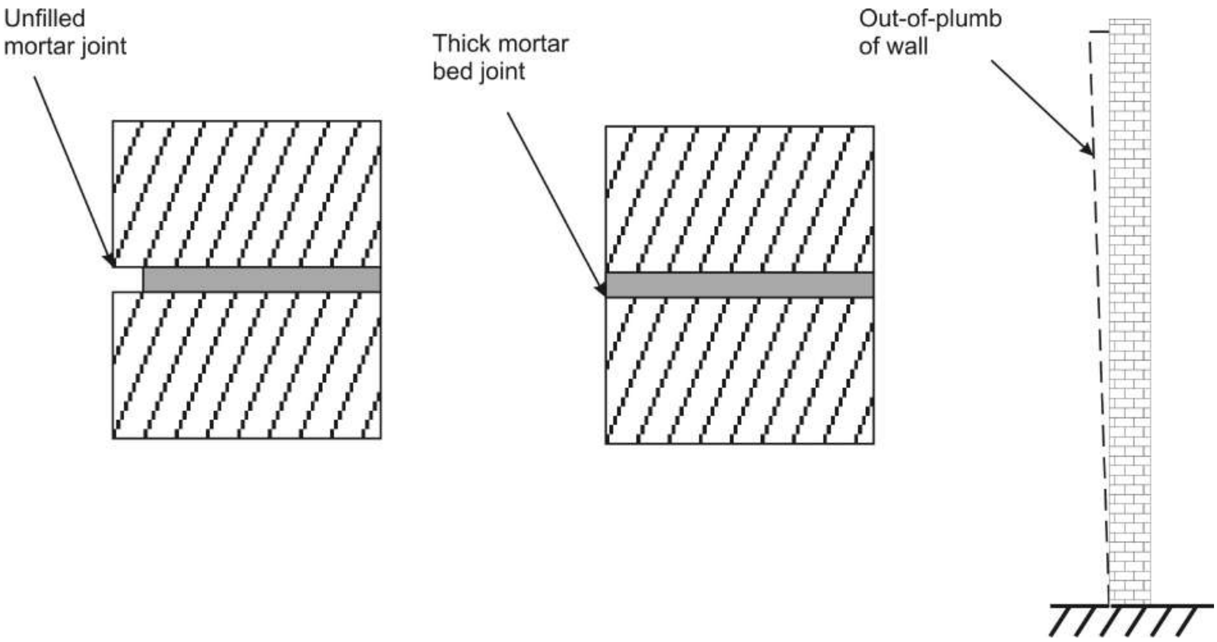
The quality of construction depends on the quality of material and workmanship. Workmanship relates to the mason's work such as general appearance, alignment, complete filling of joints, rejection of damaged units, the use of properly mix mortar and general attention. A mason poor work reduces the seismic performance of a house and its resistance against environmental and natural forces. Defects in construction practices with their effect on the performance are as follows;

Unfilled mortar joint: Here joints are filled only on one side and the other side remains unfilled reducing its tensile, shear and compressive strength (responsible for seismic resistant) up to 1/3 and also producing serviceability problem due to less resistance to rain penetration.

Thick bed joint or thick layer of mortar: The compressive strength of masonry reduces up to 20 to 30 % with increasingly thick mortar layer of 10mm to 20mm.

Out-of-plumb: The wall must be horizontally and vertically in plumb. Misalignment and initial deviation increases the eccentricity of loading of wall resulting in reduced capacity. In a 3m storey height a 200mm thick wall with a 13mm displacement can bring a reduction in axial load up to 15%.

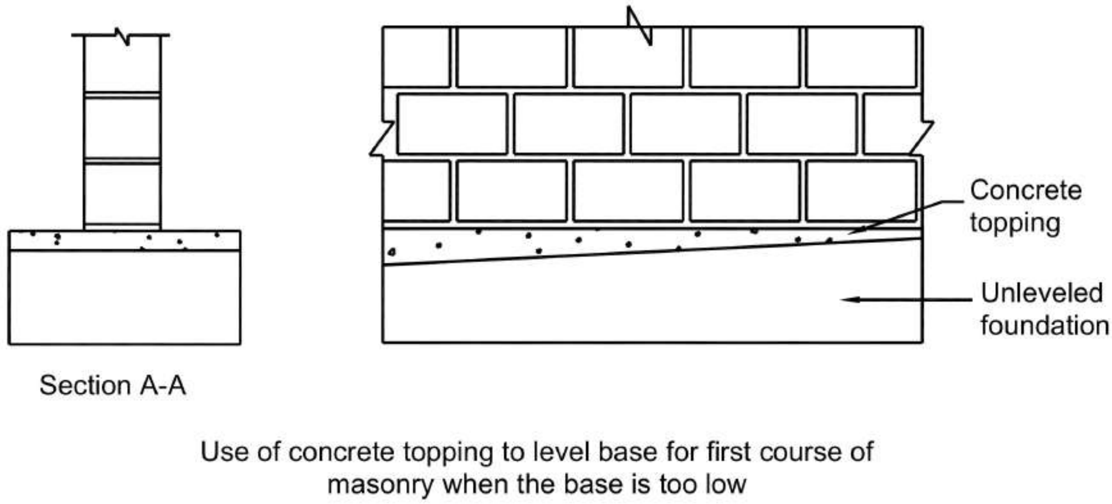
All these may unitedly reduce the strength of a wall up to 38 to 45%.



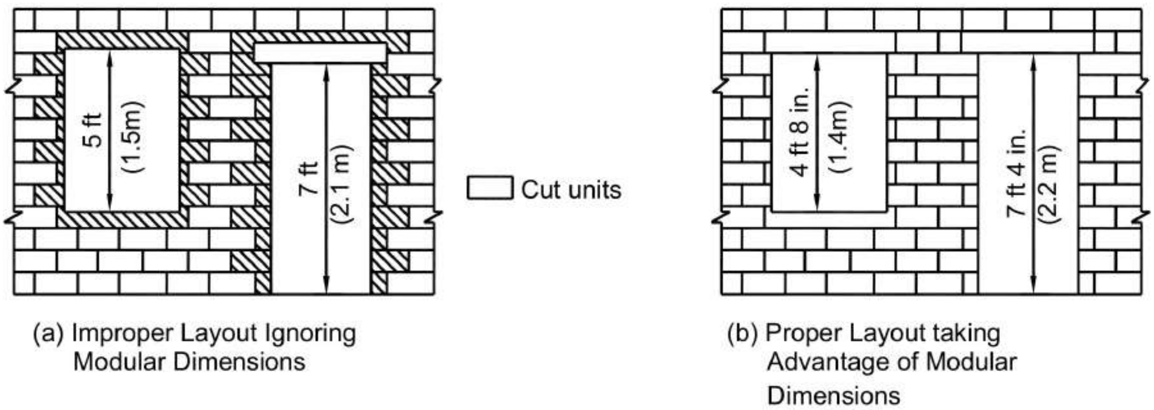
Common causes of poor workmanship [7]

4.2 Does the seismic performance of a house depend on level and alignment of masonry?

The foundations and floor slabs for laying masonry must be checked for level and alignment. If the discrepancies exist, they must be removed at the level of base course either by concrete topping or by changing the thickness of mortar joint. Prior to using mortar the base course should be dry to avoid the irregular or broken bond patterns. Openings and wall length should be in dimension to reduce the amount of cutting or it should be accurately cut to maintain uniform mortar joint thickness. Adhoc dimension schemes are not acceptable as they result in poor arrangement.



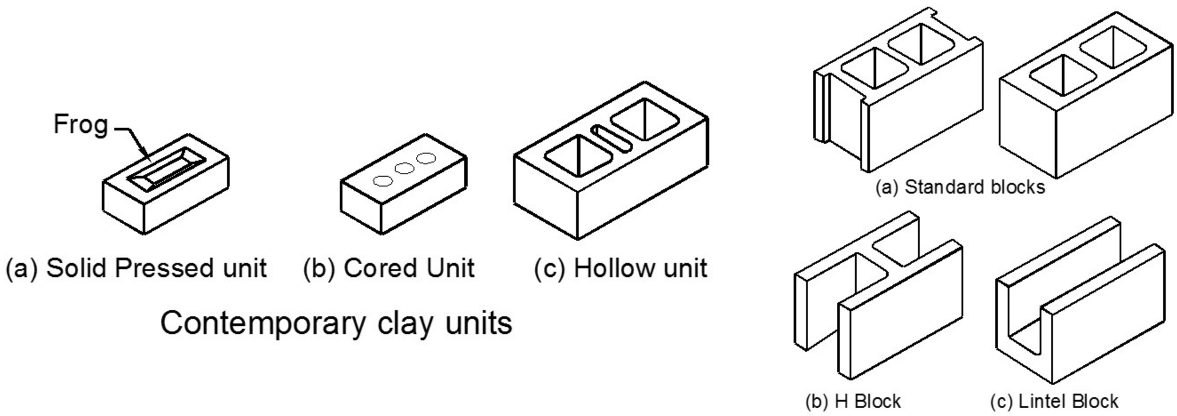
Proper way to remove any misalignment at foundation level[7]



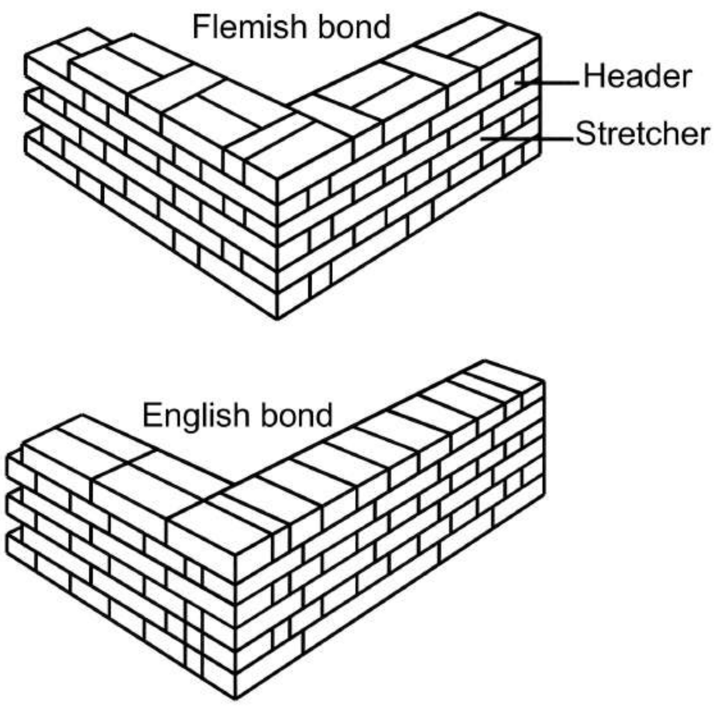
Planning consideration in masonry wall openings[7]

4.3 Which type of unit is preferable in earthquake resistant construction?

Masonry units must be sound with sufficient strength. It must also be durable not exhibiting efflorescence. Solid masonry units are usually laid with full mortar bed; hollow units are laid in face shell bedding with mortar applied to the face shell of each unit. The compressive strength of masonry is directly related to units' strength while the tensile strength governed by bond between mortar and unit. Masonry bond strength depends on the correct match of the mortar and unit properties. Practically the value of bond strength is particularly susceptible to workmanship effect and mortar. Laying of units in running bond with offset between $\frac{1}{4}$ and $\frac{1}{2}$ unit length is recommended.



Different types of unit in masonry construction

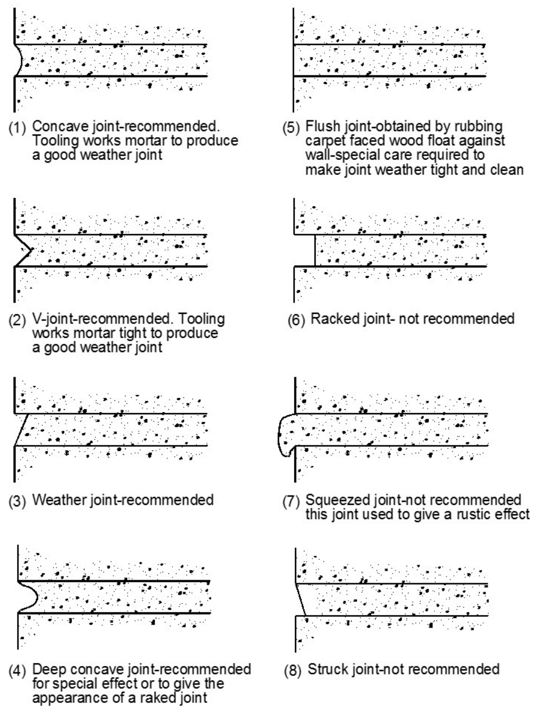


Types of bonds in masonry construction[7]

4.4 Which type of mortar is preferable in earthquake resistant construction?

Mortar is the most important ingredient for the strength and durability of the masonry construction. It fills the irregularities, provides the resistance to the penetration of light and water and bonds the unit together. But in earthquake resistant construction it provides tensile and shear strength resisting the lateral loads. The most effective mortar consists of cement, lime and well graded sand. The proportion of individual ingredients is to be based on compressive strength or field experience. But higher cement content leads to bond breaks and opening of cracks in head joints. Lime increases workability, plasticity and water retention of the mix for maximum bond strength. Hydrated lime is generally recommended. If sand is too little the mortar becomes sticky or fatty while too much sand results in lean mortar difficult to spread. Admixtures available in the market should only be used when specified and recommended. Generally the masons have a good knowledge in mix proportion rather than laboratory technicians.

An improperly matched mortar may either get stiffened prematurely or remain fluid for too long. Laying of subsequent courses is not allowed because they tend to move out of horizontal and vertical alignment. Pre wetting the surface reduces suction preventing premature stiffening. Mortar materials should be measured in gauge box or hoppers in appropriate volume ratios. It should have good consistency. Best results may be obtained when mortar is at lowest workable consistency producing highest bond. It should be re-tempered to replace the water loss by evaporation. The rate of vertical construction may be limited so that the weight of the upper courses may not deform the plastic mortar below. Excess mortar protruding from the joints after laying should be struck of cleanly to remove mortar strain later.



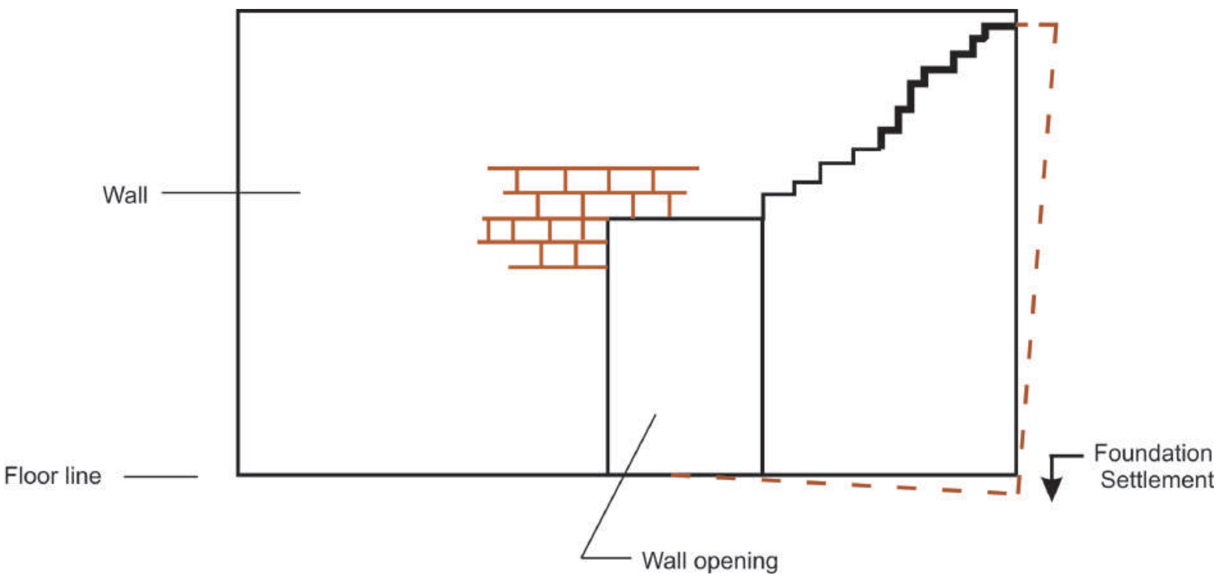
Type of joint in masonry[7]

Chapter 5

Effects of other Parameters excluding Earthquake on the performance of a Masonry House

5.1 What are the main causes of cracking in the house?

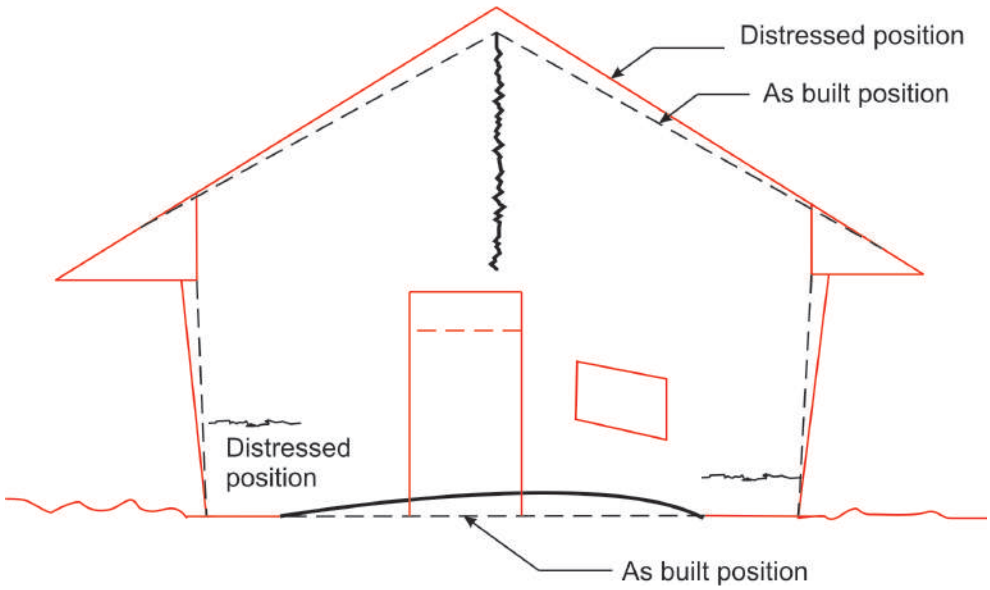
The main cause of cracking in a house excluding earthquake is the foundation movement or uneven settlement or moisture movement in plastic soils such as clay and silt. In settlement some portion of the foundation drops below the original due to a number of reasons such as soil consolidation, shear failure, variable type soil, compaction of infill, erosion of supporting soil, dehydration and loss of soil moisture. It may mostly be observed at the perimeter of the floor slab, and corner. Cracks resulting from uneven settlement of foundation, may be in any form, are mostly diagonal, vertical and tapered.



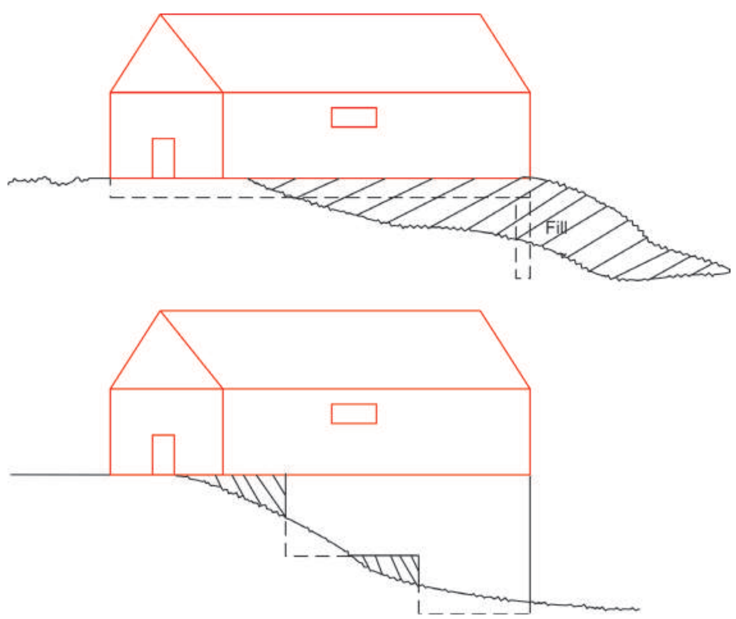
Cracking due to differential settlement[12]

5.2 What may be additional causes of cracking in a house?

Clay and silt types of plastic soil increases and decreases in volume with moisture contents. Slight movement of houses is inevitable in seasonal change. Two types of movements may also be possible beside such settlement namely, (i) Upheaval - relates to the raising of internal areas of the foundation above as-built position. In high-clay soil this results from excessive moisture under the foundation caused by improper drainage, underground water, from domestic sources such as leaks in supply or waste systems. Normally the whole house is affected due to up-heaving leading to sticking doors and windows, or cracking of brick walls, ceiling cracks etc. (ii) Sliding - occurs when a structure is erected on slope and the movement is not limited to vertical position but possesses a lateral or horizontal component.



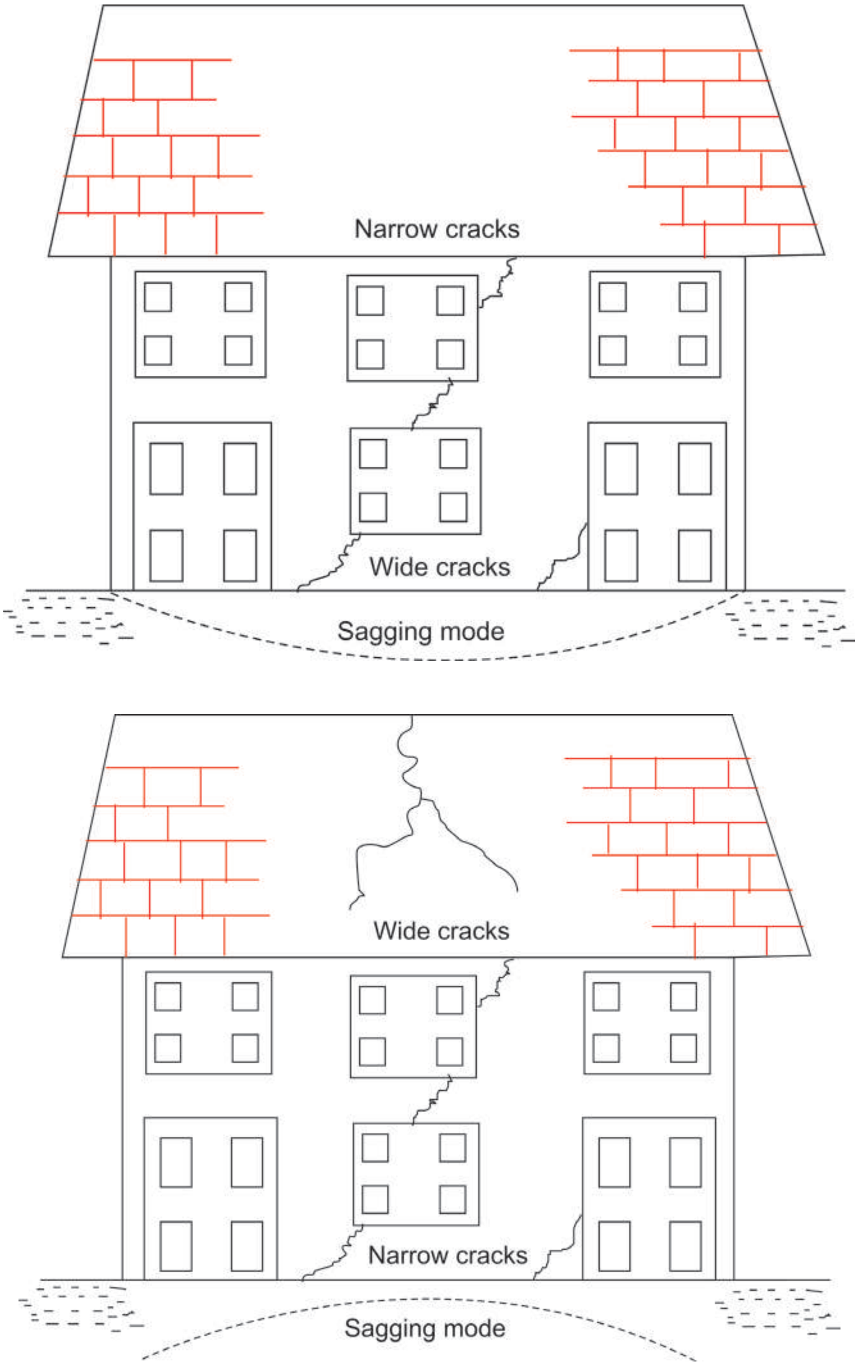
Cracking due to Upheaval



Possibility of sliding failure[5]

5.3 How the cracks developed due to foundation movement differ from earthquakes?

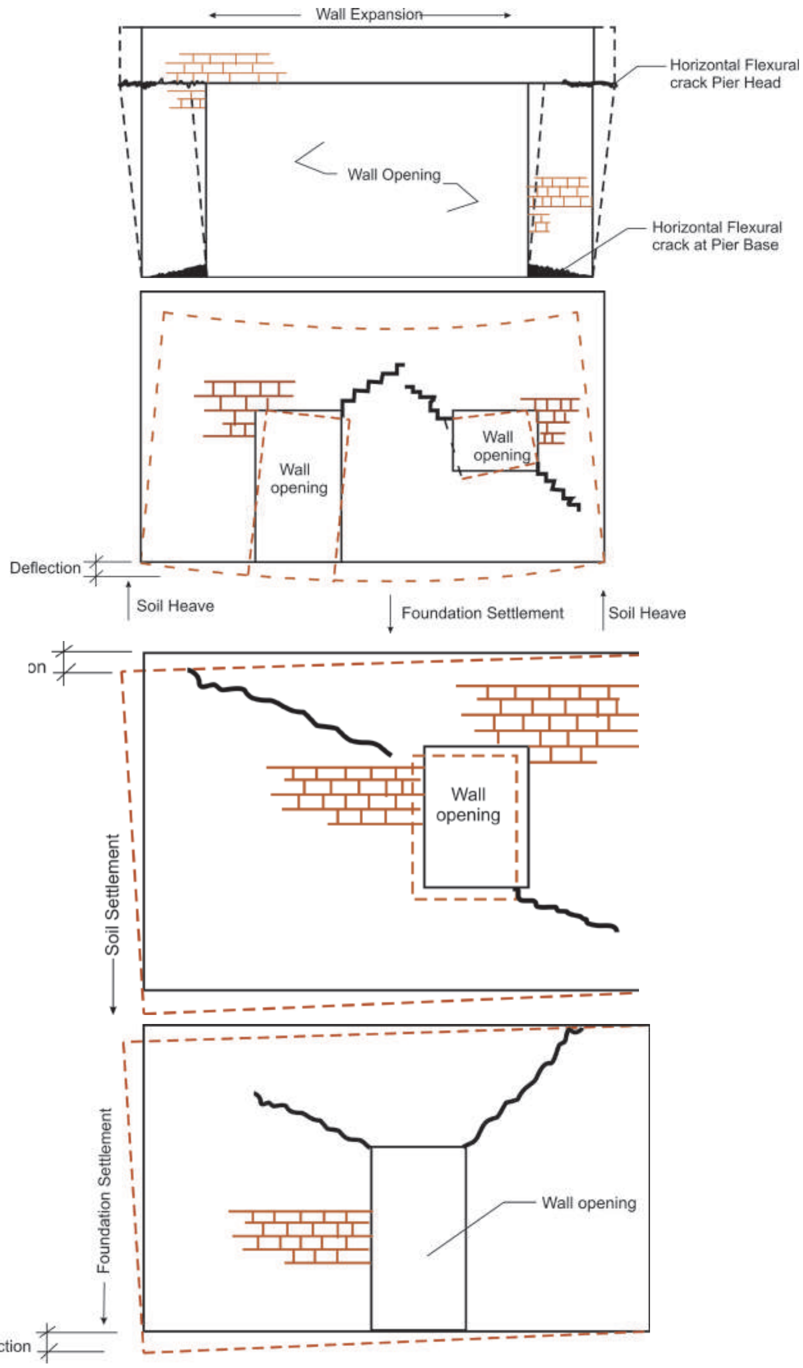
Foundation movement can cause distortion and cracking in the buildings. There are general indications to identify damage such as few isolated cracks at weak points in structure, Cracks taper from top to bottom, Cracks continuous through DPC, Cracks exceed 3 mm width, Cracks occur both externally and internally at the same location, Doors and window sticks.



Identification of cracking in a house due to foundation movements[12]

5.4 What are the common locations of cracking in a house due to foundation movements?

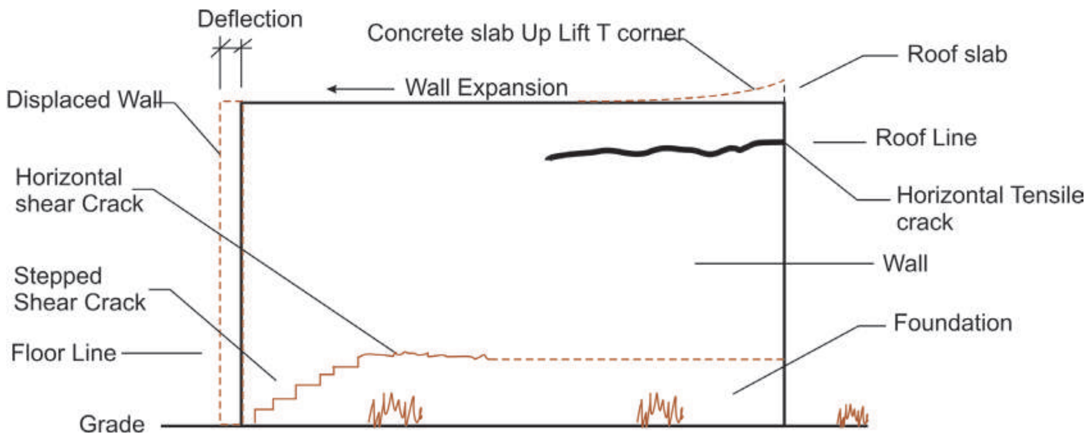
Foundation movement results in continuous cracks at weaker sections, such as window openings and doors and at points of change in foundation depth. Other indications cause doors and window to stick, effect on partitions, ceiling, floors and roof, resulting from rocking. The best way of confirming is to measure how much external walls are out of plumb and brick courses are out-of-level. The capacity of 9 inch thick brick walls to carry loads is reduced to more than 30% by a stepped or slanted crack up to 1 in. (25mm) wide.

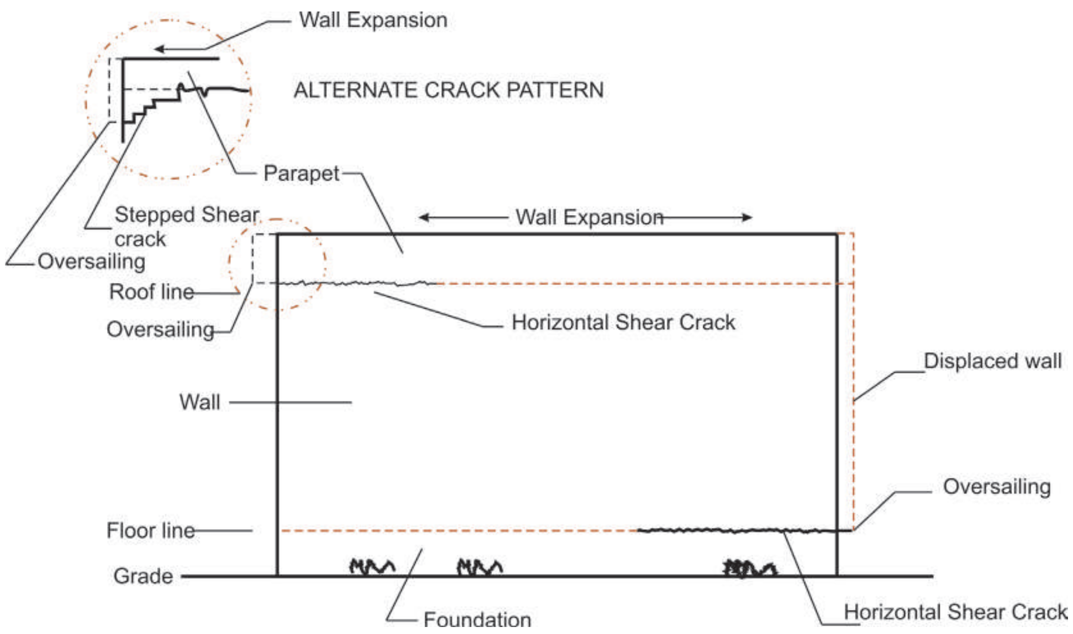


Possible location and type of cracking in a house due to foundation movements[12]

5.5 Can houses crack due to environmental reasons?

Common causes of cracking due to environmental reasons are frost attack, thermal expansion, and contraction, drying shrinkage, over-stressing of walls or floors and chemical attack. Expansion of masonry may be caused by heat, moisture, or freezing which may result in sailing of upper portions of a wall over lower ones, diagonal shear cracks, bowing of walls, and flexural cracking at corners, restrained longitudinal expansion and bowing of parapets. Cracks result from strain including stress in excess of strength in compression, tension, or shear due to impositions of loads or by restrain volume changes in the masonry materials.





***Cracks due to environmental reasons in a house*[12]**

5.6 How much cracking is vulnerable to a house?

Cracks do not hamper the vertical load capacity of the wall significantly but the differential foundation movement increases the tensile and shear stress in the walls and very large movement can make the wall unstable. Therefore, in case of earthquake, these cracks are responsible for damage in houses and be repaired as suggested below[10];

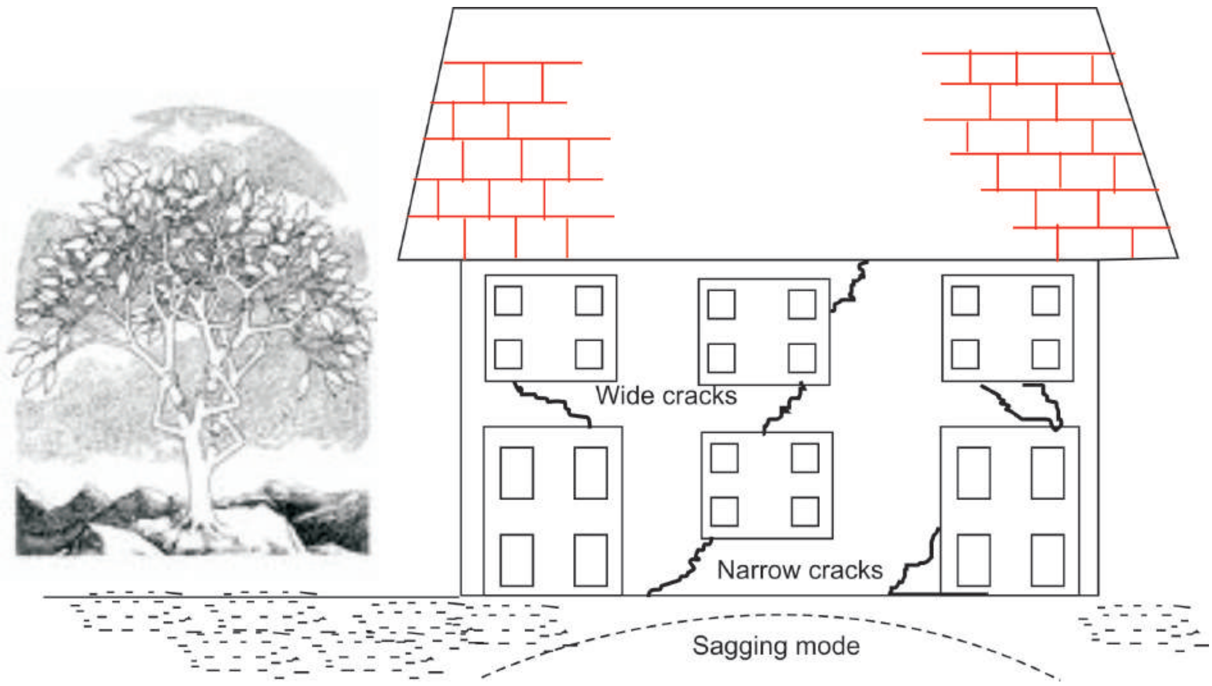
Category of cracks	Description	Repairing method/action needed
0	Hairline cracks, normally indistinguishable, typical crack width 0.1mm	No action required
1	Fine cracks, typical crack width 1.0 mm.	Filling of cracks from epoxy grout injection
2	Cracks courser than category 1, typical crack width 5.0 mm	Cracks will normally close during subsequent winter and can be filled with epoxy or cement mortar
3	Typical crack width larger than category 2, typical crack width 5mm to 15mm. Door and window sticks	Cracks can be patched up by mason, re-pointing of external brickwork and possibly a small amount of brickwork to be replaced,
4	Typical crack width are 15mm to 25mm	Underpinning or partial rebuilding are likely to be required, expert advice from outside can be solicited.
5	Typical crack width larger than 25mm	Rebuilding is likely to be required, expert advice from outside can be solicited.

5.7 Do trees or vegetation near a house may be the cause of cracking?

Plants such as weeping willow can cause foundation problem even if located at some distance from a house. Plants with large, shallow root system can grow under the foundation in diameter; produce an upheaval in the foundation. It is suggested that the trees be planted at a safer distance of about 1.5 times the ultimate anticipated heights.

Plants also remove water from foundation soil producing foundation settlement. In case of excessive differential movement, foundation failure may likely be possible.

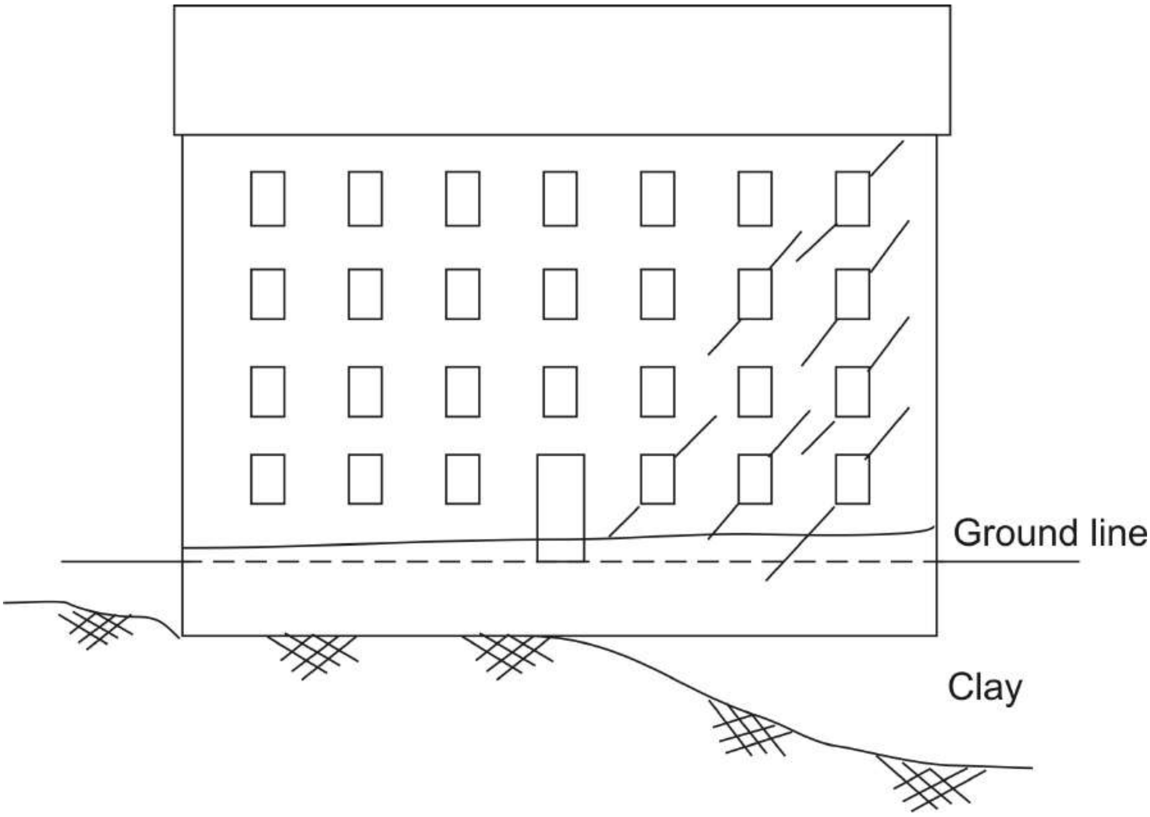
Where the tree is older than the house, calculation of heave potential in the soil adjacent to the foundations be carried out before removal of tree.



Cracks due to trees or vegetation near a house[10]

5.8 Is it possible to eliminate the cracking due to foundation movement?

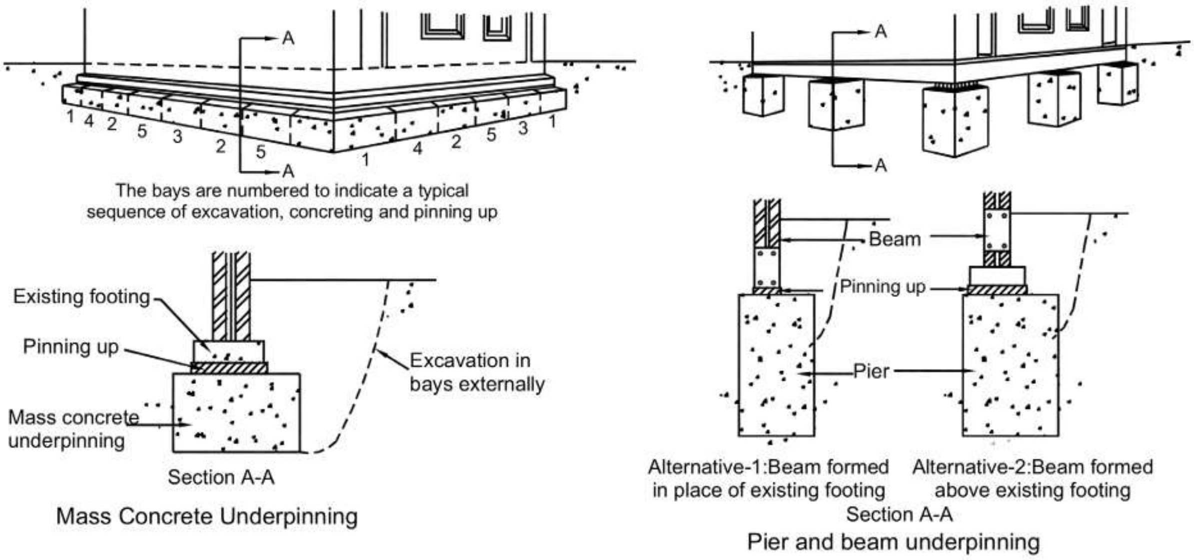
Yes, it is possible to prevent further movement by improving the stability of the underlying soil; for example, cavities in the soil caused by erosion can be filled by injecting cement-based grout under pressure. However, the stabilization of clay soils moving due to changes in moisture content is less straightforward. In theory, the characteristics of the clay and its tendency to change volume can be significantly altered by adding certain chemicals. Shrinkage potential in particular can be reduced by using lime which replaces sodium ions in the clay mineral with calcium ions. This technique is effective in the laboratory and can be used to treat clay fill, but the extremely low permeability of most shrinkable clays makes the technique extremely difficult to use in field.



Differential settlement damage resulting from uneven soil properties[10]

5.9 What is underpinning and how is this technique used for repairing the existing foundation of a house?

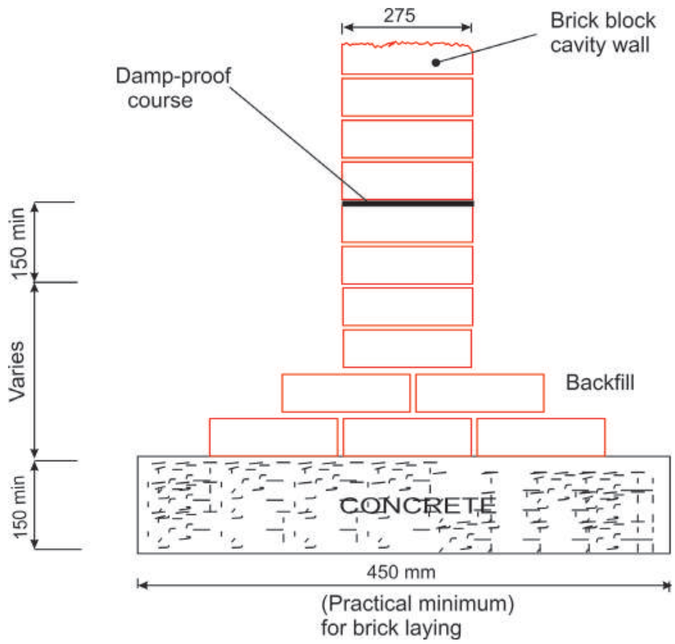
If foundations fail, a prompt and competent repair is required. One solution is to do underpine of the foundations, i.e. either providing new foundations or extending the existing ones downwards to reach more stable ground. Foundation area depends on the bearing capacity of the underlying soil. The depth of foundation should be deep enough so that it is not affected by moisture changes. Floor slabs are also susceptible to damage because of clay shrinkage and swelling. It is advisable to suspend a floor or a 150 mm depth of void is recommended under it, rather than a slab bearing on the ground.

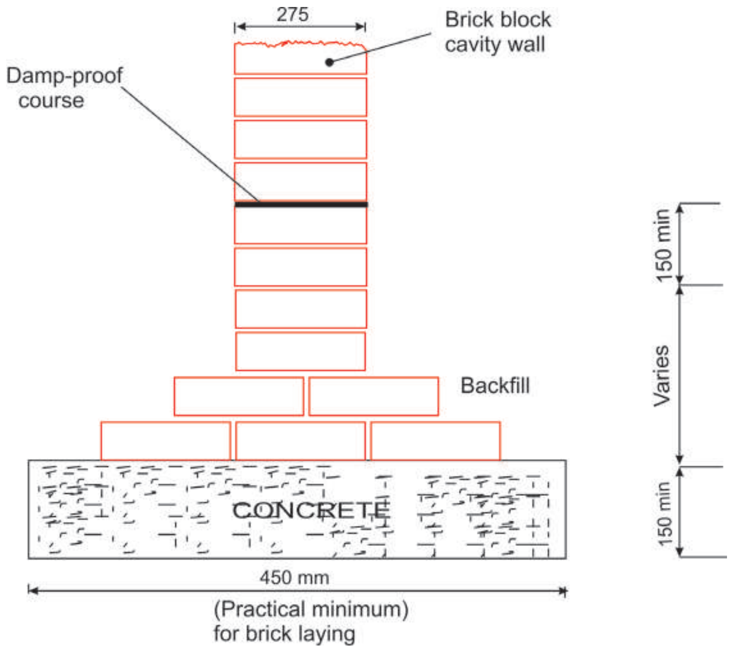


Underpinning of existing foundation of a house[10]

5.10 What precautions have to be considered in laying the foundation of the extension of a house?

The foundation requirements for extensions are essentially the same as those for new houses, with one important provision i.e. a vertical movement joint about 10mm wide throughout the height of the building including foundation. Movement joints are essentially deliberate cracks, helping to prevent the walls from cracking and it should be adequately protected from rain and wind.





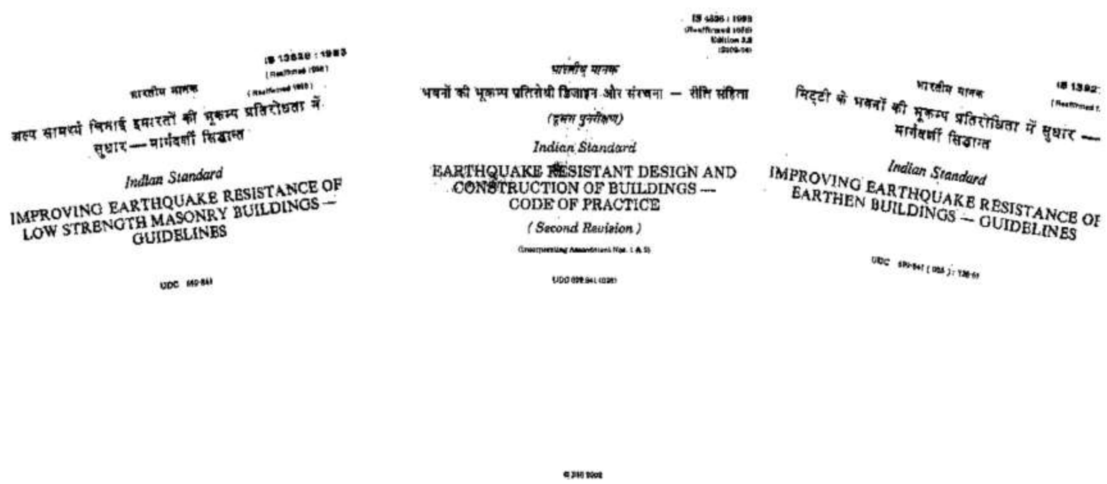
Foundation requirement for the extension of a house

Chapter 6

Earthquake Resistant Provisions in a Masonry House

6.1 What are the criteria for earthquake resistant provisions in masonry houses?

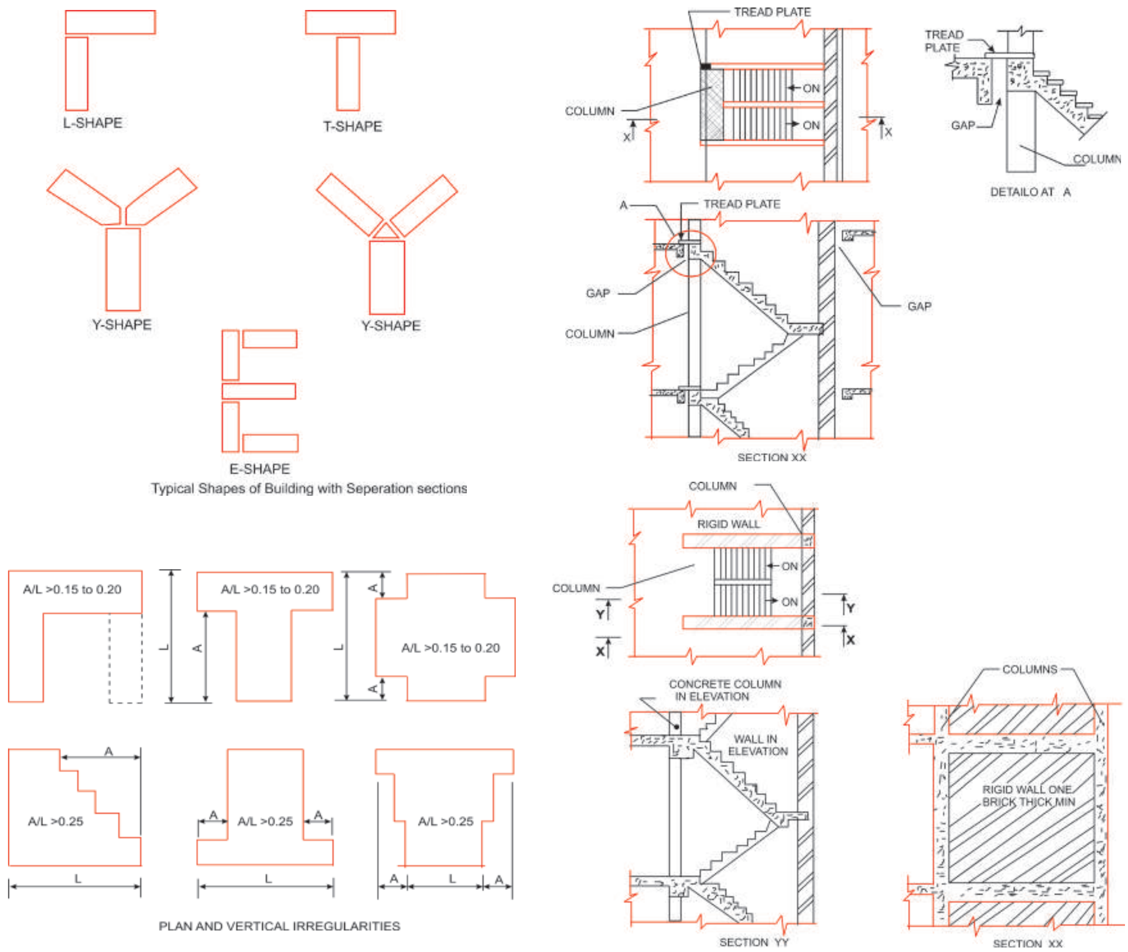
Masonry building is usually designed for dead and live loads ignoring the seismic load since the earthquake codes have not been incorporated into building regulations in past. On the contrary masonry construction has several inherent weaknesses which make it more vulnerable to earthquake forces like absence of integration among all the load bearing walls and connections between floor and roof, limited ductility, heavy mass etc. Therefore, to construct a masonry house some additional features must also be incorporated in the conventional construction known as earthquake resistant (ER) measures. These earthquake resistant features along with the general guidelines are given in IS: 4326 and IS: 13928 which deals with the selection of materials, special features of design and construction using rectangular masonry units, timber construction and building with prefabricated flooring/roofing elements. A guideline for construction of earthen buildings is covered in a separate code i.e. IS 13927: 1993.



BIS codes on earthquake resistant provisions in masonry houses

6.2 What are the general provisions laid by the BIS codes for the earthquake resistant house?

- Building (particularly roof and upper storey) should be light in weight
- Integrity and continuity in construction so that it forms a continuous load path between the foundation and all diaphragm levels, and ties all portions of building together
- Projection/ suspended ceiling should be avoided while other should be reinforced and firmly attached to main structure
- Plans & elevation should be symmetrical with respect to mass and stiffness, other wise use separation joints
- Avoid close proximity between buildings to avoid pounding, use separation gap and minimum width of separation shall be 25mm
- Use either separated staircase or enclosed with rigid walls when not possible to use sliding joint
- Sloping roof system be adequately braced in both orthogonal direction (horizontal tie member and cross bracing) and be properly anchored into the RC band.
- Foundation should be firm and uniform, otherwise separate the building in different units. In case of loose soil, improve the soil



General Provisions for masonry construction as per IS: 4326[15]

6.3 What are the provisions for masonry unit and mortar for the earthquake resistant house?

Masonry unit

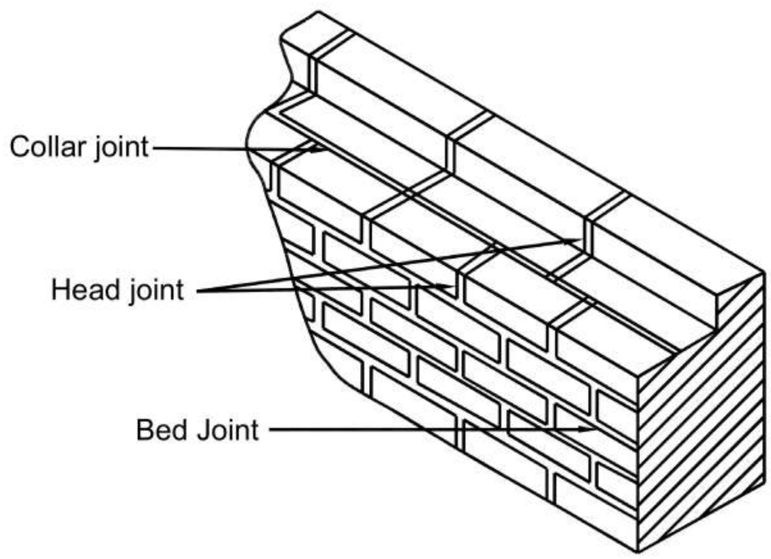
- Well burnt bricks or solid concrete blocks having a crushing strength > 35 MPa
- Squared stone masonry, stone block masonry or hollow concrete block masonry, as specified in IS: 1597 (Part 2): 1992 having adequate strength

Mortar

- Category A: M_2 (Cement-sand 1:6) or M_3 (Lime-cinder 1:3) or even richer
- M_2 (Cement-lime- sand 1:2:9 or Cement sand 1:6) or richer§
  H_2 (Cement- sand 1:4) or M_1 (Cement-lime-sand 1:1:6) or richer

Masonry Bond

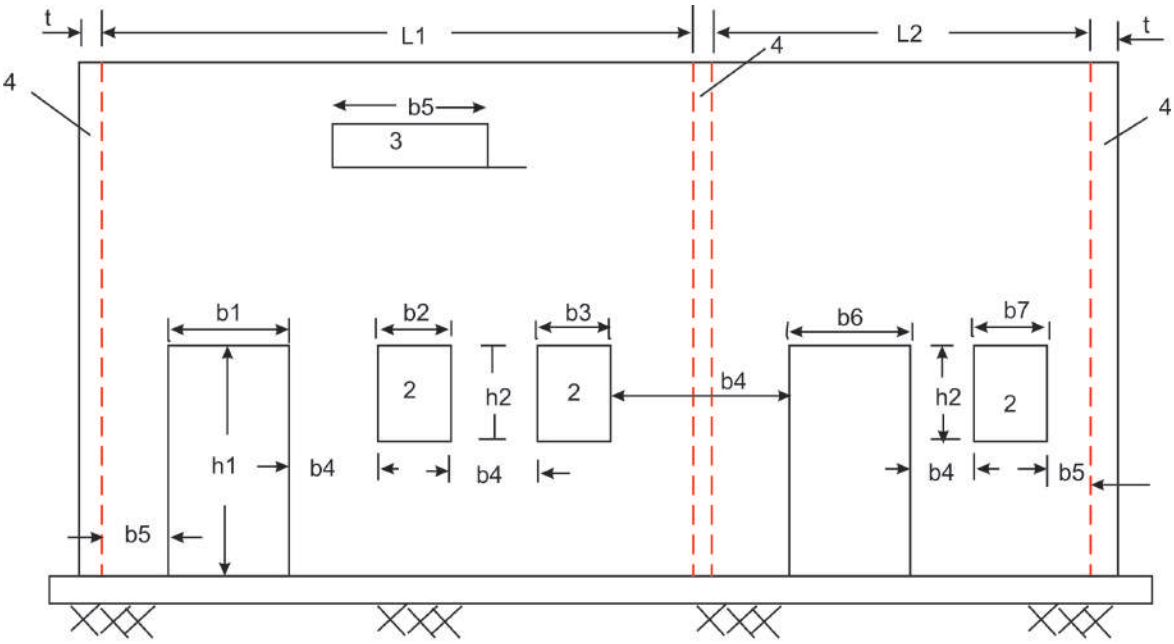
- Usual bond but vertical joints should be broken properly from course to course
- Make a sloping joint by making the corner first to a height of 600mm and then bulging the wall in between them
- A toothed joint perpendicular walls, alternatively in lifts of about 450mm



Alternating toothed joints in walls at Corner and T- Junction[15]

6.4 What are the provisions for openings in walls of an earthquake resistant masonry house as per BIS codes?

Openings in door and window should be as small as possible, these should be placed centrally. Top level of openings should be same, covered with lintel band. If do not comply with code, strengthen by RC lining with 2 HYSD of 8f. Avoid arches over the opening otherwise use steel ties.

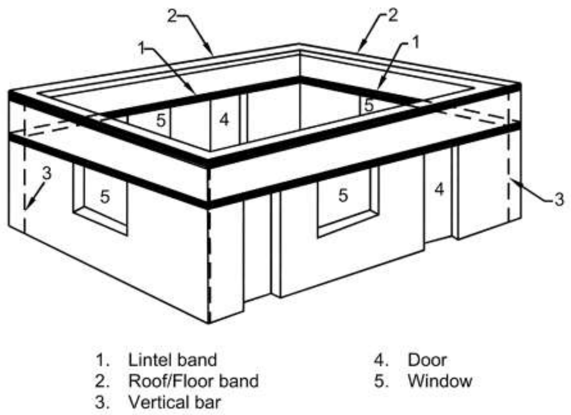


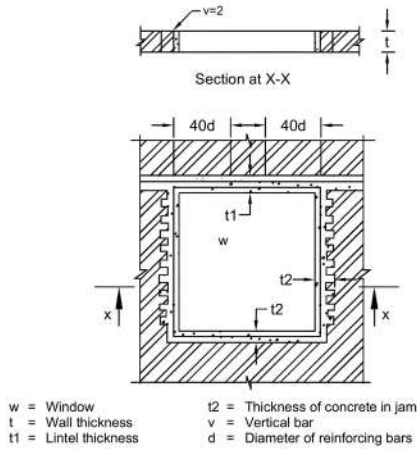
Details of opening in a masonry wall of a house as per IS: 4326[15]

Size and Position of Openings in Bearing Walls

S. No.	Position of Opening	Details of Opening for Building category		
		A and B	C	D and E
1.	Distance b_5 from the inside corner of outside wall, *Min*	Zero mm	230 mm	450 mm
2.	For total length of Openings, the ratio $(b_1 + b_2 + b_3)/I_1$ ɒ $(b_6 + b_7)/I_2$ *shall not exceed*: a) one-storeyed building b) two- storeyed building c) 3 or 4-storeyed building	 0.60 0.50 0.42	 0.55 0.46 0.37	 0.50 0.42 0.33
3.	Pier width between consecutive openings b_4, *Min*	340 mm	450mm	560 mm
4.	Vertical distance between two openings one above the other h_3, *Min*	600 mm	600 mm	600 mm
5.	Width of opening of ventilator b_8, *Max*	900 mm	900 mm	900mm

6.5 What are the seismic strengthening provisions laid by the BIS codes for the earthquake resistant house?





Seismic strengthening arrangement as per IS: 4326[15]

Strengthening Arrangements Recommended for Masonry Buildings (Clause 8.4.1)

Building Category	Number of Stories	Strengthening Provisions in all Storey
(1)	(2)	(3)
A	1 to 3	a
	4	a,b,c
B	1 to 3	a,b,c,f,g
	4	a,b,c,d,f,g
C	1 and 2	a,b,c,f,g
	3 and 4	a to g
D	1 and 2	a to g
	3 and 4	a to h
E	1 to 3*	a to h

Where

Masonry mortar (see 8.1.2).

Lintel band (see 8.4.2).

Roof band and gable band where necessary (see 8.4.3 and 8.4.4).

Vertical steel at corners and junctions of walls (see 8.4.8)

Vertical steel at jambs of openings (see 8.4.9)

Bracing in plan at tie level of roofs (see 5.4.2.2).

Plinth band where necessary (see8.4.6), and

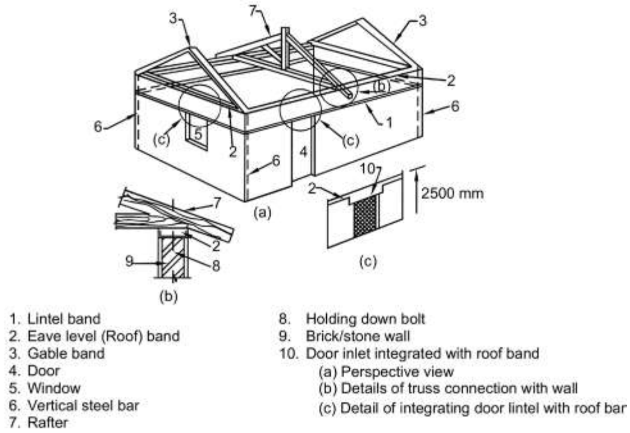
Dowel bars (see 8.4.7).

*4th storey not allowed in category E.

NOTE: In case of four storey buildings of category B, the requirements of vertical steel may be checked through a seismic analysis using a design seismic coefficient equal to four times the one given in (a) 3.4.2.3 of IS 1893:1984. (This is because the brittle behaviour of masonry in the absence of vertical steel results in much higher effective seismic force than that envisaged in the seismic coefficient provided in the code). If this analysis shows that vertical steel is not required the designer may take the decision accordingly.

6.6 What is the life safety band in a house to protect from earthquake?

Masonry houses are strengthened by horizontal bands or bond beams at critical levels with vertical reinforcing bars at corners and junctions of walls forming a "box" like framing system which improve the integral action as well as transfer the earthquakes forces from the floors to structural walls to foundations. These horizontal bands are called life safety bands for a house and depending upon their location it may be termed as roof, lintel, sill, and plinth band. The reinforcing details of these bands are available elsewhere (IS 4326, 13928, IAEE etc). In combination with vertical reinforcement, it improves the strength, ductility and energy dissipation capacity of a house. The descriptions of each horizontal band with its individual function are as follows:



Overall arrangement of reinforcing masonry building having pitched roof[15]

Plinth Band: Provided at the plinth level of walls on the top of the foundation, it is useful in sustaining differential settlements particularly when foundation soil is soft or has uneven properties. Plinth band is overlaid by damp proof courses to prevent a passage of moisture from the ground to the structures.

Gable Band: It is provided at the top of gable masonry below the purlins. The band shall be made continuous with the roof band at the eave level to restrict the out-of-plane failure of gable wall, which is susceptible to earthquake forces.

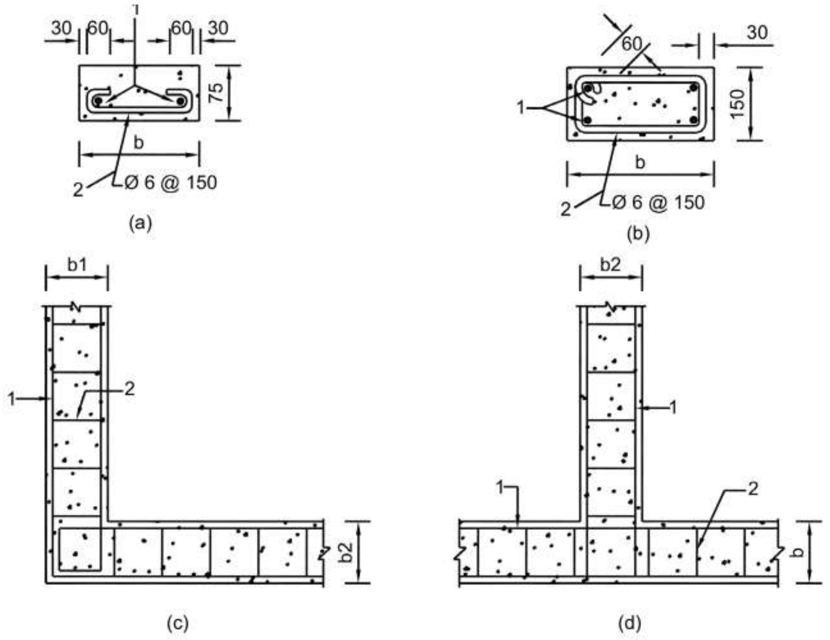
Roof Band: Roof band is similar to lintel band but is provided below the roof or floors to improve the in-plane rigidity of horizontal floor diaphragms.

Lintel Band: It is provided at lintel level on all internal and external longitudinal and cross walls including partition walls. It gives integrity to the structure and resistance to out-of-plane wall bending. It enhances the stability of partition walls and prevents the collapse of roof.

Sill Band: Similar to lintel band, it is provided at sill level to reduce the effective height of masonry piers between openings resulting in reduction of shear cracking in piers.

Vertical Steel: It is provided at corners and junctions of walls and around jambs of doors and windows. It is embedded in plinth masonry of foundation, roof slab or band so as to develop its tensile strength in bond. It should pass through the lintel bands and floor slabs in all stories in the form of steel bar of 10mm to 12mm diameter.

6.7 What are the details of life safety bands in a house to protect from earthquake?



1. Longitudinal bars
2. Lateral ties
b1, b2 - Wall thickness

(a) Section of band with two bars
(b) Section of band with four bars
(c) Structural plan at corner junction
(d) Structural plan at T-junction of walls

All dimensions are in mm.

***Details of life safety bands*[15]**

Recommended Longitudinal Steel in Reinforced Concrete Bands (*Clause 8.4.5*)

Span	Building Category B		Building Category C		Building Category D		Building Category E	
	No. Of Bars	Dia	No. Of Bars	Dia	No. Of Bars	Dia	No. Of Bars	Dia
(1) m	(2)	(3) mm	(4)	(5) mm	(6)	(7) mm	(8)	(9) mm
5 or less	2	8	2	8	2	8	2	10
6	2	8	2	8	2	10	2	12
7	2	8	2	10	2	12	4	10
8	2	10	2	12	4	10	4	12

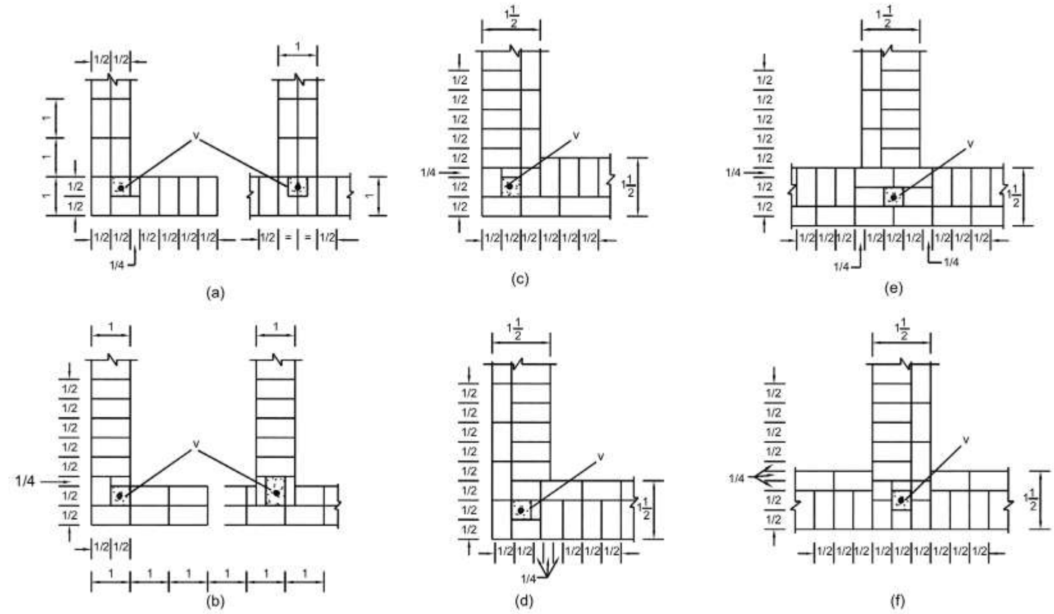
NOTES:

1. Span of wall will be the distance between centre lines of its cross walls or buttresses. For spans greater than 8m it will be desirable to insert pilasters or buttresses to reduce the span or special calculations shall be made to determine the strength of wall and section of band.
2. The number and diameter of bars given above pertain to high strength deformed bars. If plain mild-steel bars are used keeping the same number, the following diameters may be used:

High Strength Def. Bar dia	8	10	12	16	20
Mild Steel Plain bar dia	10	12	16	20	25

3. Width of R.C band is assumed same as the thickness of the wall. Wall thickness shall be 200 mm minimum. A clear cover of 20 mm from face of wall will be maintained
4. The vertical thickness of RC band be kept 75 mm minimum, where two longitudinal bars are specified, one on each face; and 150 mm, where four bars are specified.
5. Concrete mix shall be of grade M15 of IS 456:1978 or 1:2:4 by volume.
6. The longitudinal steel bars shall be held in position by steel links or stirrups 6 mm dia spaced at 150 mm apart.

6.8 How to provide vertical reinforcement in a house to protect from earthquake?



1- One brick length, 1/2-Half brick length, V- Vertical steel bar with mortar/concrete filling in pocket
(a) and (b) - Alternate courses in one brick wall
(c) and (d) - Alternate courses at corner junction of $1\frac{1}{2}$ brick wall
(e) and (f) - Alternate courses at T-junction of $1\frac{1}{2}$ brick wall

Details of vertical reinforcement[15]

Vertical Steel Reinforcement in Masonry Walls with Rectangular Masonry Units

No. of storeys	Storey	Diameter of HSD Single Bar in mm at Each Critical Section			
		Category B	Category C	Category D	Category E
One	--	Nil	Nil	10	12
Two	Top	Nil	Nil	10	12
	Bottom	Nil	Nil	12	16
Three	Top	Nil	10	10	12
	Middle	Nil	10	12	16
	Bottom	Nil	12	12	16
Four	Top	10	10	10	Four storeyed building not permitted
	Third	10	10	12	
	Second	10	12	16	
	Bottom	12	12	20	

NOTES
1. The diameters given above are for H.S.D. bars. For mild-steel plain bars, use equivalent diameters as given under Table 6 Note 2.
2. The vertical bars will be covered with concrete M15 or mortar 1:3 grade in suitably created pockets around the bars (*see* Fig.12). This will ensure their safety from corrosion and good bond with masonry.
3. In case of floors/roofs with small precast components, also refer 9.2.3 for floor/roof band details.

6.9 What do International Codes recommend for the earthquake resistant house?

Euro code 8 provides some rules for simple low-rise masonry buildings which can be used without further quantitative analysis. The main rules concern the minimum area of wall that should be provided in each direction as a % of the total floor plan area; as given in table below[25];

Rules for minimum area of shear walls for 'simple' masonry buildings, from Eurocode 8

		Acceleration at site, $a_g S$			
		$\le 0.07kg$	$\le 0.10kg$	$\le 0.15kg$	$\le 0.20kg$
Type of construction	Number of storeys, n (note 2)	Minimum sum of cross-sectional areas of horizontal shear walls in each direction, as percentage of the total floor area per storey, $p_{A,min}$			
Unreinforced masonry	1	2.0%	2.0%	3.5%	n/a
	2	2.0%	2.5%	5.0%	n/a
	3	3.0%	5.0%	n/a	n/a
	4	5.0%	n/a (note 4)	n/a	n/a
Confined masonry	2	2.0%	2.5%	3.0%	3.5%
	3	2.0%	3.0%	4.0%	n/a
	4	4.0%	5.0%	n/a	n/a
	5	6.0%	n/a	n/a	n/a
Reinforced masonry	2	2.0%	2.0%	2.0%	3.5%
	3	2.0%	2.0%	3.0%	5.0%
	4	3.0%	4.0%	5.0%	n/a
	5	4.0%	5.0%	n/a	n/a

Notes:
1. The table is based on a minimum compressive strength of 12N/mm^2 for unreinforced masonry and 5N/mm^2 for confined and reinforced masonry.
2. Roof space above full storeys is not included in the number of storeys.
3. For buildings where at least 70% of the shear walls under consideration are longer than 2m, the factor k is given by $k = 1 + (l_{av} - 2)/4 \le 2$ where l_{av} is the average length, expressed in meters, of the shear walls considered. For other cases k=1.
4. n/a means 'not acceptable'.

There are a number of additional features that make an earthquake resistant masonry house;

EC8 specifies a horizontal concrete beams or steel ties to be placed around the building perimeter at every floor level with a minimum steel area of 200 mm^2.The plan shape must be approximately rectangular, with a recommended minimum ratio of shortest to longest side of 0.25, and with projection of recesses from the rectangular plan area not exceeding 15%.

The building should be stiffened by shear walls (load bearing walls), arranged symmetrically in plan in two orthogonal directions

A minimum of two parallel walls should be placed in two orthogonal directions, the length of each wall be greater than 30% of the length of the building in the direction of wall under consideration.

At least for the walls in one direction, the distance between these walls should be greater than 75% of the length of the building in the other direction.

At least 75% of the vertical loads should be supported by the shear wall.

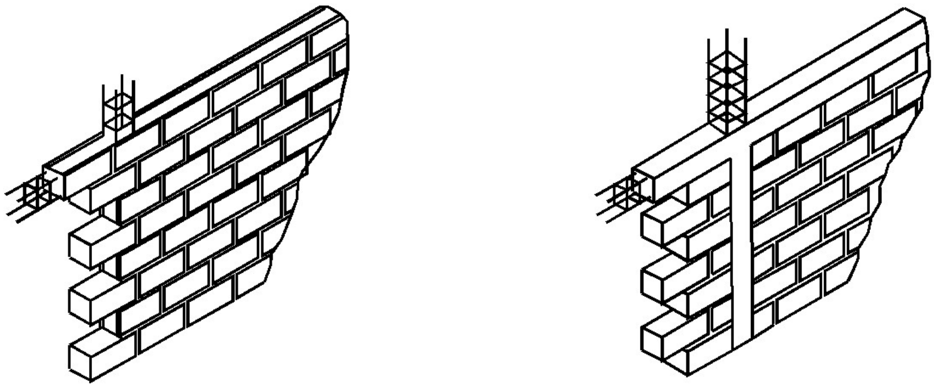
Shear walls (load bearing walls), should be continued from the top to the bottom of the building

Difference in mass and shear wall area between any two adjacent storeys should not exceed 20%.Walls in one direction should be connected with walls in the orthogonal direction at a maximum spacing of 7m.

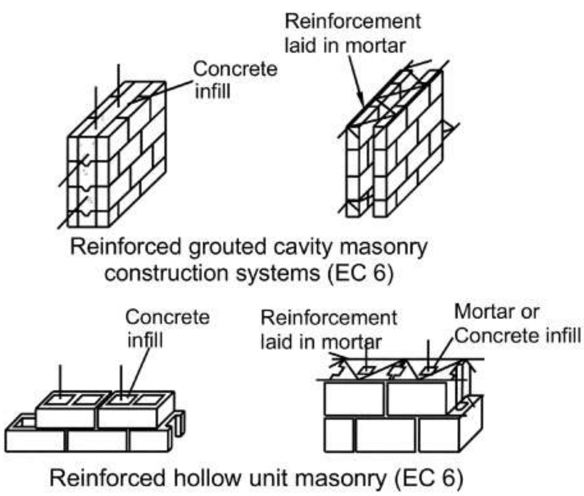
6.10 What are the other options to construct a high rise masonry building in severe seismic zones?

Two types of masonry constructions are prevalent i.e. *confined masonry* and *reinforced masonry* for high rise masonry buildings. As in case of unreinforced masonry house construction, a number of earthquake resistant measures are provided horizontally and vertically at regular intervals. Lintel band, roof band even sill band along the height of building and vertical steel at corners, door and window, jamb openings etc. are recommended along the length. On the similar concept, confined masonry is to be constructed which is a good and efficient alternative way of construction. Basic concept of confined masonry is that all walls in the house must be well confined on all four sides by reinforced members like bond beams and tie columns which are interconnected.

Reinforced masonry construction is another development in this series to construct a high rise earthquake resistant masonry building. Two types of reinforced masonry are used namely (i) Reinforced grouted masonry or simply grouted masonry and (ii) Reinforced hollow unit masonry. In grouted masonry, two wythes of bricks or solid concrete block or stone units are filled with grout, binding the two wythes together and providing a space wherein the reinforcement can be placed and bonded with the surrounding masonry.



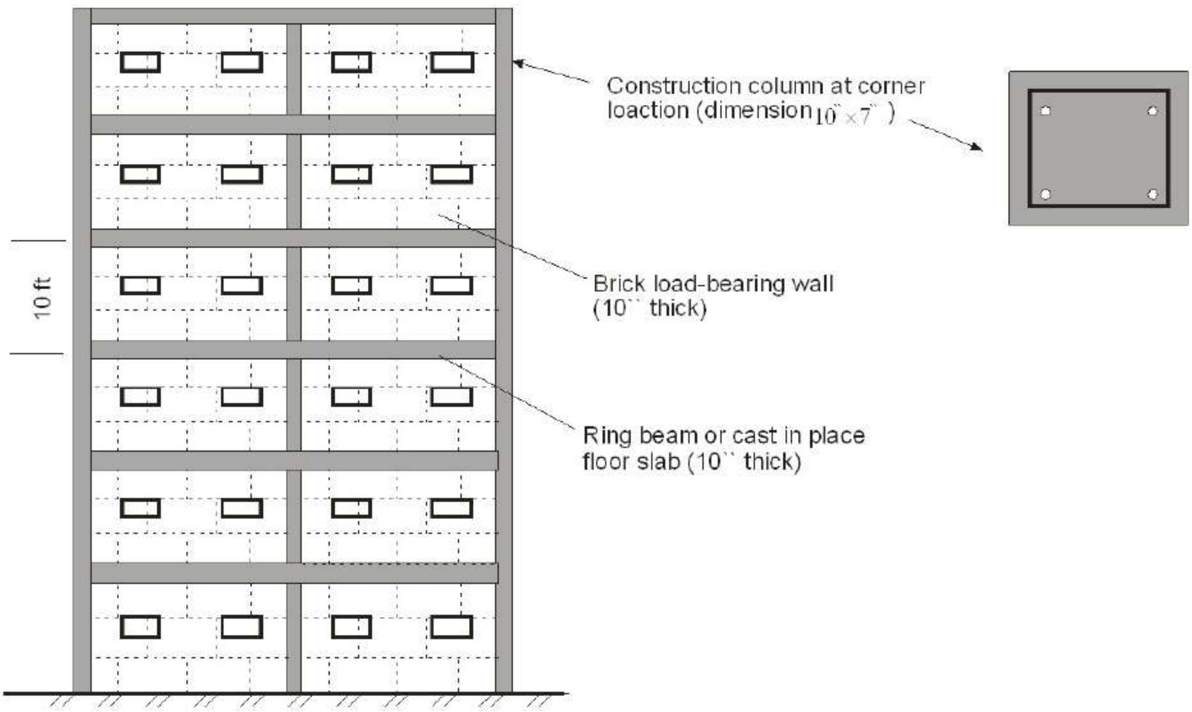
Masonry confined within (a) reinforced masonry and (b) reinforced concrete bond beam and column



Type of reinforced masonry[23]

6.11 What are the construction details of confined masonry?

Confined masonry buildings consist of load-bearing un-reinforced masonry walls, bricks or concrete blocks confined by cast-in-place reinforced concrete vertical tie columns. These tie columns with rectangular section of dimensions corresponding to the wall thickness and depth of about 200mm, are located at regular intervals and connected together with horizontal reinforced concrete tie beams cast after construction of masonry walls. The tie columns are required at least at the corners of the building, at the free ends of the walls and around openings exceeding 1.5 m^2 in area. In no case the spacing between tie columns exceed 5m. Horizontal tie beams are required at each floor level (or at 4m centers, if less) and around openings. Rules are given for the minimum longitudinal and transverse steel required in the confining elements. Both tie column and tie beams must have at least four 10mm diameter longitudinal reinforcement and stirrups of 6mm diameter must be spaced 100mm at the extremes and 200 mm at center of the elements. The confinement prevents damage due to out-of-plane bending effects and improves wall ductility. The confined masonry has exhibited good earthquake resistance in past earthquake and can safely be constructed upto 4 stories. This type of construction differs from masonry infill panels built into the concrete frame.



Details of confined masonry

6.12 How does the reinforced masonry be constructed?

In reinforced masonry construction, minimum % of steel in horizontal and vertical direction is required. Horizontally minimum 0.05% and vertically 0.08% of steel is required. The table summarizes[25] some requirements for steel in special reinforced masonry buildings in seismic region. Reinforced masonry can achieve much higher shear strength. The reinforcement not only increases the shear strength, but also provides ductility.

Minimum steel requirements in ACI 530 for special reinforced masonry buildings

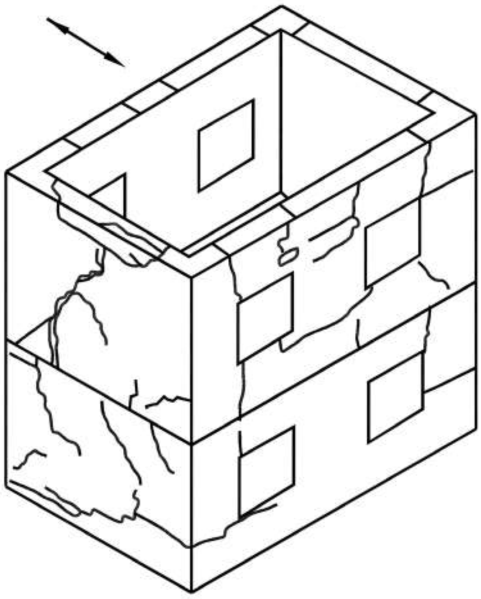
Maximum horizontal and vertical spacing reinforcing steel	(i) $\frac{\text{Length or height of building}}{3}$ (ii) 1219 mm if less
Minimum area of vertical steel	One third minimum required area of horizontal shear steel
Horizontal shear reinforcement must be anchored around vertical reinforcement with a standard hook	

Chapter 7

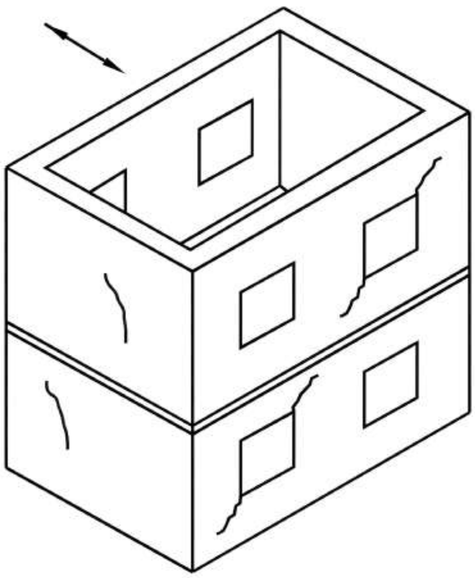
Strengthening and Retrofitting of a Seismically Deficient House before the next Earthquake Strikes

7.1 Why strengthening and retrofitting is required in case of existing undamaged masonry houses?

There are a large number of masonry houses or buildings not designed for seismic forces and also lacking in earthquake resistant provisions. In case of moderate to strong earthquakes such houses either completely collapse or severely damage causing massive death toll and extensive loss. The demolition or replacement of such houses is not feasible due to economic reasons, immediate shelter problems etc. The retrofitting is the only being economically feasible solution for seismic deficient house.



A seismic deficient house after earthquake (severe cracking/collapse)



A retrofitted house after earthquake (minor cracking without collapse)

7.2 Can an existing house be made earthquake resistant?

The seismic performance of an existing house may be improved by seismic retrofitting. However, the selection of retrofitting techniques differs from case to case. Therefore, it is desirable; that a design professional for the seismic retrofitting be employed to apply technical guidelines available on their relative merits, cost, effectiveness etc. Retrofitting without technical know how lead to more vulnerable house.

7.3 How to know whether the house is seismically deficient or not?

To check the seismic adequacy of a house is a specialized job. There is no well defined procedure but if earthquake resistant provisions are missing a house may be assumed as seismically deficient. Here is a list of common features which make a house seismically vulnerable or deficient. Let as begin from roof to foundation;

Presence of flexible roof/floor or diaphragm – A flexible roof is one whose in-plane rigidity is not sufficient to transfer the seismic forces from one to other elements. For example; tiled gable roof, wooden roof, an RCC or RB type roof of thickness less than 75mm etc.

No connection of floor with the walls – Improper connection in between the roof and wall causes damage in gable wooden or tile type roof. In case of RCC or RB slab, a friction connection may be assumed sufficient for transferring the seismic forces.

Insufficient wall area in both principal direction of a house –the minimum wall area remains less than requisition (at least 5% of plan area as per EC 8) to resist the seismic forces in both the principal directions. For adequate resistance against out-of-plane rigidity, a minimum thickness of wall 230 mm i.e. 9" is recommended to resist the out-of-plane forces.

Very large openings in walls – The openings in the walls reduce the seismic capacity in in-plane or out-of-plane. Ideally the opening must be small, centrally placed and reinforced.

Absence of connecting ties among the walls- Improper wall to wall connection checks the box like action of a house. Therefore absence of horizontal bands makes the house more vulnerable.

Absence of interconnection of two layers of same walls – In old masonry construction particularly in stone masonry, walls are constructed in multi-wythes i.e. multi-layer (inner and outer layers) with no proper bonding in two layers.

Absence of vertical reinforced elements from roof to foundation – No vertically reinforced concrete tie member at the corners or at the opening of the house will greatly reduce overturning capacity.

Absence of plinth band or beam- This may induce strong possibility of differential settlement.

Inadequate foundation – Isolated foundation prevents the building to behave as a single unit.

Inadequate connections of non-structural elements with structural system – Non-structural elements such as parapet walls are subjected to amplified motions, it may lead to out-of-plane failure. Similarly, balconies, overhang projection, canopies are also prone to failure due to the vertical component of earthquake motion.

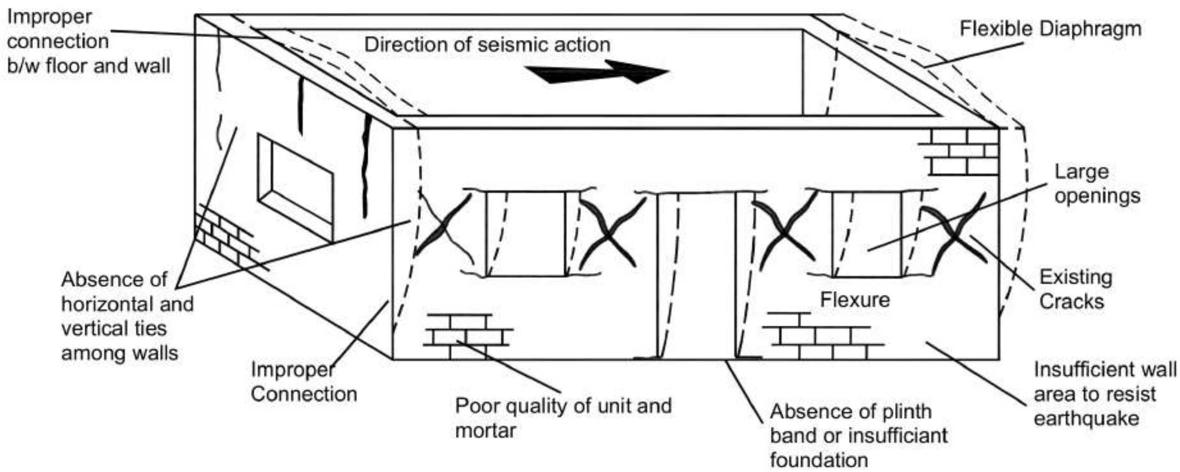
…….Continued

Some particular features in the old masonry which make a house more vulnerable are;

Presence of already existing cracks – Foundation settlement, expansion of masonry due to environmental reasons etc are some reasons causing cracks which reduce the lateral strength and provide a weaker structure.

Poor quality of mortars - The quality of the mortar deteriorates with the age of construction resulting in decrease of tensile, shear and bond strength.

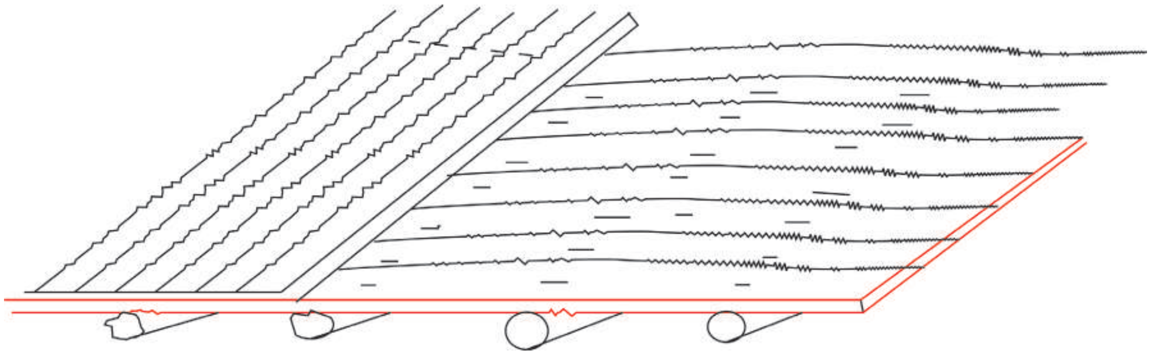
These are the common seismic deficiencies present in a masonry construction. In the subsequent slides the retrofitting solution of each seismic deficiency are presented in general. On this basis, the design professional or engineer may modify the given retrofitting technique for the particular house or can also adopt alternate techniques on the similar lines.



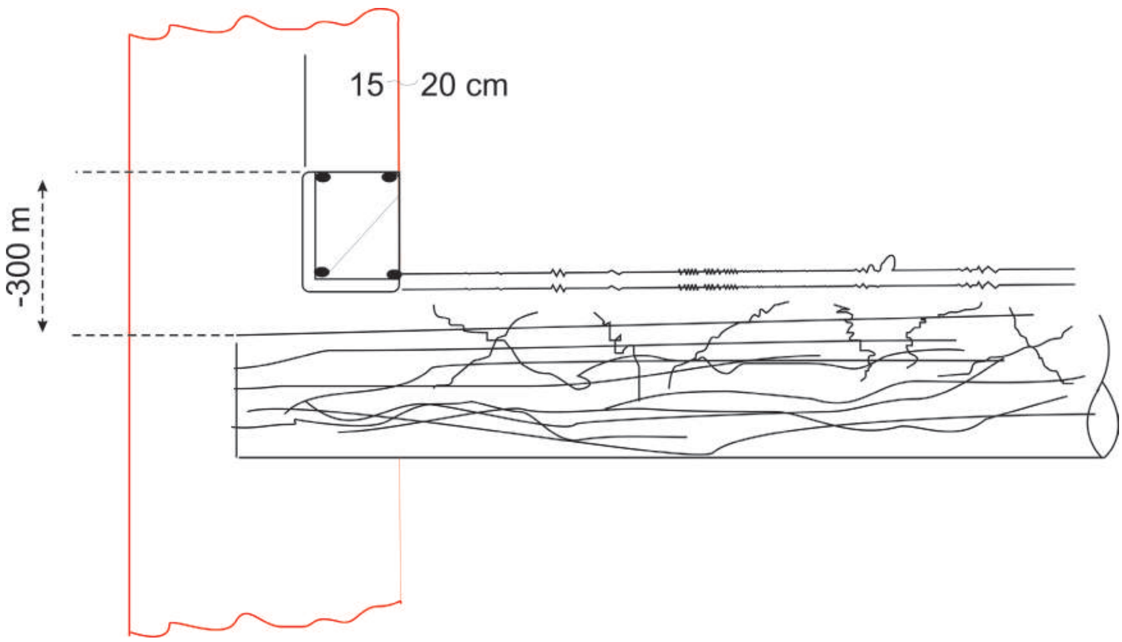
Common causes that makes a house seismic vulnerable[23]

7.4 How far the seismic performance be improved in case of flexible roof/floor or diaphragm?

There are two methods of stiffening an existing wooden floor connecting to the surrounding walls. The first is based on the use of traditional materials i.e. wood and steel. The second implies creation of a composite structure formed by the wood structure and by reinforced concrete thin slab. Replacement of flexible wooden floors by RC slabs is a common practice in the renovation of masonry buildings because RC slabs are more durable than flexible wooden floors and require little maintenance, providing better serviceability, and a flat ceiling surface. For seismic resistance point of view cast -in-situ RC slabs well anchored to the walls provide spatial interconnection of walls with distribution of lateral forces through their in-plane rigidity. Finally, the weight of cast-in-situ RC slabs increases the compressive vertical stresses of walls and reduces principal tensile stresses under the combination of gravity and earthquake loads.



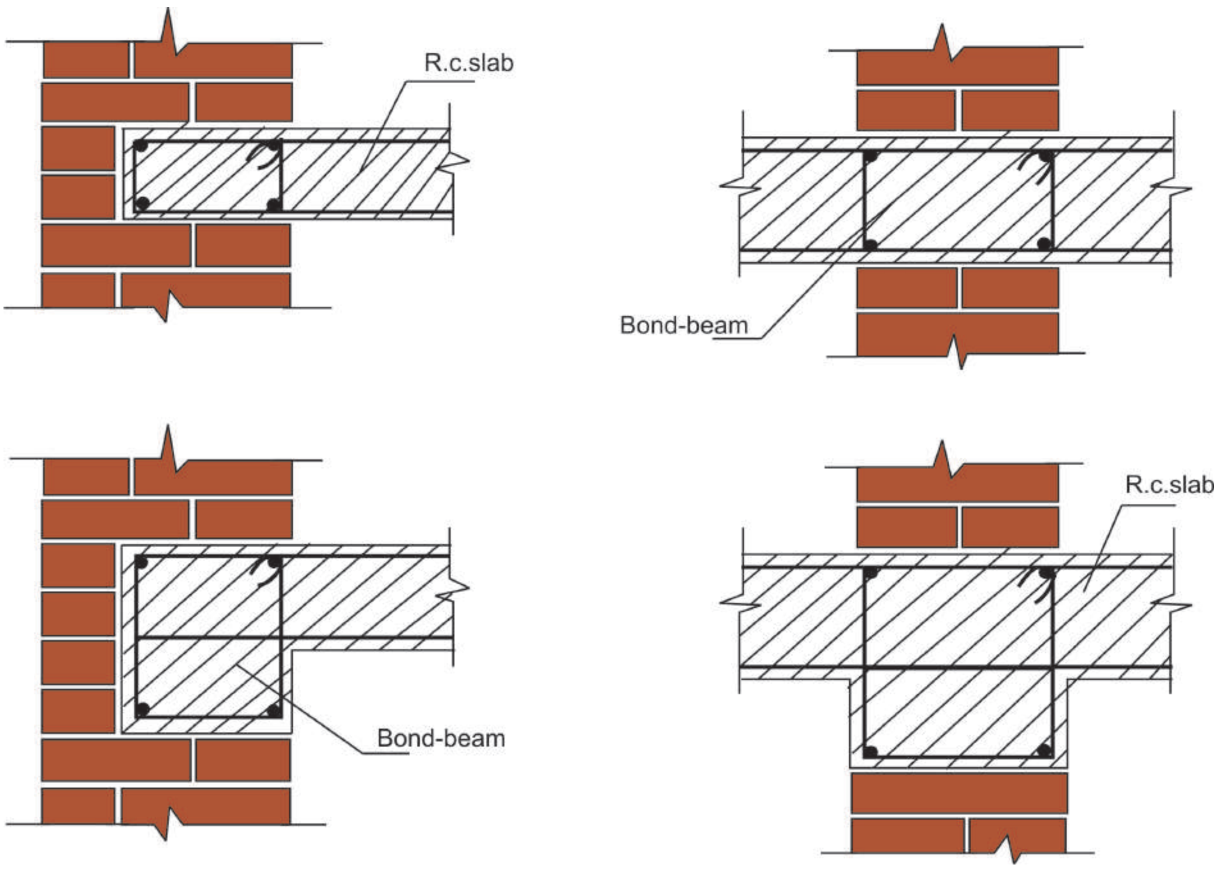
Stiffening an existing wooden floor by the use of traditional materials i.e. wood and steel



Stiffening an existing wooden floor by the use of composite materials i.e. wood and RC[23]

7.5 How can a good connection be provided between roof/floor and walls?

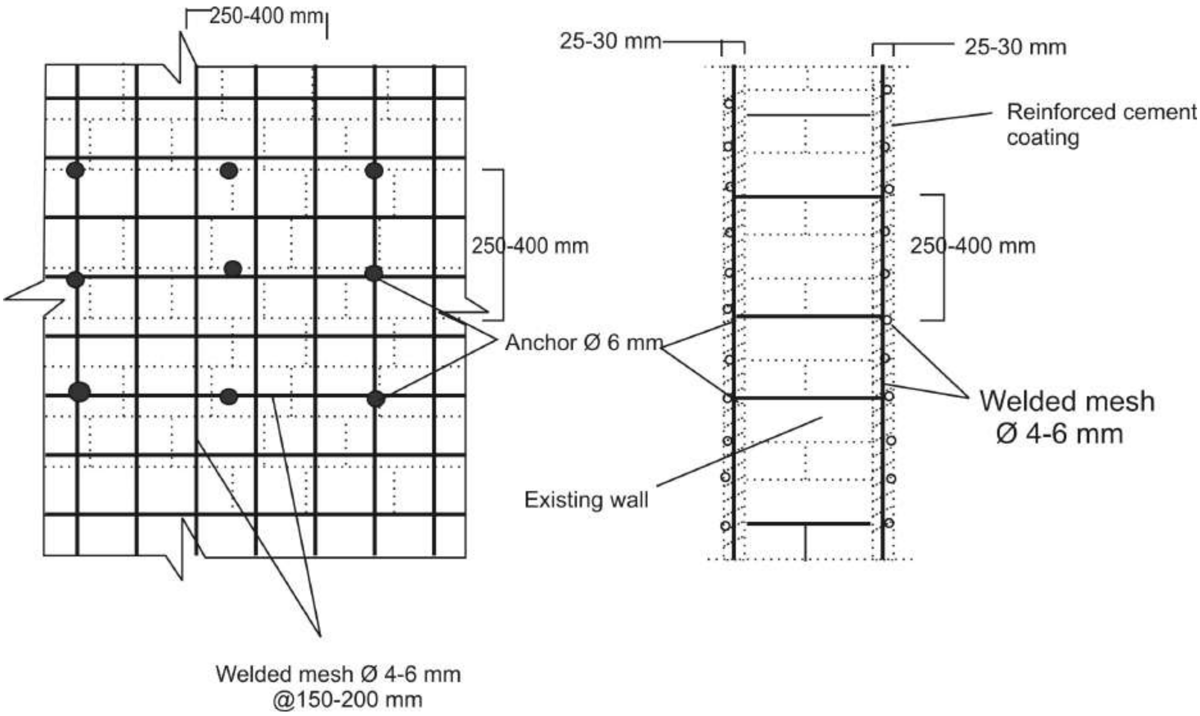
The connection between the masonry walls with floor/roof is a vital link on which the distribution of earthquake forces depend and ensures a monolithic and integrated behavior of structure to resist the earthquake forces.. A poor connection between the floors and walls is responsible for the independent behavior of wall and transverse walls may collapse due to out-of-plane forces. In most existing buildings, the floor and roofing joists have only gravity connection with the walls – typically, direct bearing with sparse anchorage. These gravity connections are inadequate in resisting out-of-plane seismic force. Therefore, the roof has to be properly connected to the walls through appropriate keys.



***Proper way of connection between the masonry walls and a RC floor/roof*[23]**

7.6 How to increase the thickness of wall or shear area of walls by using Ferro-cement to resist earthquake forces?

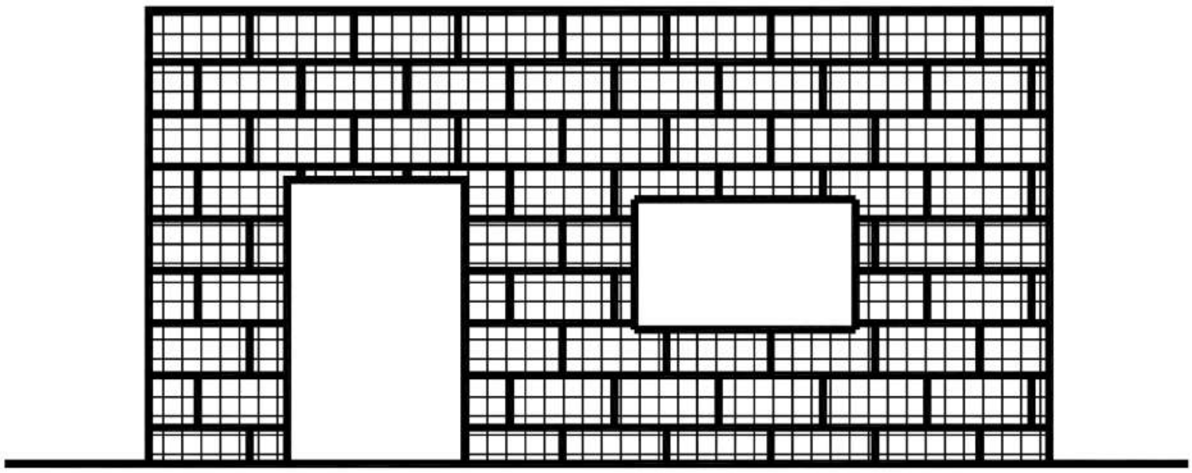
Ferro-cement consists of closely spaced multiple layers of hardware mesh embedded in a high strength cement mortar layer with a reinforcement ratio of 3-8% and mortar strength in between 15-30 MPa. Ferro-cement may be a solution to increase the in-plane as well as out-of-plane capacity of wall. Total thickness may vary from 10 to 50mm and mortar is toweled on through the mesh with covering thickness of 1-5mm.Typical mortar mix consists of 1 part cement, 1-3 parts sand with approximately 0.4 w/c ratio. The behavior of mortar can be improved by adding 0.5 to 1% of a low-cost fiber such as polypropylene. The mesh helps to confine the masonry units and thus improves in-plane inelastic deformation capacity.



Surface treatment by Ferro-cement

7.7 Can FRP be useful to increase the in-plane and out-of-plane capacity of a masonry wall?

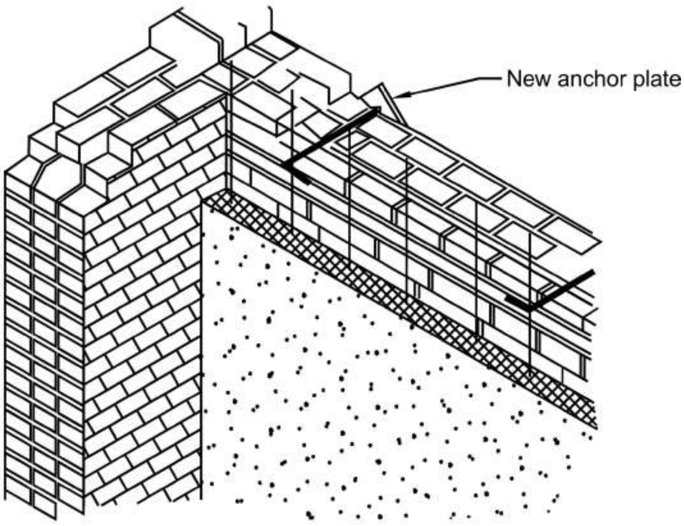
A layer of fiber reinforced polymers (FRP) on the surface of the walls which has high strength to weight ratio, stiffness to weight ratio, corrosion and fatigue resistance may be another alternative of steel mesh. FRP consisting of stiff and strong reinforcing fibers (primarily carbon and glass), held together.

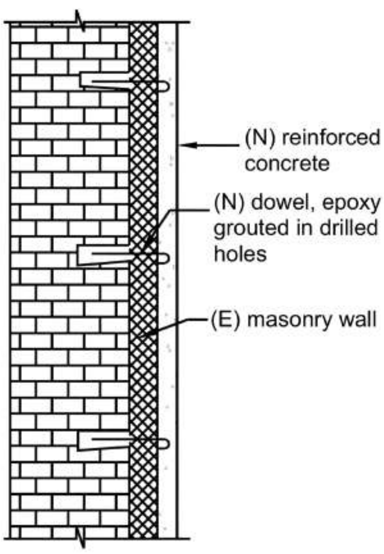


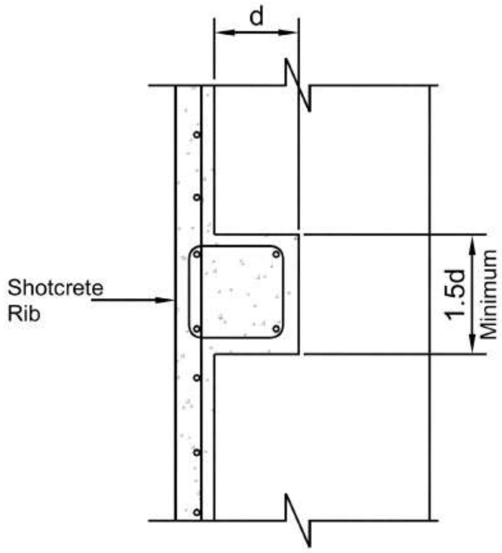
Brick masonry wall confined by wrapping with FRP

7.8 What is shotcrete and how it can be applied to increase the in-plane capacity of masonry walls?

In the case of weak masonry, absence of enough solid piers, or inadequate shear wall area, shotcrete is ideally suitable which is concrete mix pneumatically applied to the surface of a masonry wall over a mesh of reinforcing bars with an overlay thickness of at least 60mm typically reinforced with a welded wire fabric. Shear dowels (6-12mm diameter @ 25 -120 mm) are fixed using epoxy or cement grout into holes drilled to transfer the shear stress across shotcrete masonry interface and the masonry wall. Moreover design of concrete mix, selection of correct process and skilled workmanship are essential for effective use of shotcrete. It significantly increase the ultimate load of the retrofitted wall as the shotcrete overlay is assumed to resist all the lateral loads applied to a retrofitted wall with the brick masonry being neglected altogether.



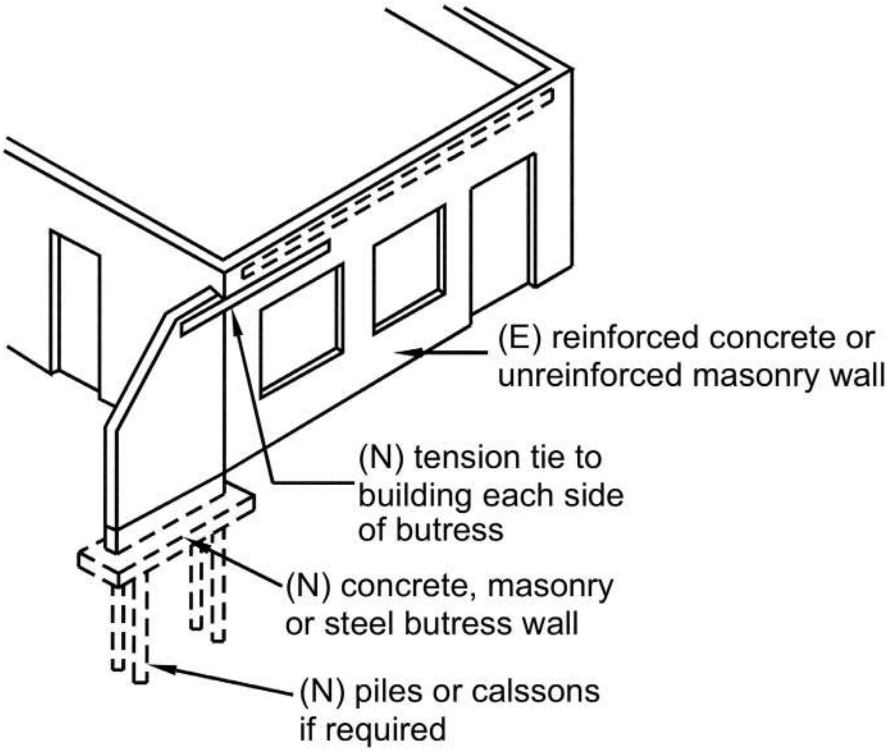




Typical details for providing Shotcrete and alternative details for fixing of reinforcement with the existing wall[11, 19]

7.9 How the external buttress may be helpful to increase the in-plane capacity of masonry walls?

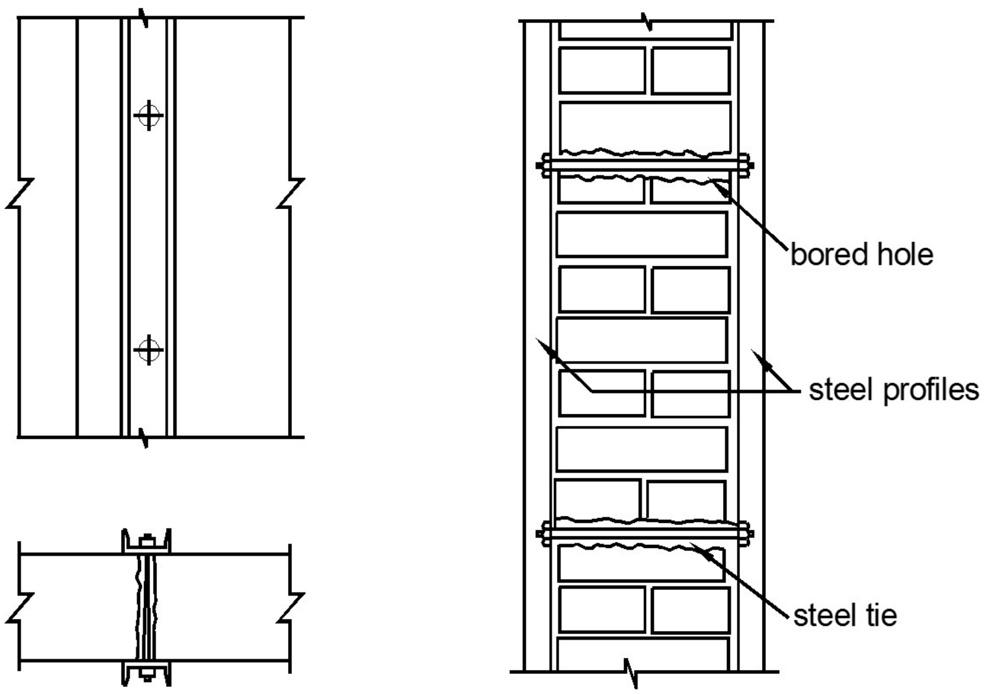
Provision of additional external buttresses at the perimeter of the building are the examples of exterior supplemental devises to increase in-plane strength of the existing masonry walls. The buttress has sufficient capacity against overturning forces and uplift forces therefore it requires an additional foundation. It should have proper connection with the existing walls through dowel. This technique has limitations in case of buildings constructed at the property lines or not having much space.



Details for providing external buttress and its connection with existing wall[11]

7.10 How can out–of-plane capacity of the walls be enhanced by using steel sections?

Use of steel sections as buttresses may prove to be the best solution to increase the out-of-plane strength of wall. Strengthening schemes consist of two steel sections (channel section or angle section) having full wall height placed on both the sides of the existing wall and attached to the roof or floor diaphragms. Steel sections are interconnected with each other in between by drilling in anchors through masonry at regular intervals of 50cm. The steel sections may be painted or covered with cement plaster to protect from corrosion.

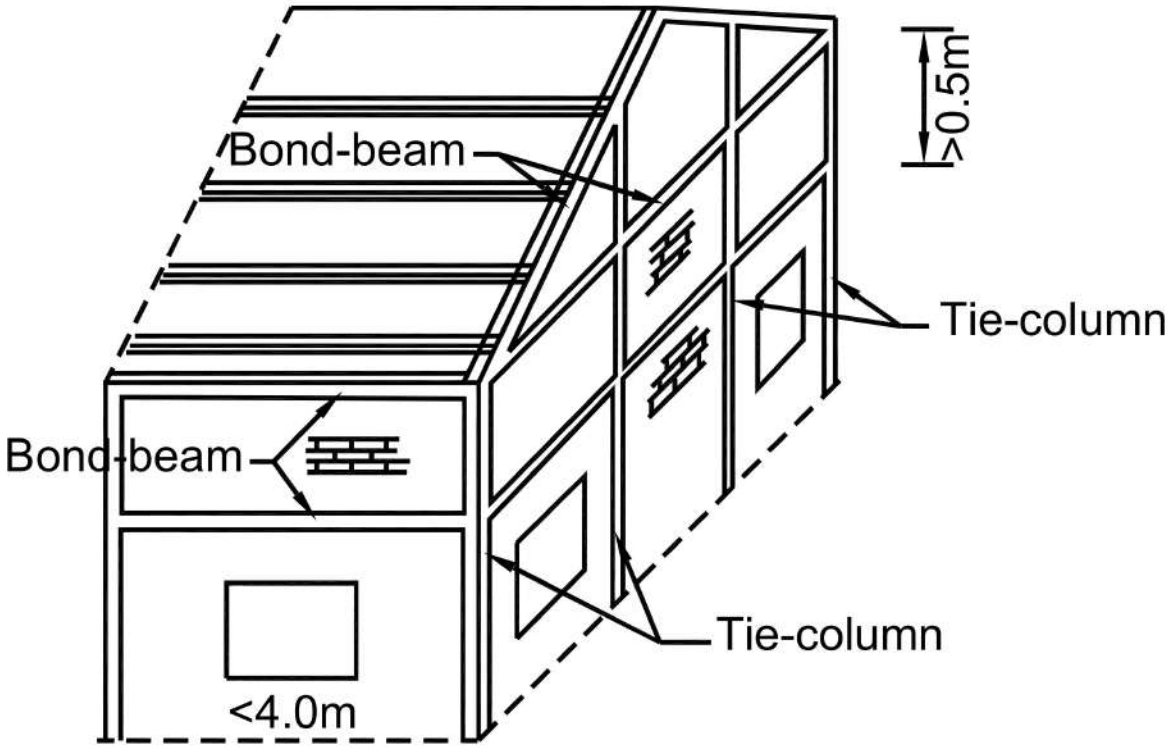


Strengthening of existing un-reinforced masonry by confinement with steel sections[27]

7.11 Can cross tie members increase the out–of-plane capacity of the walls?

The crosstie elements (tie beam and tied column) may also be used to ensure integral action of bearing walls like a crate.

The splint and bandage is another approach to strengthen the walls as well as bind them together economically. The horizontal bands (bandage) and vertical steel (splints) are welded which consist of welded wire mesh provided on both outer and inner surfaces at critical sections nailed to the masonry and covered with micro-concrete.

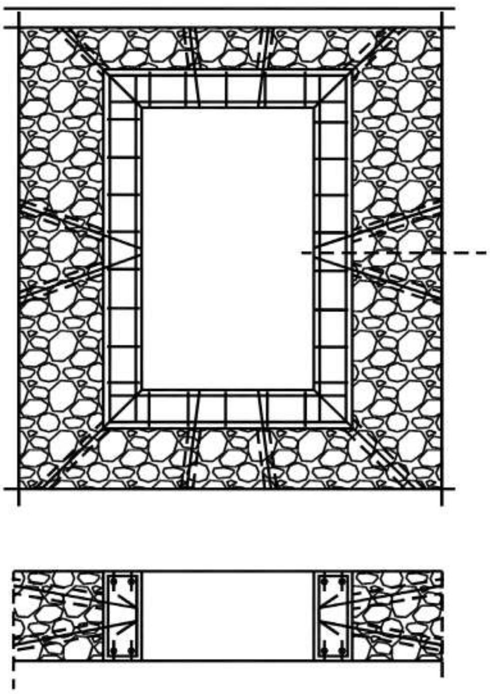


Cross tie elements in the form of tied beam and column or bandage for integration of a masonry house[23]

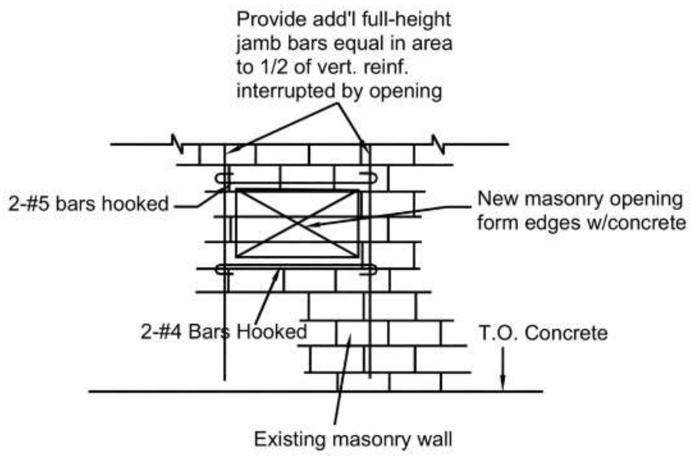
7.12 How to strengthen the large openings of the walls?

The in-plane strength of masonry walls reduced due to openings may be increased by employing an RC or steel frame inside the opening. The weakness of the wall caused by the opening can be effectively counteracted by the frame.

Vertical jamb steel may be placed at the edges of opening, while the horizontal edges of the opening are reinforced with reinforcement bar spanning between the jamb reinforcement. Jamb reinforcement can be inserted from one side and be anchored to foundation pad and slab or lintel band/lintel if available. The horizontal bars may be hooked at the ends or may extend past the openings edging upto an adequate distance.



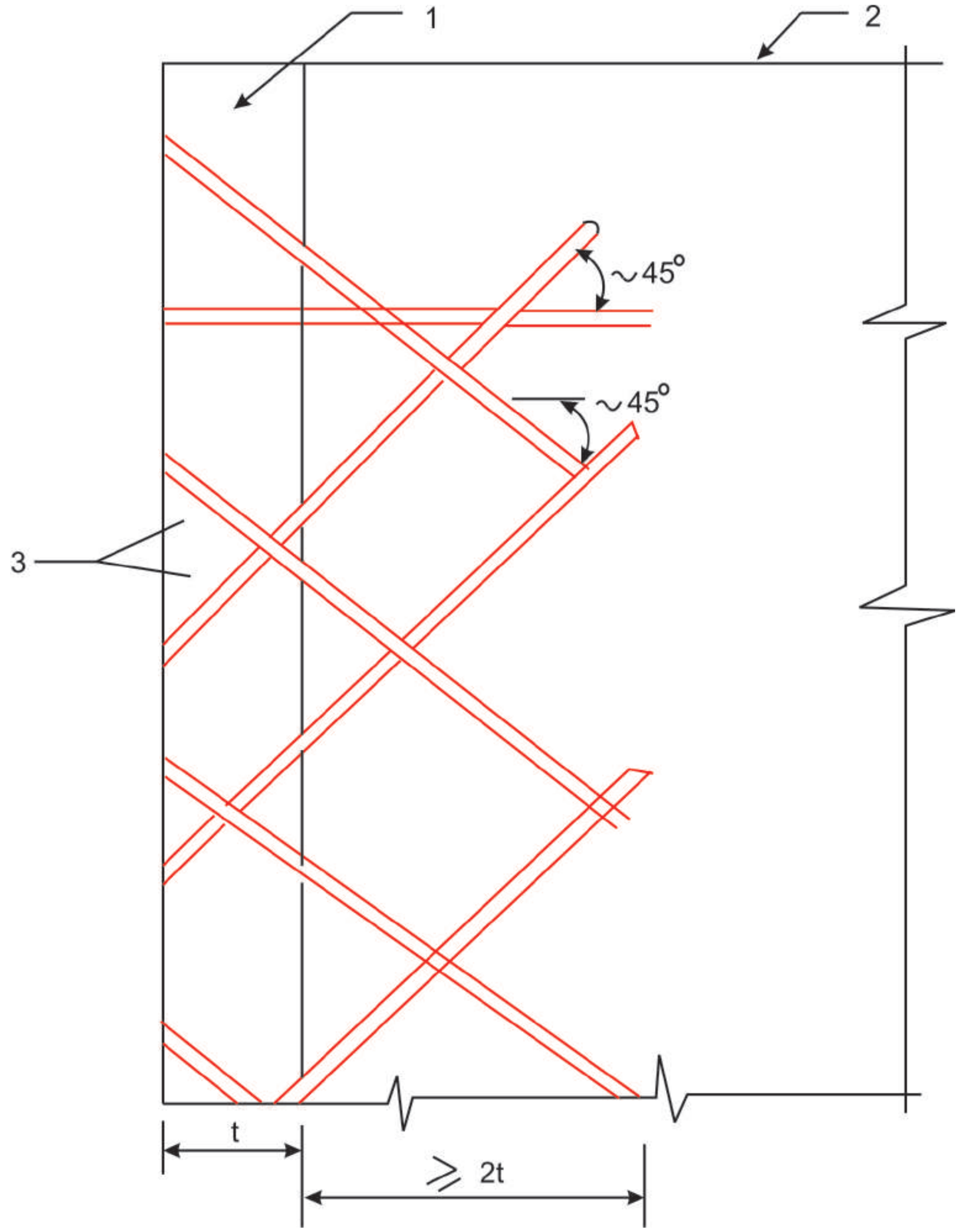
***Use of steel and RCC frame to strengthened the openings*[44]**



***Use of Vertical and horizontal reinforcement may another alternative to strengthen the openings*[27,19]**

7.13 How to improve wall-to-wall connection in a house to resist earthquake forces?

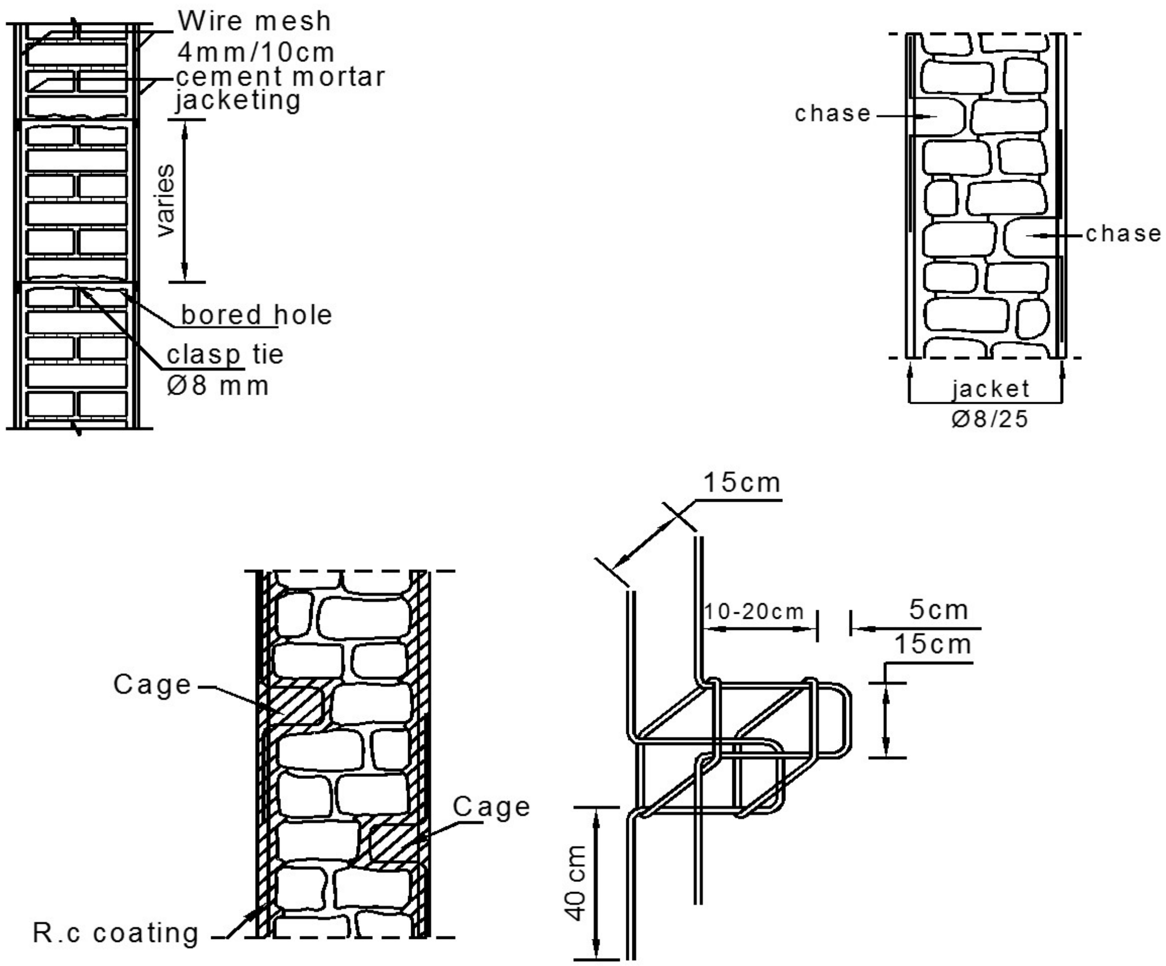
Corners and wall intersection zones are always susceptible to heavy damage during earthquakes. The corner or wall intersection can be strengthened by stitching techniques. Holes are drilled in orthogonal walls of the structure at a regular interval of 0.5m. After cleaning of holes with water, steel rods of about 12mm diameter are inserted into both intersecting walls. The holes are filled with cement grout.



Stitching technique to strengthen the intersection region/corners of a house[13]

7.14 How external jacketing may be useful to integrate all the walls of a house to resist earthquake forces?

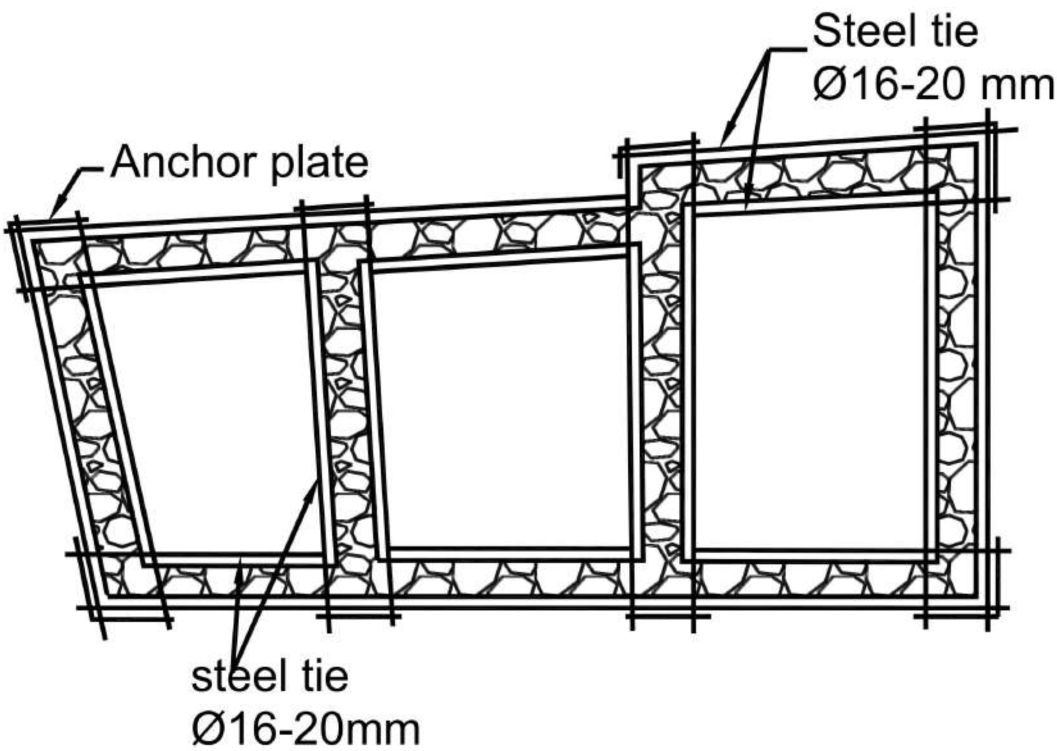
The application of reinforced cement coating forming a jacket on one or both sides of the wall is used to improve the lateral resistance. It is easy to apply and very efficient, hence is widely used. Ferro-cement or wire fabrics like FRP materials are more efficiently used. In retrofitting by jacketing, plaster is first removed from the wall and joints between the bricks are cleaned from the mortar. The joints are filled with cement sand (1:1) mortar. A welded wire mesh or wire fabric is placed around the entire damaged region. Steel ties are inserted at regular intervals of 0.5m to 0.6m in order to tie the mesh with the wall followed by concreting or shotcrete of about 3 to 4 cm for simple brick work upto 8 cm or greater thickness for heavy masonry on the welded mesh. Two-sided jacketing is more effective rather than one sided jacketing. But one-sided jacketing has an advantage as it leaves the exterior facade of the building unaltered.



The possible way to fix the reinforcement mesh or steel cage in existing masonry wall for external jacketing[27]

7.15 How to integrate all the walls of a house to resist earthquake forces by using steel ties?

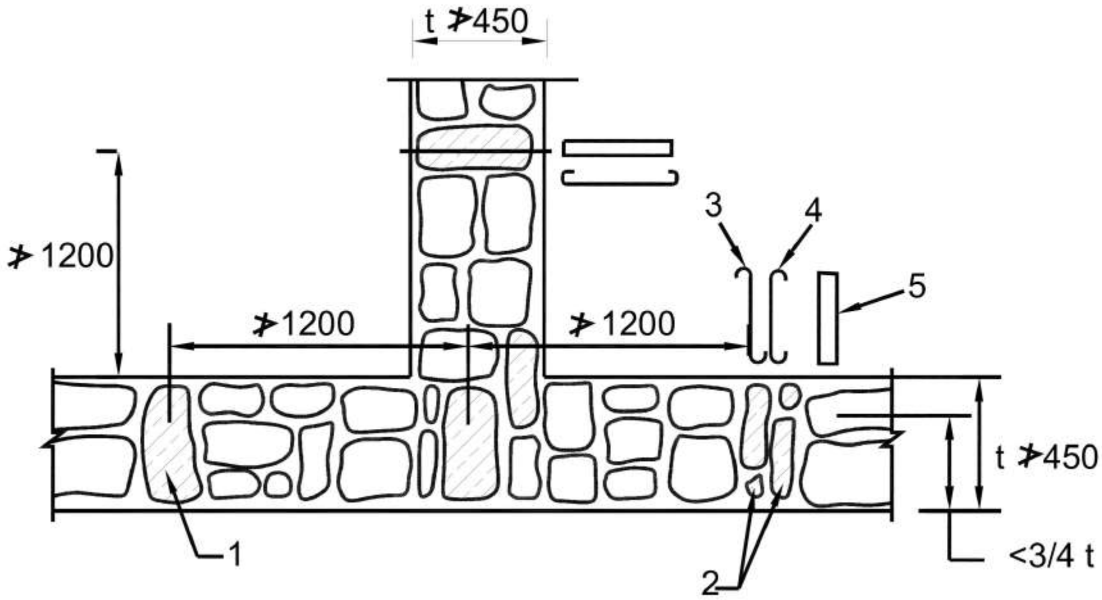
Horizontal reinforced concrete tie beams are another way to tie all the external walls of a house and strongly recommended in a number of design codes. The addition of a continuous tie beam is relatively easy at the top floor. It binds all the external walls together, prevents their separation and act as a horizontal bending beam resisting the floor mass and transfers it to the shear walls.



Integration of masonry walls by using steel ties[27]

7.16 How can the multi-layer walls be integrated to acts as a unit thickness?

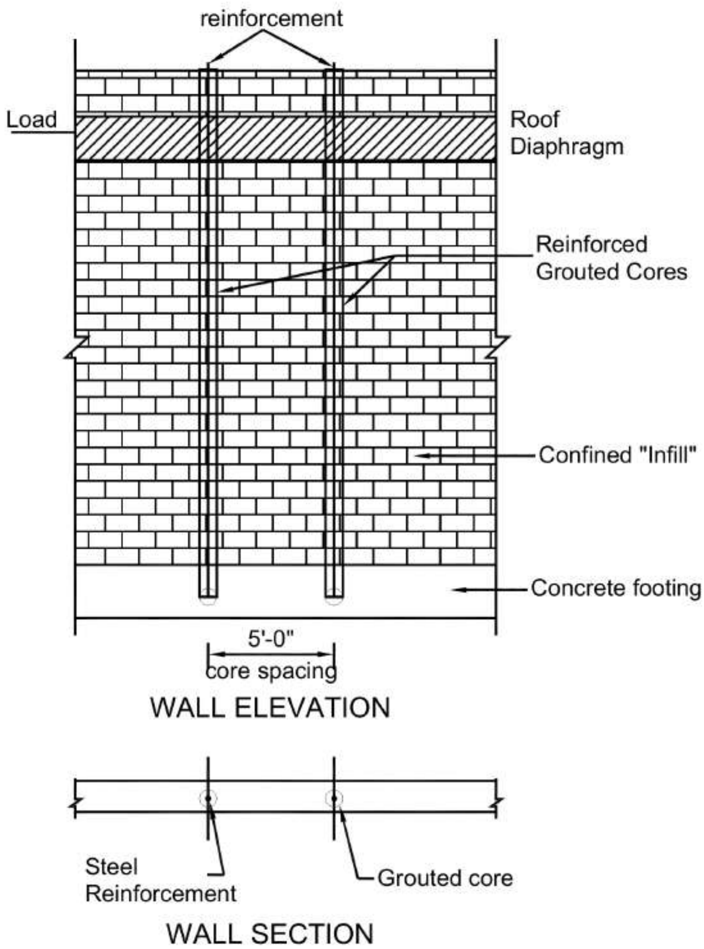
Vertical separations of internal and external wythe through middle of the wall thickness generally occur in stone masonry constructions. Use of "Through" stones of full-length equal to wall thickness may be one solution to integrate both the wythes of walls. These stones may be inserted at an interval of 0.6m in vertical direction and 1.2m in horizontal direction. Other alternatives of through stones are use of "**S**" shape elements of steel bars 8 to 10 Φ or a hooked link with a cover of 25mm from each face of the wall or wooden bars of size 38mm x 38mm cross section or equivalent.



Details for providing through stones in a stone masonry wall[13]

7.17 How to provide vertical elements in the existing system to transfer the earthquake forces efficiently?

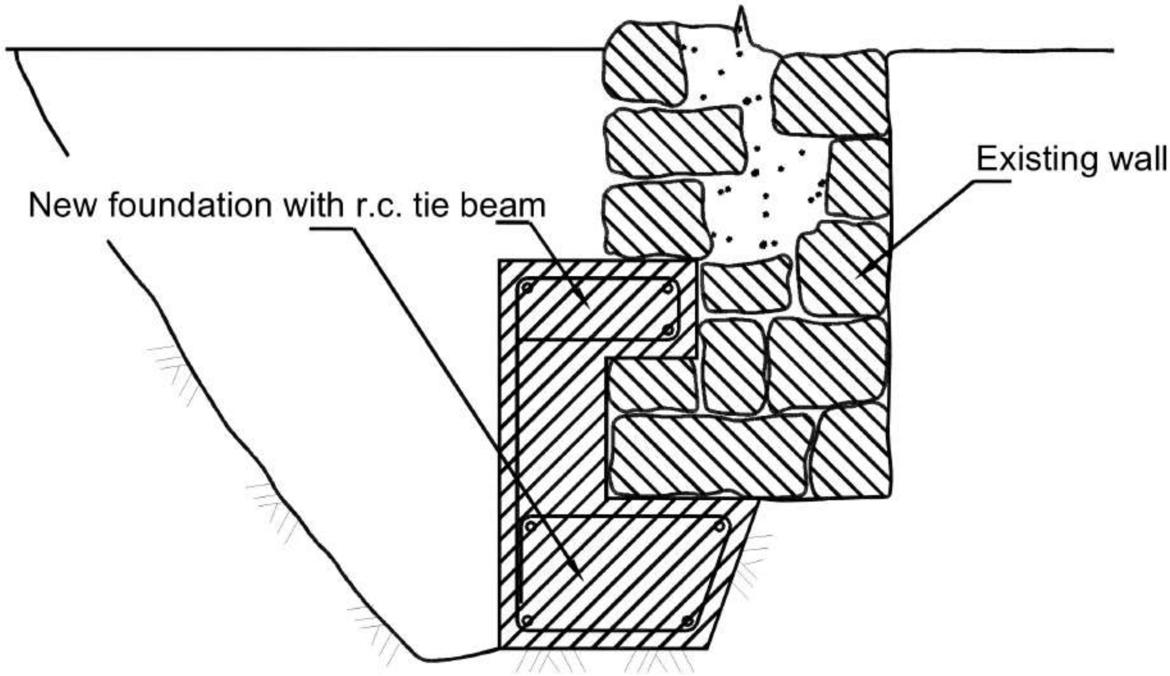
Unreinforced walls have been converted to reinforced system by the introduction of steel reinforcement directly into drilled holes and anchoring using cementitious grouts, polymers –resins or epoxies. A center core technique is the method to provide the vertical reinforcement in the unreinforced walls. It consists of a reinforced, grouted core placed in the center of an existing unreinforced wall. A continuous vertical hole is drilled from top of the wall into its basement wall. A filler material is pumped from the top of the wall to the bottom such that core is filled from the bottom under pressure controlled by the height of grout. Wall anchors for lateral ties to the roof and floor are placed at the core location to make a positive connection to the wall. The system has several advantages as it will not alter the appearance of wall's surface as well as the function of the building will not be impaired since the drilling and reinforcing operation can be done externally from the roof.



***Center core technique for providing vertical reinforcement in a masonry house*[19]**

7.18 How to integrate a house at foundation level?

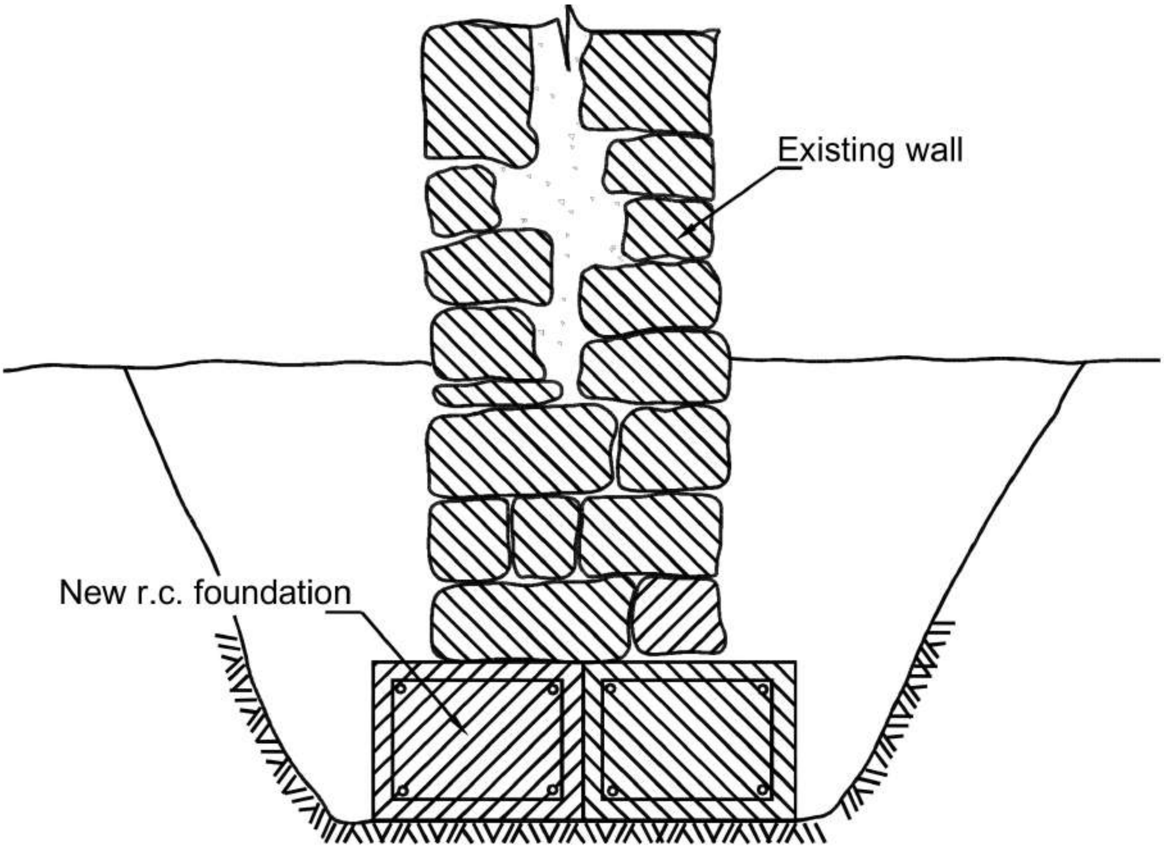
It may be achieved by providing structural ties at the foundation to enable the building behave as a single unit when the ground movement occurs. A reinforced concrete belt is used all around the building at the foundation level or to build a tie beam along the inner side of the foundation.



A process to provide a seismic belt at foundation level to integrate the foundation[27]

7.19 How can the foundation of existing house be strengthened?

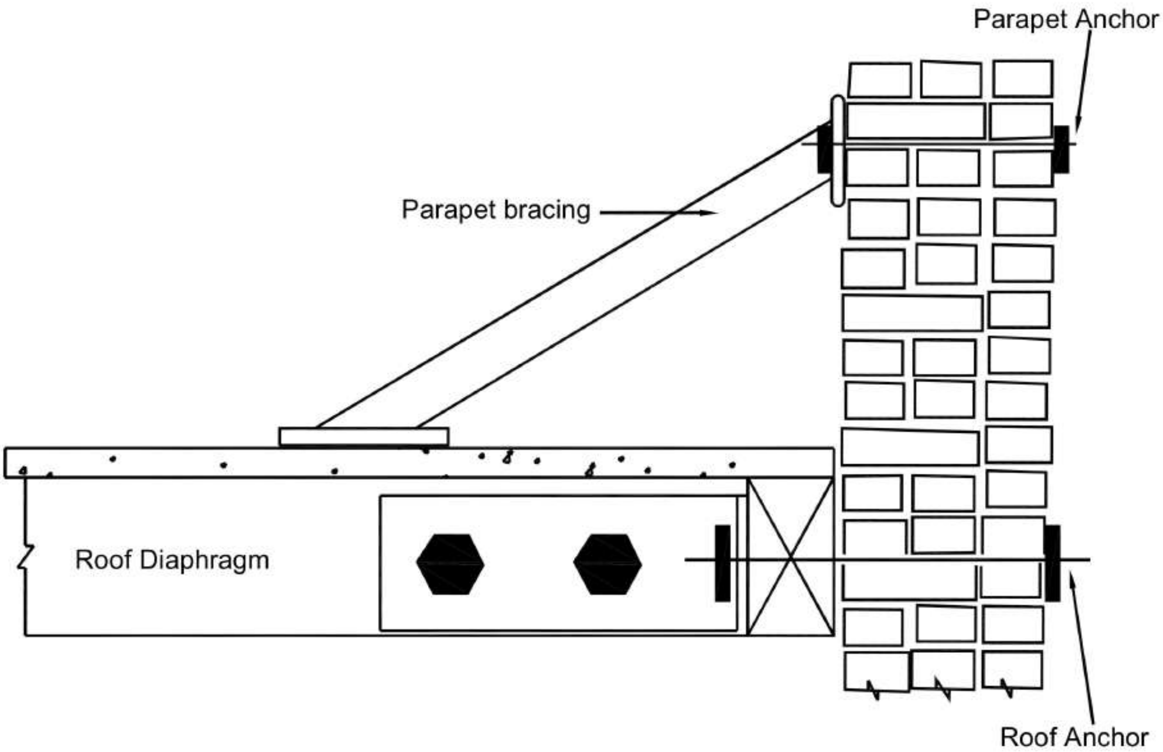
Differential settlement in masonry building can bring about serious damage. This settlement can be spread up through various floors and affect the stability of the entire building. Strengthening of the existing foundation is the only solution which can be done by two methods (i) increase the bearing area when the strengthening or reconstruction increases the building's weight and/or increased overturning forces (ii) Underpinning can also be used if the existing foundation has some internal strength by digging down on both sides of the wall and removing the material underneath the foundation in sections



Strengthening of existing foundation in a masonry house[27]

7.20 How to protect parapet wall and other non-structural elements from earthquake damage?

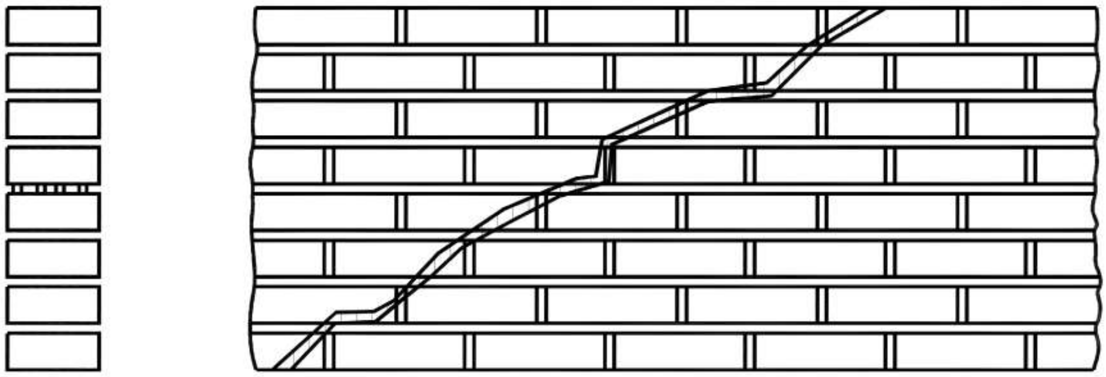
Retrofitting of un-reinforced masonry parapets is a considerably effective method of minimizing hazard. Parapets are more prone to failure if the height to thickness ratio is greater than 2.5. The basic element of seismic retrofitting of vulnerable parapets involves bracing parapets, roofs and connecting floor diaphragms to walls through anchor. Similarly, existing balconies and overhangs are the horizontal non-structural features projecting outward or away from a building. The chances of failure are similar to parapet walls. The solution either reduces the projection length or a suitable connection in accordance with current building codes, providing some external bracing system.



Strengthening of parapet walls by using steel bracing [6]

7.21 How to repair existing cracks in a masonry house?

Grout injection and filling the voids and cracks are the popular techniques to restore the original integrity of the existing wall. In multi-wythe masonry walls, injecting grout enhances composite action between adjacent wythe. For injection, epoxy resin is used for relatively small cracks (less than 2mm wide); while cement based grout is considered more appropriate for filling of larger cracks, and empty collar joints in multi-wythe masonry walls. Horizontal and vertical prestressing may also be effective in strengthening masonry walls as well as repairing widespread cracks because it provides a more consistent distribution of the stresses.

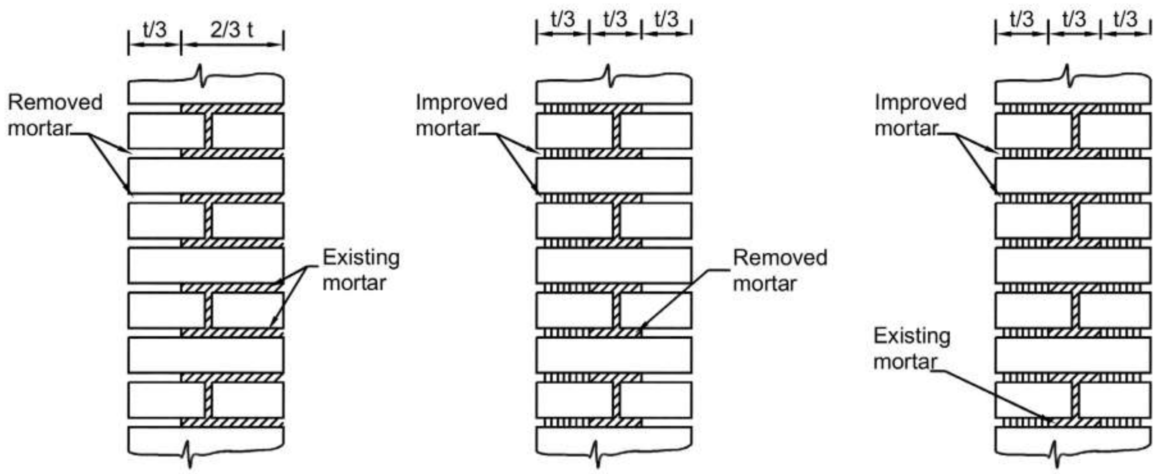


***Repairing of masonry cracks through cement or epoxy injection*[18]**

7.22 How do the quality of existing unit and mortar be improved?

All the building materials deteriorate with time and age. A weak or deteriorated mortar is often the cause of damage to masonry building. Freezing and thawing in cold weather, expansion and contraction from extreme thermal changes can bring about the deterioration. Extreme stress causes mortar to crack. Hence improve the strength of mortar by re-pointing when there is crack in the mortar, loose or missing mortar, weak or crumbly mortar, gaps between the mortar and the masonry, loose bricks or damp spots on the surface of masonry. Repointing is carried out by removing old and deteriorated mortar from between courses of masonry and replacing it with new sound mortar.

At first, the mortar is removed up to a minimum depth of 2- 1/2 times the thickness of the mortar joint, followed by proper cleaning of the surfaces with the help of compressed air brush or stream of water and insertion of new mortar. The joints are then re-pointed with cement mortar. After sufficient strength of the mortar is attained the procedure of re-pointing is repeated on the other side of the wall. Repointing mortar consists of sand (10-12 parts), lime (5-6 parts) or cement (1-2 part white Portland).



A process of re-pointing in masonry building to improve the quality of mortar[23]

Chapter 8

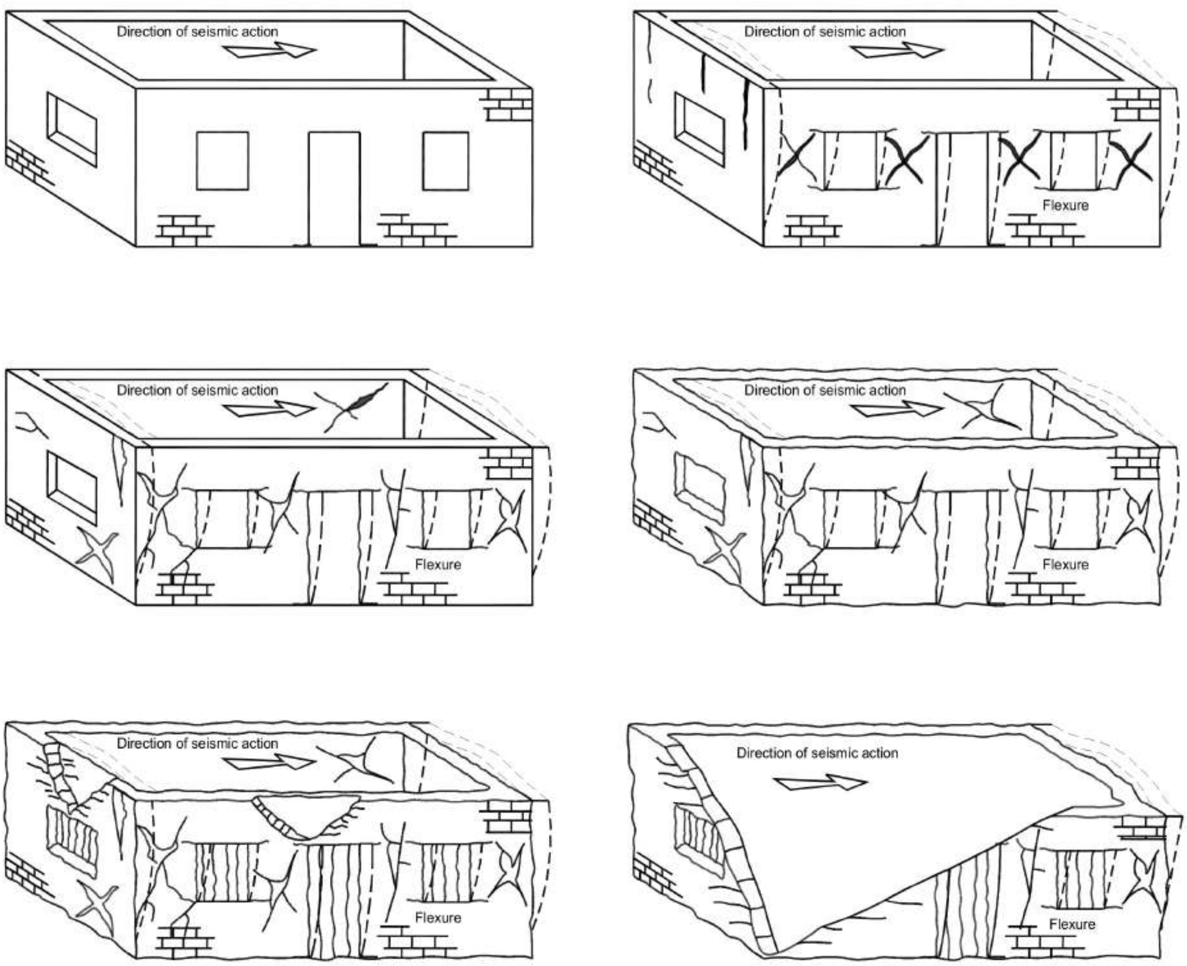
Repair and Retrofitting of a House after the Earthquakes

8.1 How to retrofit a house after damage during earthquake?

A house can be retrofitted effectively and efficiently. The severity of damage is categorized in six levels depending upon the crack width as follows;

Grade	Damage level	Crack width in load bearing masonry walls
G0	Undamaged	No visible damage
G1	Slight damage	Minor cracking (Hairline cracks up to 5mm width)
G2	Moderate damage	Moderate cracking (Cracks 5-20mm)
G3	Heavy	Severe cracking (Cracks 20mm or wall material dislodge)
G4	Partial destruction	Complete collapse of individual wall material or individual roof support
G5	Collapse	More than one wall or more than half of roof collapse

Retrofitting procedure can be implemented maximum up to G3 grade of damage.

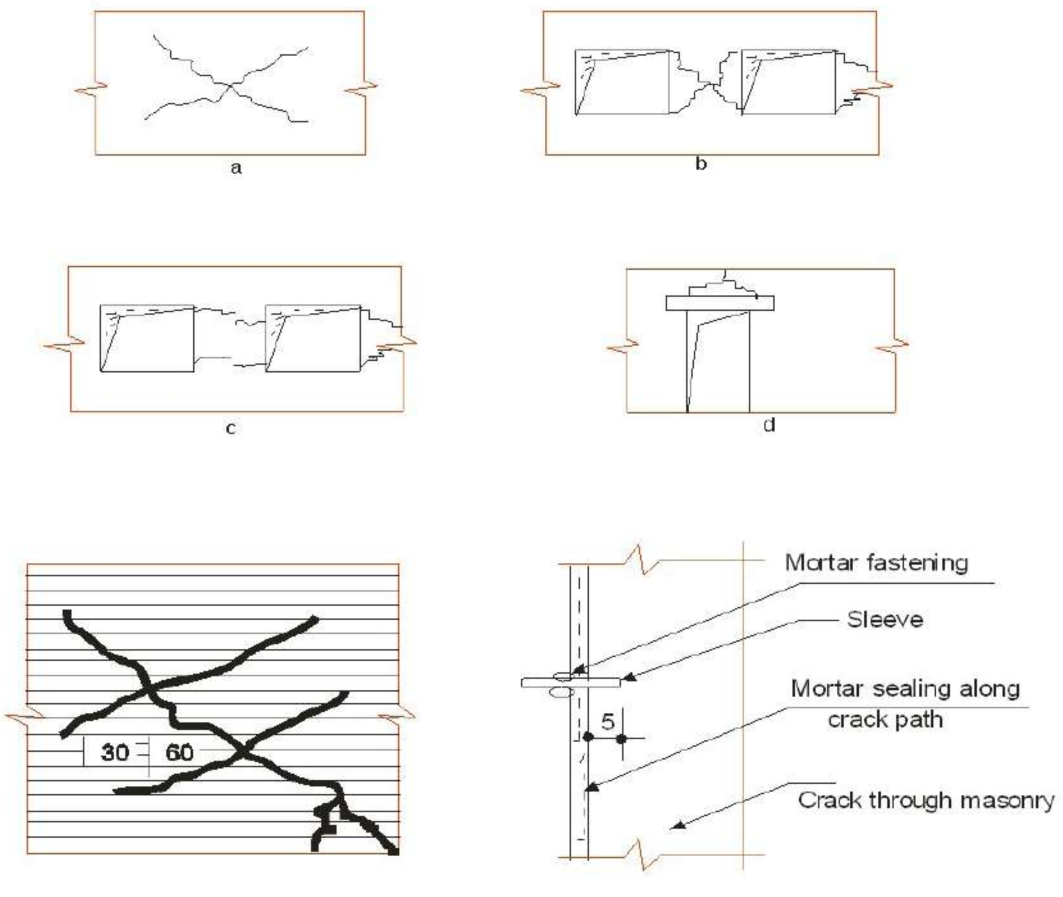


Severity of damage is categorized in six levels

8.2 How to repair minor cracking occurred in a house after earthquake?

In order to repair minor cracks, pressure injection of cement grout containing admixtures against shrinkage or epoxy is recommended. For very fine cracks epoxy injection is preferred. The process of repairing is as follows;

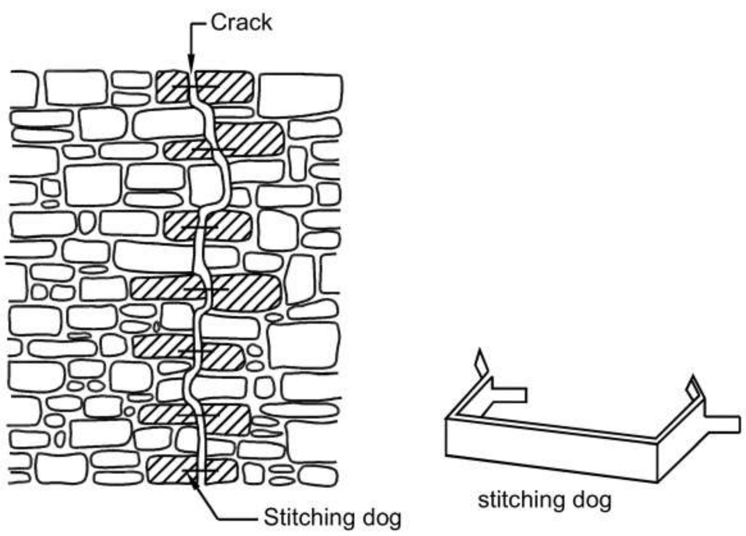
(i) Expose and clean the cracked area by air or water (ii) Drill holes at an interval of 30 to 60cm along the cracks and insert the nipple/sleeves about 5cm deep into the holes and fix with cement mortar (iii) insert stopper in each nipples and cover the cracks with cement mortar, along their whole length (iii) injecting of cement fluid or epoxy either by manual pump or mechanical pump should start from extreme bottom and proceed upwards by removing the stopper from the two successive nipples/sleeves until the injecting material overflows from the upper one (iv) Remove nipples/sleeves from the port and fill with cement mortar (v) Check the strength and compactness by appropriate tests



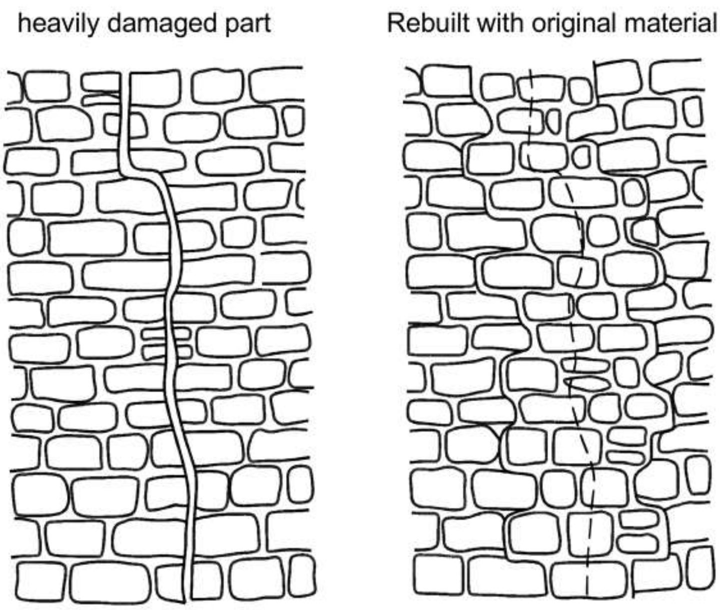
Repairing process for minor cracking

8.3 How to repair moderate vertical cracks occurred in a house after earthquake?

The repairing process of moderate type cracking somewhat depends upon the pattern of cracks. In general two types of cracks have been noticed vertical or inclined /diagonal. Vertical cracks can be repaired by placing stitching dog across the crack along the cracked zone at suitable intervals by removing the wall over about half its thickness as shown below and finally fill the void with concrete or new masonry. Same procedure can be repeated on the opposite side of the wall if necessary. Another alternative is to remove cracked zone and reconstruct it with new units in rich mortar. The procedure for inserting the new units is shown below



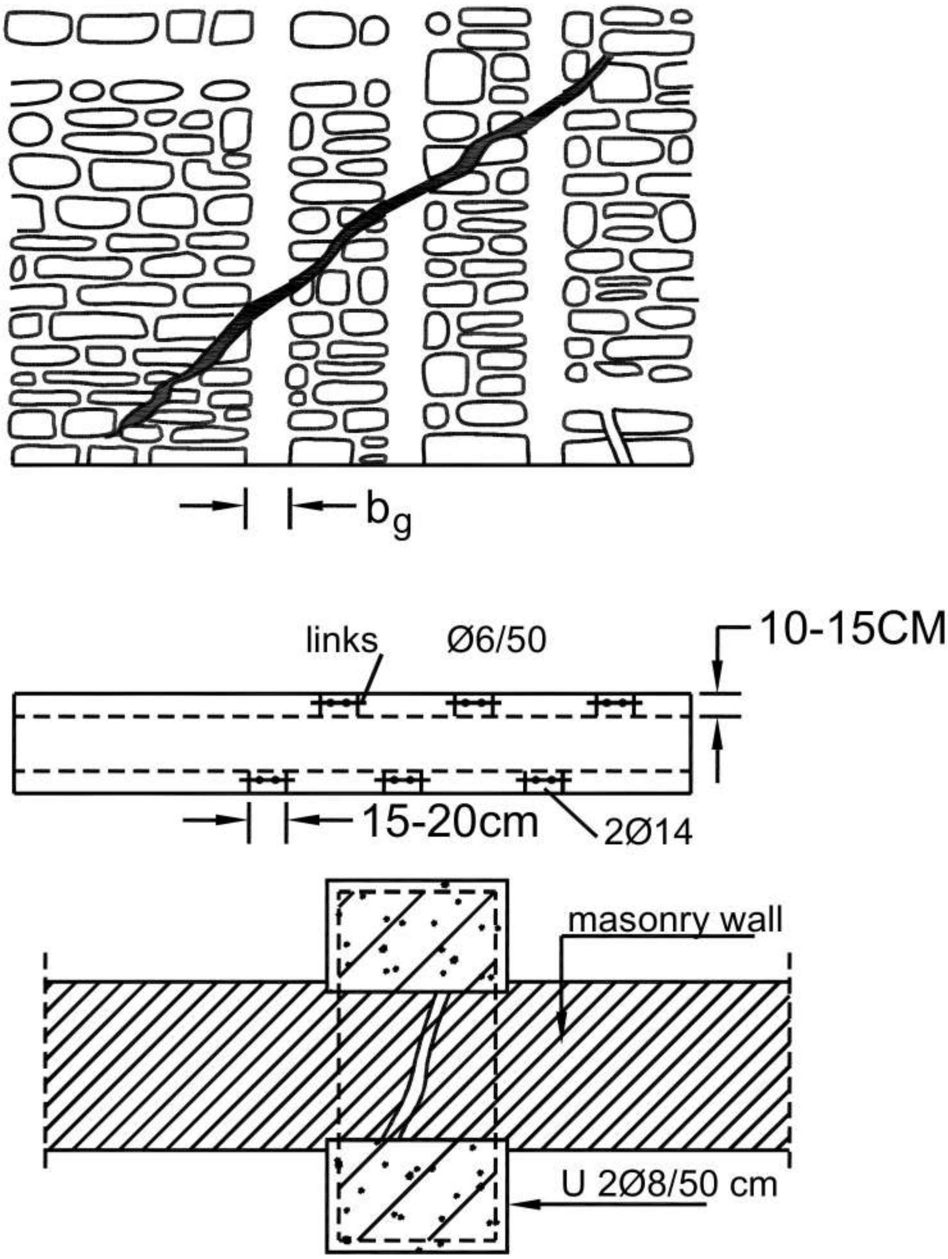
***Repairing process for vertical cracks of moderate size by stitching dog*[27]**



***Repairing process for vertical cracks of moderate size by rebuilding*[27]**

8.4 How to repair moderate diagonal/inclined cracks occurred in a house after earthquake?

Inclined/diagonal cracks can be repaired by insertion of vertically tied reinforced columns by creating a cavity in the cracked wall, removing its units from a vertical zone 15 to 20 cm wide and 10 to 15 cm deep as shown below. The same procedure can be repeated from the other side of the wall. These columns may be placed externally or even internally but should be well connected to the existing wall.



Repairing process for inclined/diagonal cracks[27]

8.5 How to repair Loss of connection among the multi-wythe masonry walls occurred in a house after earthquake?

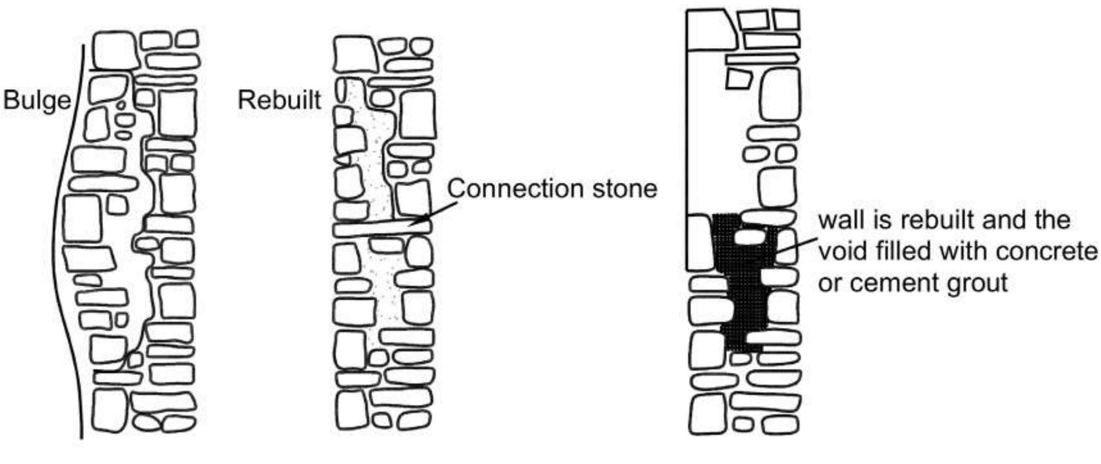
Repair depends upon the exact condition of the house after the earthquake. Here are a few instances providing an exposure to repairing techniques for a severely damaged house during the earthquake.

Use of Through Stones/ Bond Stones

Vertical separation of internal and external wythe through middle of the wall thickness occurs in stone masonry constructions. Reconstruction may be preferred if one of the layers is stable enough to be used as framework. Full length "Through" stones equal to wall thickness may be inserted at an interval of 0.6m in vertical direction at 1.2m in horizontal direction.

Cement Grouting

Grouts are most frequently used to repair and strengthening masonry walls having large voids. Grout injection binds the inner and outer wythes together. The selection of grout depends on the desired strength, bonding properties and on the size of the crack network or void system. A cement grout consisting of 1-part Portland cement, ½ part type S hydrated lime; ½ part type fly ash may be used for repairing earthquake damaged un-reinforced masonry building. if the wall wythes are poorly interconnected, the wall being grouted can split apart and collapse. Low lifting grouting is preferred as it reduces the hydrostatic pressure. Grouting begins from the bottom of the wall and proceeds up to the top.

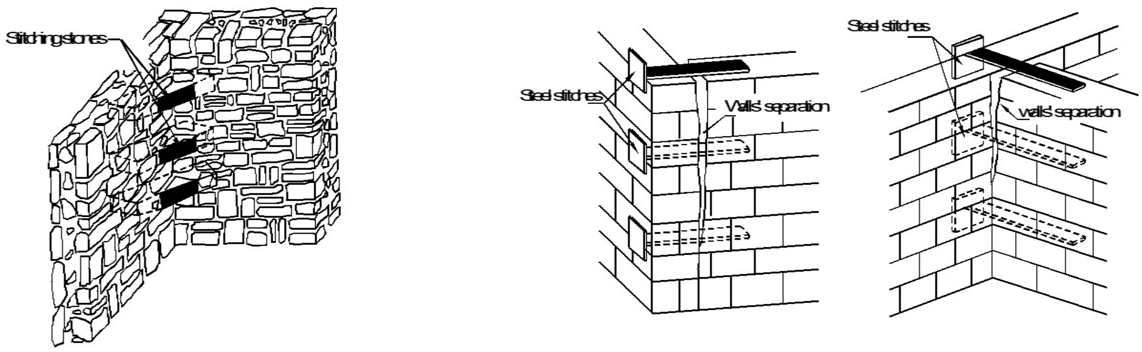


Repairing process of a severely damage multiple wythe wall using through/ bonding/connecting stones or using cement grouting[27]

8.6 How to repair loss of connection between intersecting walls in a damaged house after earthquake?

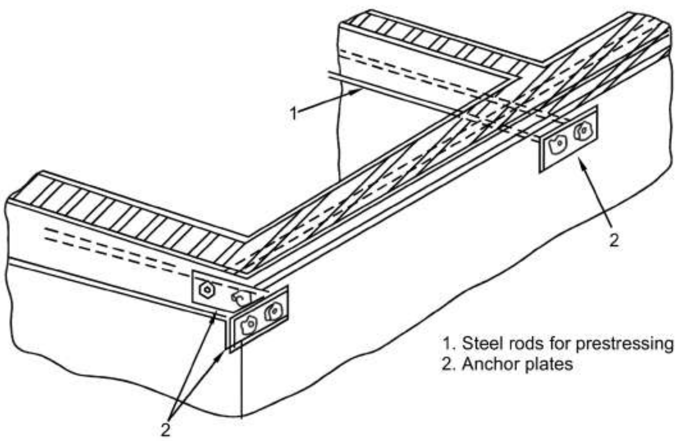
The damaged wall intersection zones may be repaired by sealing the cracks with grouting, and strengthened by stitching. Adjacent units are removed as denoted by “1” and “2” and installing a new unit, denoted by “3”, common to both walls. The new stitching unit should be embedded in rich cement grout, at a spacing of about 60 to 90 cm.

In case of separated wall sections they are tied up with steel plate (i.e. 40 x 4 in cross section), embedded in rich cement grout in between two brick or stone layers after some bricks or stones have been removed. Such plates can be very effective in reinforcing the corner but they cannot bring the walls back to vertical position. The gap is then filled with cement mortar.



Repairing process of wall intersection by stitching stone and by steel plate[27]

Another alternative of steel plate is to drill horizontal holes in the masonry through vertical crack and grouting or epoxing steel rods in the holes. To reduce the separation prior to repair, tie rods, installed on both sides of the walls and tightened by turnbuckles as shown below. Remaining crack should be filled with cement mortar.

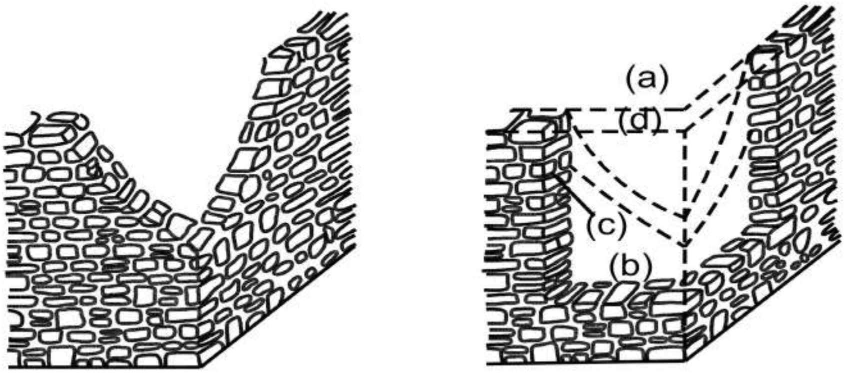


Repairing process of wall intersection by steel rods[18]

8.7 How to repair severe damage at the corner region occurred in a house after earthquake?

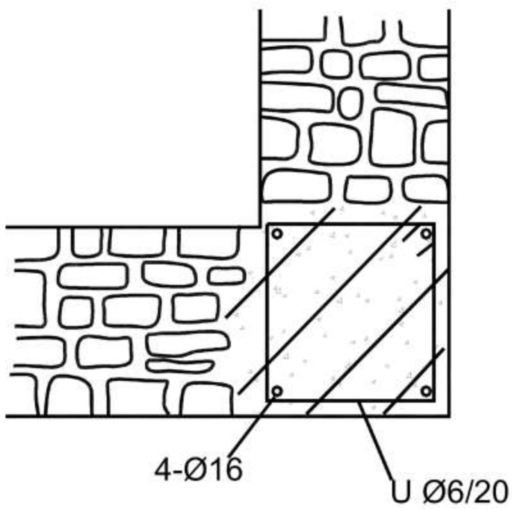
In case of total collapse of corner region, strengthening requires rebuilding of the corner with proper bond of the rebuilt part with the wall. A horizontal belt like a seismic band of thickness 15 to 20 cm, reinforcement 4 @ 16mm and stirrups 6m at 20cm, should be added.

A reinforced concrete corner column properly tied into the intersecting walls could be added to strengthen the wall intersection with minimum reinforcement of 4 @ 16mm and stirrups 6mm at 20 cm.



a - removal or support of part of the roof
b - additional wall removal
c - contact surface preparation and careful rebuilding
d - additional construction of a strengthening bolt, thickness 15 to 20 cm, reinforcement 4 Ø 16 and stirrups Ø 6 / 20.

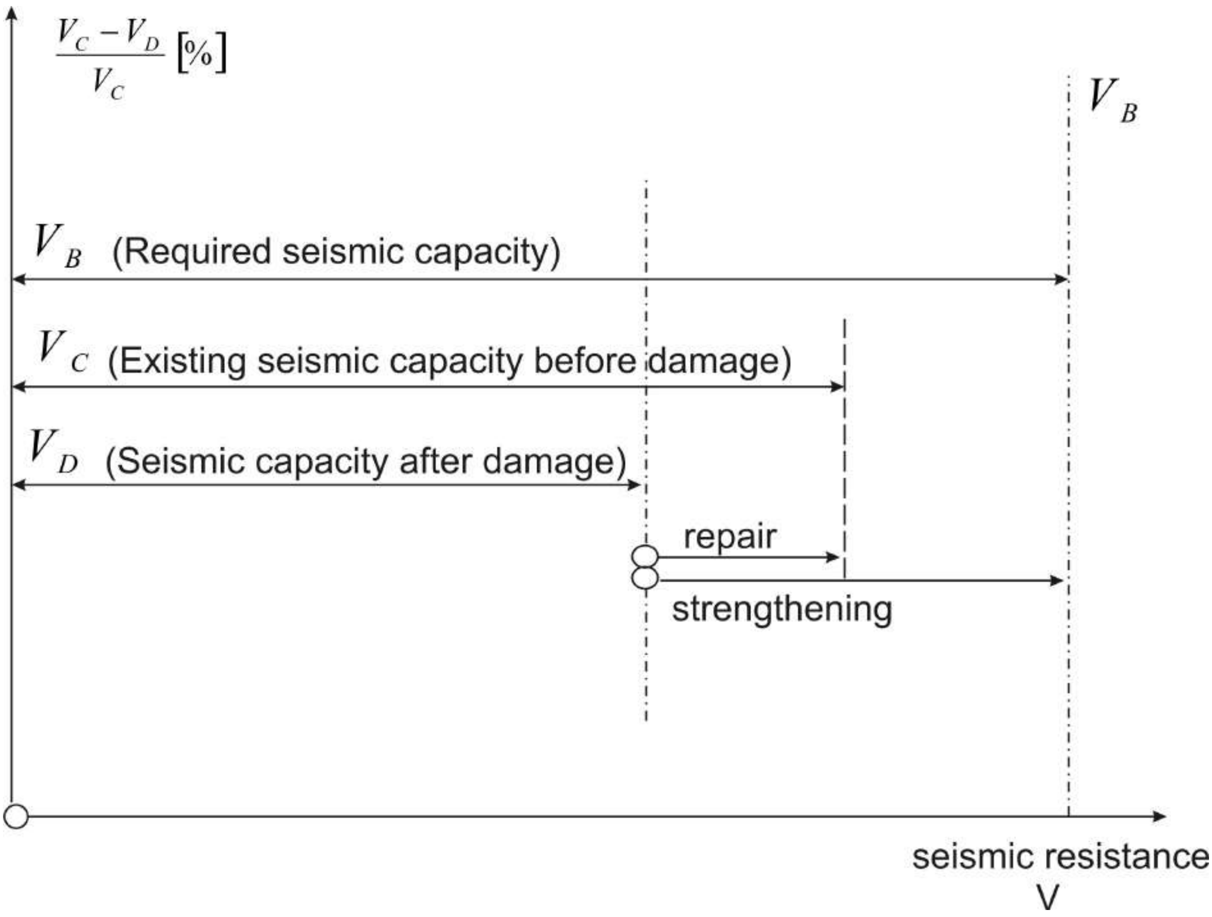
Repairing process for a collapse corner by using horizontal belt[27]



Repairing process for a collapse corner by reinforced concrete column[27]

8.8 Are the repairing techniques of different types of cracking sufficient for a house in case of a future earthquake?

Any repairing process is only helpful to retain at the most original strength of any house. It is not helpful to increase beyond the original strength. Since, a house damages due to insufficient strength; one has to increase its original strength. Therefore, retrofitting techniques should be employed as explained in Part VII to increase either in-plane strength and out-of-plane strength or both in which the house is lacking[28].



Section

II

How

to

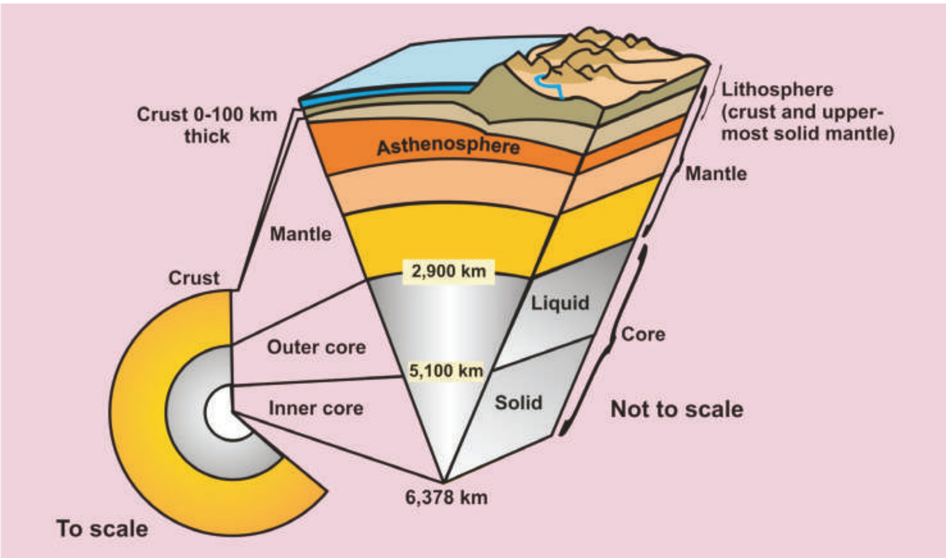
Ensure the Seismic Safety of Multistory Reinforced Concrete Buildings?

Chapter 1

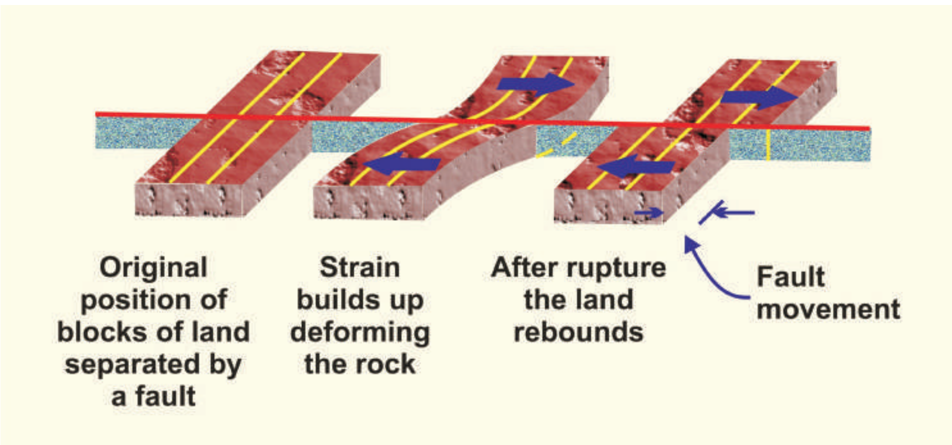
Earthquake and its Effects on Building Response

1.1 What is the earth and how earthquakes occur?

Five billion years ago the Earth was formed by a massive conglomeration of space materials. The heat energy released by this event melted the entire planet, and it is still cooling off today. The earth is divided into three main parts i.e. the crust, the outer most layer (5 to 40 km); the mantle, next layer with a thickness of 2900 km; and the core, the inner most layer with a thickness varying from 2700 km to 6400 km. The crust is cool enough to be tough and elastic, and is known as the lithosphere or rock sphere. It consists of a number of hard tectonic plates, sitting on relatively soft and plastic layer called the asthenosphere. They move as rigid bodies and interact with each other in different waves (known as plate tectonic theory). The occurrence of earthquake in different regions of the world is best explained by plate tectonic theory. This interaction between plates produces significant strain in the crust. When this strain exceeds the strength of the rock, fracture occurs and releases great energy in the form of seismic waves that travel through the earth's crust and cause shaking that we feel during an earthquake. The crust is much thinner, cold, rocky and brittle, so it can fracture in earthquakes. The plane of earth's crust from where this energy releases is called fault which may be horizontal or vertical or some arbitrary angle in between ranging from a few millimeters to thousands of kilometers. This phenomenon is called elastic rebound theory.



Earth's interior

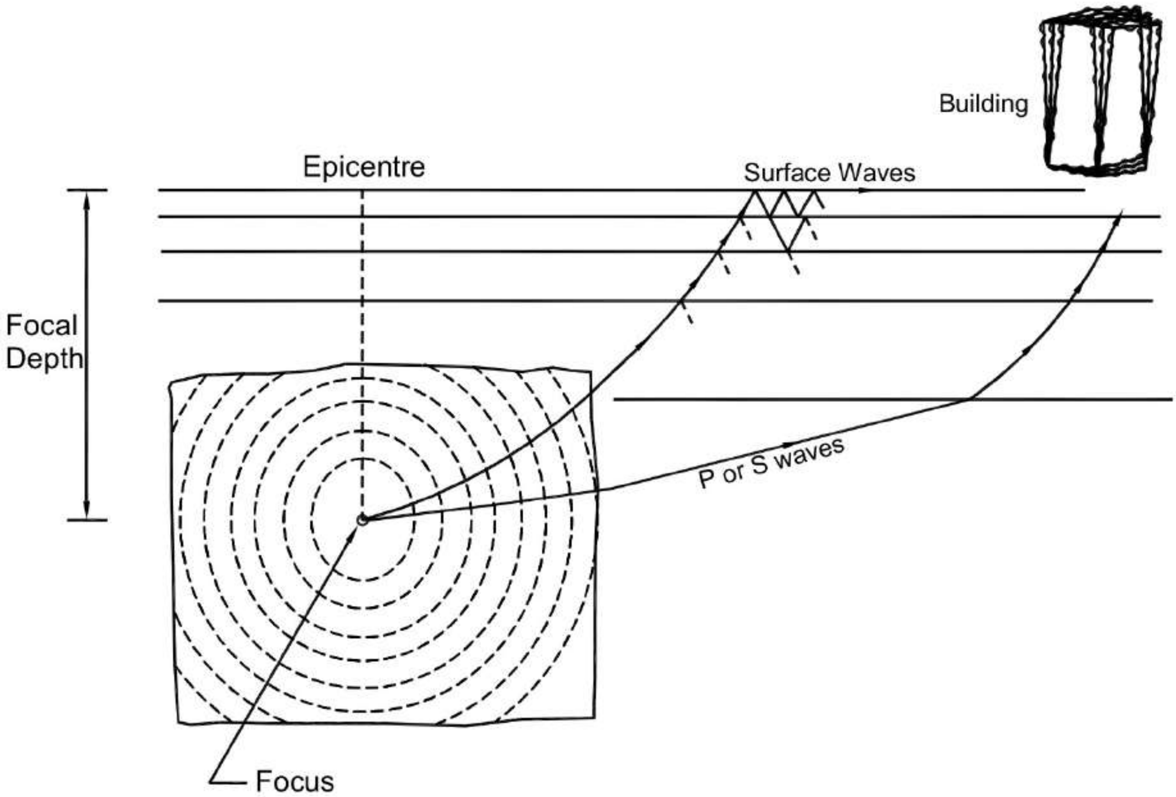


Elastic rebound theory

1.2 Why the ground and the buildings shake during earthquake?

When a fault ruptures, seismic waves are propagated in all directions. As a result the ground vibrates at frequencies ranging from about 0.1 to 30 Hertz causing building to vibrate. Ground shaking is a term which describes the vibration of the ground during an earthquake. Ground shaking is explained in terms of body waves and surface waves. Body waves are compressional, or P (first to cause vibration of a building) and shear, or S (arriving next and causing building to vibrate from side to side and most damaging). P and S waves mainly cause high-frequency (greater than 1 Hertz) vibrations which are more efficient in causing low buildings to vibrate. Rayleigh and Love are surface waves causing low-frequency vibrations which are more efficient in causing tall buildings to vibrate. Body and surface waves cause the ground to vibrate, and consequently a building, to vibrate. Damage takes place if the building cannot withstand these vibrations. The objective of earthquake-resistant design is to construct a building that can withstand the ground shaking.

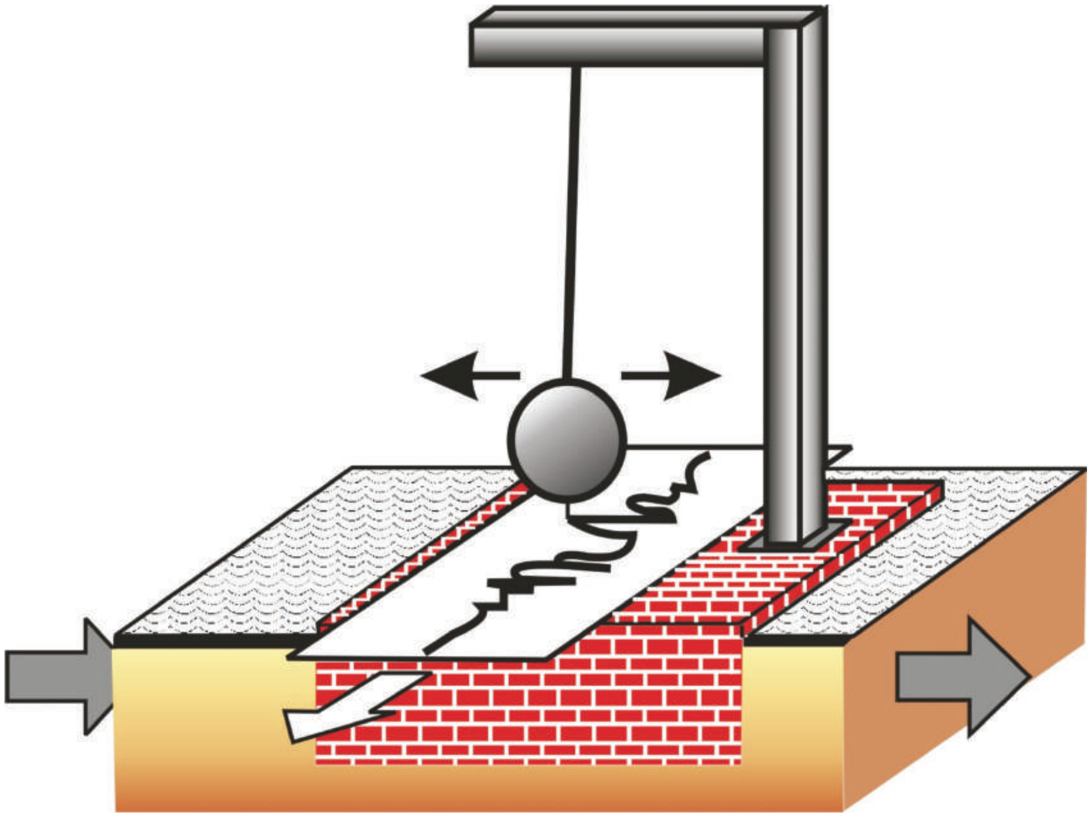
The position of the fault where an earthquake occurs is called focus or hypocenter, of the earthquake and the point on the earth surface directly above the focus is called epicenter. The velocity difference between P-waves and s-waves can be used to locate the epicenter and focus of an earthquake. The time interval between the arrival of a P-wave and S-wave, at a seismograph station is called duration of preliminary tremors.



Cause of vibration of building on earth

1.3 How earthquakes are recorded and located?

Seismographs are the principal instrument to record the earthquakes, which consists of a simple pendulum. When the ground shakes, the base and frame of the instrument move with it, but inertia keeps the pendulum bob in place. As it moves, it records the pendulum displacements as they change with time, tracing out a record is called a seismogram. One seismograph station, having three different pendulums sensitive to the north-south, east-west, and vertical motions of the ground, records seismograms that allow scientists to estimate the distance, direction, magnitude, and type of faulting of the earthquake. Seismologists use networks of seismograph stations to determine the location of an earthquake.



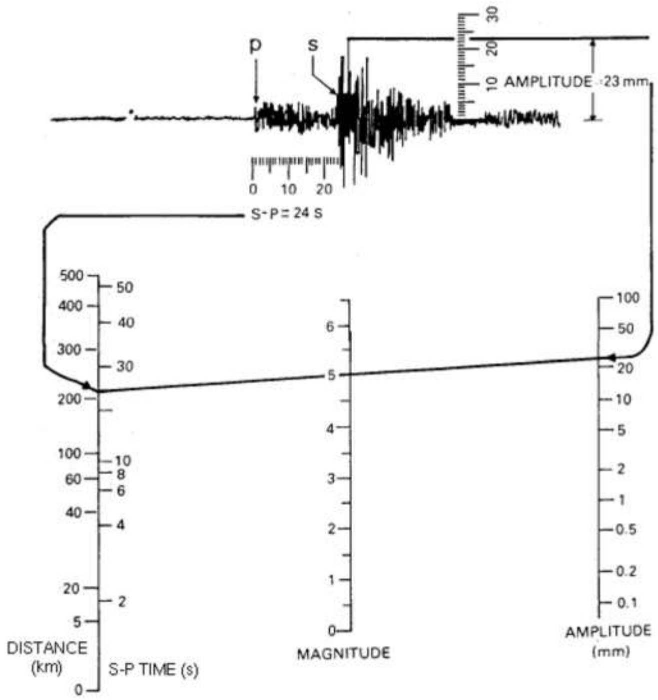
Seismograph to record the earthquake signature

1.4 How earthquakes are measured?

Richter scale is well known to measure the size of an earthquake. It was first developed by Charles Richter in the 1930's referred to as M_L, where L stands for local. This was known as the Richter magnitude. The method developed by Richter was strictly valid only for certain frequencies and distance ranges. In addition, new magnitude scales as an extension of Richter's original idea were developed like body-wave magnitude, **Mb**, and surface-wave magnitude; **Ms** which is valid for a particular frequency range and type of seismic signal. Besides M_L, mb, and Ms, a new, more uniformly applicable extension of the magnitude scale, is known as moment magnitude, or **Mw**, was developed. On the basis of magnitude, earthquakes are classified as: Great; M > =8, Major; 7 < =M < 7.9, Strong; 6 < = M < 6.9, Moderate: 5 < =M < 5.9, Light: 4 < =M < 4.9, Minor: 3 < =M < 3.9, Micro: M < 3. The typical effects of earthquakes in various magnitudes range as follows:

Richter Magnitudes	Earthquake Effects
Less than 3.5	Generally not felt, but recorded.
3.5-5.4	Often felt, but rarely causes damage.
Under 6.0	At the most slight damage to well-designed buildings. Can cause major damage to poorly constructed buildings
6.1-6.9	Can be destructive in areas up to about 100 kilometers across where people live.
7.0-7.9 8 or greater	Can cause serious damage over larger areas.

Although each earthquake has a unique Magnitude, its effects will vary greatly according to distance, ground conditions, construction standards, and other factors. The approximate average occurrences of different magnitude of earthquake annually in world-wide are as follows: Great, 1; Major 18; Strong 120, Moderate 800, Light 6,200, Minor 49000 and Micro 9000 per day.



Measurement of an earthquake

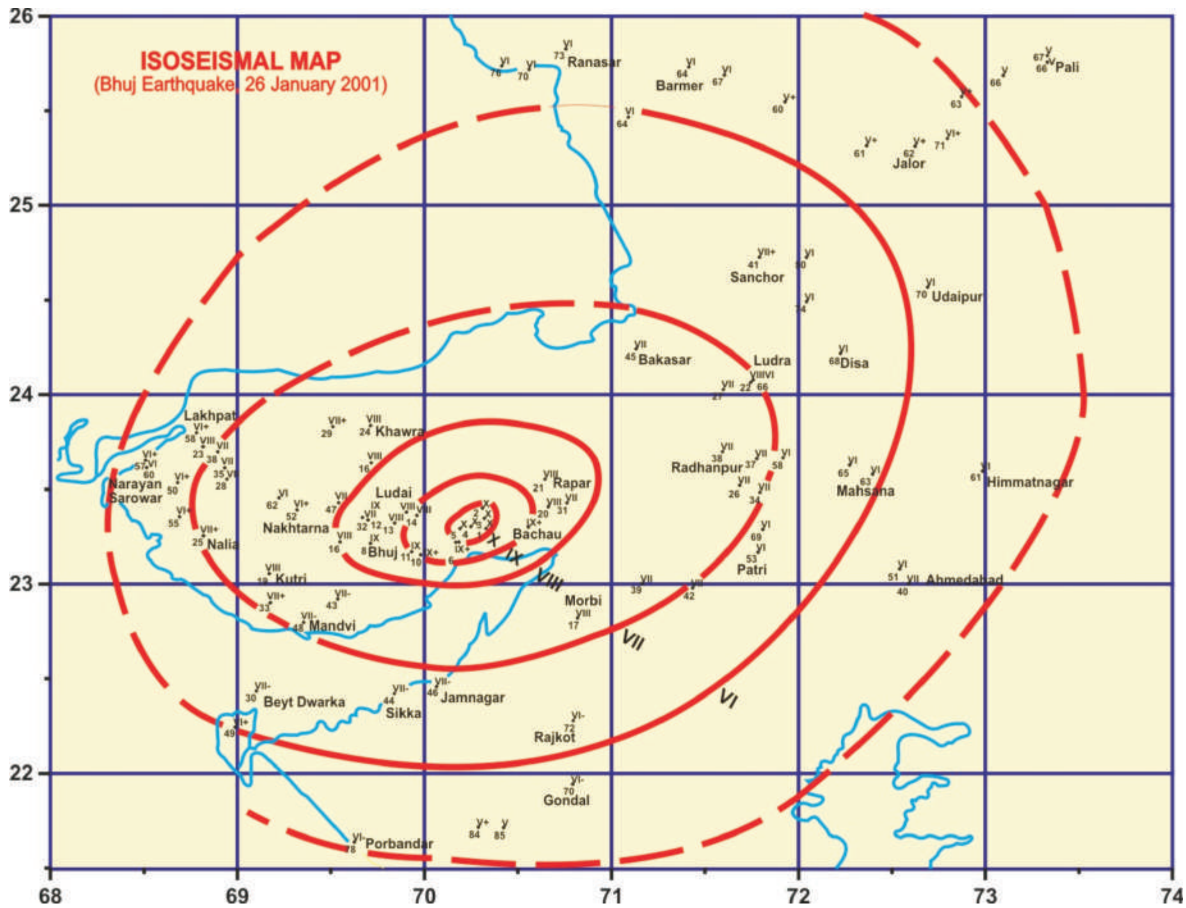
1.5 What is intensity and how is it different from magnitude?

Intensity scales, like the Modified Mercalli Scale and the Rossi-Forel scale, measure the amount of shaking at a particular location. The abbreviated version of Modified Mercalli Scale is presented below which is based on observable earthquake damage. For example a level I to V on the Modified Mercalli Scale would represent a small amount of observable damage. At this level doors would rattle dishes would break and weak or poor plaster would crack. As the level rises towards the larger numbers, the amount of damage increases considerably. The highest number is XII, representing total damage.

Attempts have been made to correlate earthquake magnitude (Richter) and Modified Mercalli Intensity (MMI). The MMI intensity corresponds to magnitude (Richter) 2, 3, 4,5,6,7 and 8 are in the range of I-II, III- IV, V, VI-VII, VII-VIII, IX – X and X respectively.

An empirical relationship between Richter magnitude M, Modified Mercali Intensity MM and focal distance d (in kilometers) is expressed as

$$MM = 8.16 + 1.45\,M - 2.46 \ln (d)$$



Intensity map at a site

1.6 Does every earthquake create disaster?

Earthquakes release a tremendous amount of energy causing destruction. The magnitude can be related to the amount of energy that is released by an earthquake. The relationship between magnitude and energy is:

$\text{Log } E_S = 11.4 + 1.5M$

Giving the energy E_S in ergs from the magnitude M.

Let's take a look at the seismic wave energy yield. For this let us use a larger unit of energy, the seismic energy yield of quantities of the explosive TNT (We assume one ounce of TNT exploded below ground yields 640 million ergs of seismic wave energy). The table below shows magnitudes with the approximate amount of TNT needed to release the same amount of energy.

Richter Magnitude	**Approximate Energy Yield (10^{20} ergs)**
1.0	
2.0	0.0000025
3.0	0.0000725
4.0	0.025
5.0	0.08
6.0	2.5
6.5	14.1
7.0	80
7.5	446
8.0	2500
8.4	10000
8.6	20000

Damage may not usually occur until the earthquake magnitude reaches somewhere above 4 or 5.

1.7 Where do earthquakes occur in the World?

Earthquakes can strike at any location at any time. But history shows that they occur in similar patterns year after year, principally in three large zones of the earth. The world's greatest earthquake belt i.e. circum-Pacific seismic belt, is found along the rim of the Pacific Ocean, with 81 percent of the world's largest earthquakes. It extends from Chile, northward along the South American coast through Central America, Mexico, the West Coast of the United States, and the southern part of Alaska, through the Aleutian Islands to Japan, the Philippine Islands, New Guinea, the island groups of the Southwest Pacific, and to New Zealand.

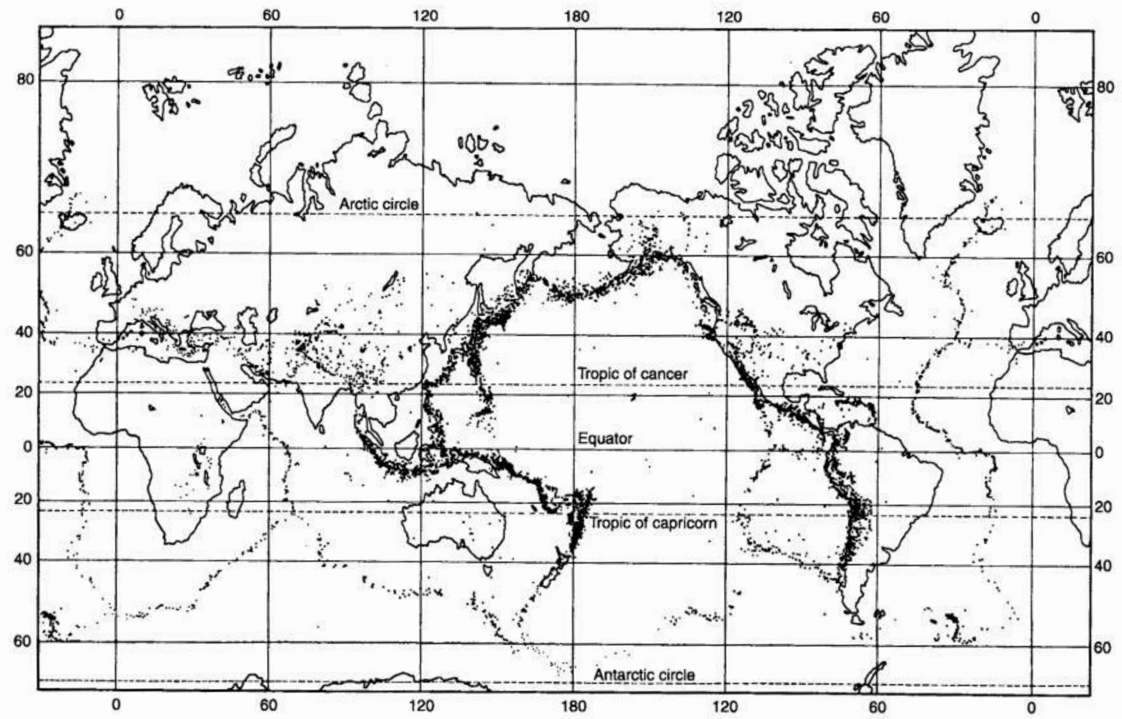


Figure 2.1 Seismicity map of the world. The dots indicate the distribution of seismic events in the mid-twentieth century (after Barazangi and Dorman (1969))

Seismicity map of the word [14]

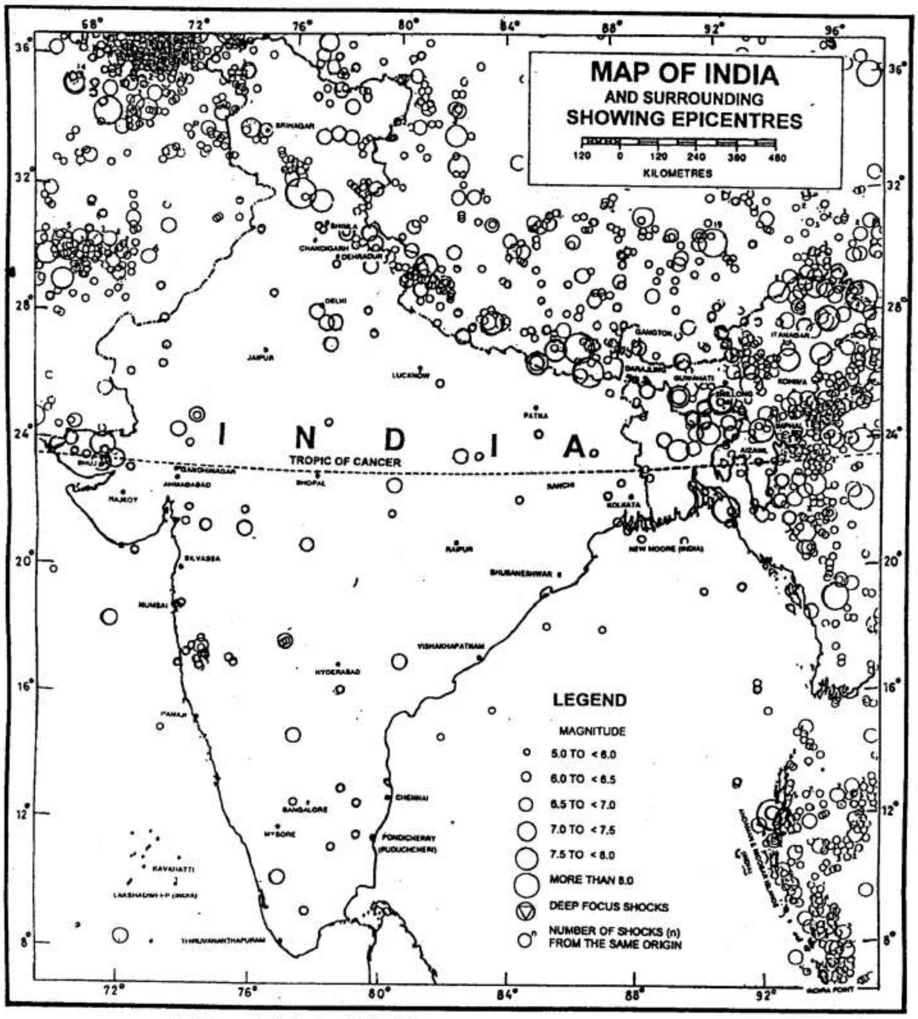
1.8 Where do earthquakes occur in India?

In India, Himalayan-Nagalushai region, Indo-Gangatic plain, Western India, Kutch and Katiawar regions are geologically unstable parts of the country, and some devastating earthquakes of the World have already occurred. A major part of the peninsular India has also been visited by strong earthquakes, but these are relatively few in number occurring at much larger time intervals with considerably lesser intensity. The figure below shows the epicenters of earthquakes in India and neighboring areas which emphasizes the gravity of the problem in the country. Studies of Indian earthquakes have been carried out through Seismic Zoning Map which classifies the area of the country into a number of zones.

Moreover there is no evidence that earthquakes are becoming more frequent. The number of larger events remains only few at larger time interval.

IS 1893 (Part 1) : 2002

ANNEX A
(*Foreword*)



© Government of India, Copyright Year 2001.

Based upon Survey of India map with the permission of the Surveyor General of India.

The responsibility for the correctness of internal details rests with the publisher.

The territorial waters of India extend into the sea to distance of twelve nautical miles measured from the appropriate base line.

The administrative headquarters of Chandigarh, Haryana and Punjab are at Chandigarh.

The interstate boundaries between Arunachal Pradesh, Assam and Meghalaya shown on this map are as interpreted from the North-Eastern Areas (Reorganization) Act, 1971, but have yet to be verified.

The external boundaries and coastlines of India agree with the Record/Master Copy certified by Survey of India.

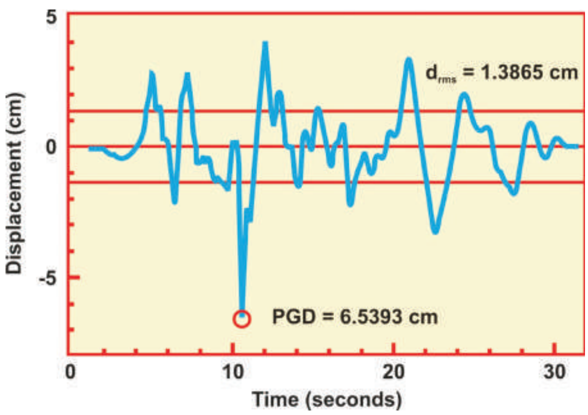
Occurrence of earthquake in India[14]

1.9 What are strong earthquakes and their main characteristics?

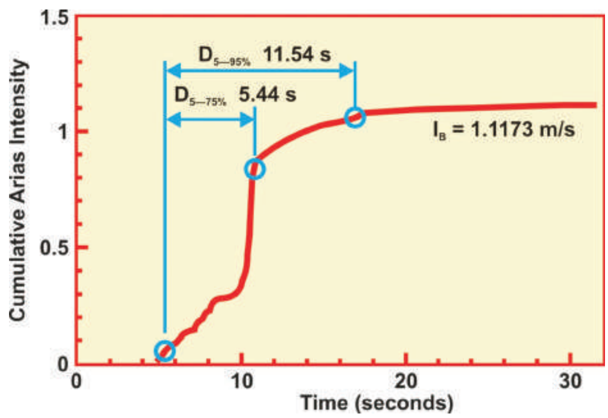
Strong ground motion is characterized by severity of the motion that can cause damage to structures. It is recorded in the form of acceleration vs time for the purpose of earthquake engineering with the help of strong motion accelerographs placed at various locations. In fact there is a network of instruments placed in a given seismic region. It is equipped with a triggering device that initiates recording when the ground acceleration exceeds a threshold value. The acceleration record of a strong earthquake consists of two horizontal components and one vertical component. The vertical component is about 2/3 of horizontal component of motion.

The characteristics of earthquake ground motions are the duration, amplitude (displacement, velocity and acceleration) and frequency of the ground motion. The duration of the shaking depends on the earthquake's magnitude, distance from the epicenter, and the tectonics of the region. Shaking at a site of soft soil, can last 3 times longer than shaking at a stable bedrock. Acceleration is the rate of change in velocity of the ground shaking. Velocity is the measurement of the speed of the ground motion. Displacement is the measurement of the actual changing location of the ground due to shaking. All the three values can be measured continuously during an earthquake. Frequency is defined as the number of complete cycles of vibration made by the wave per second. It is measured in units called Hertz. The ground shaking produced by an earthquake is actually very complex. Strong motions are described by the peak velocity; peak acceleration; frequency and duration.

The complexity of earthquake ground motion is due to three factors: 1) The seismic waves generated at the time of earthquake fault movement are not all of a uniform character; 2) As waves pass through the earth on their way from the fault to the building site, they are modified by the soil and rock **media** through which they pass; 3) Once the seismic waves reach the building site they undergo further modifications, which are dependent upon the characteristics of the ground and soil beneath the building. The three factors referred above are called as source effects, path effects, and local site effects.



Strong earthquake[43]



Duration[43]

1.10 How do strong earthquakes affect buildings?

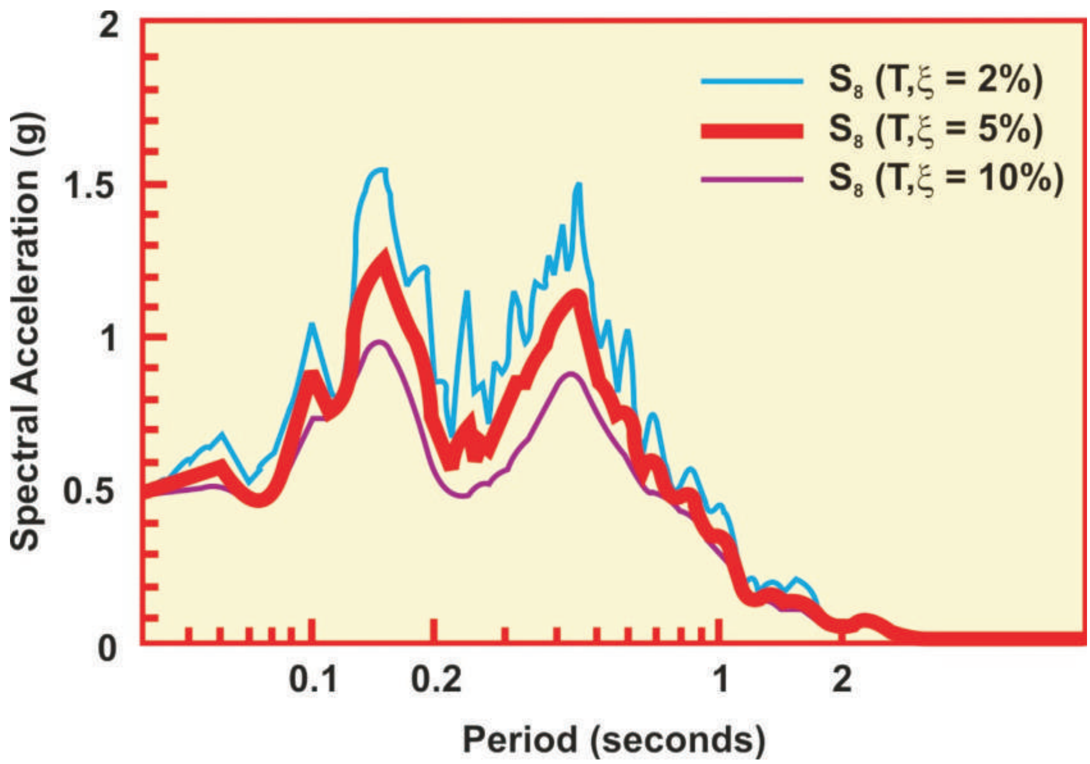
When an earthquake occurs beneath a building, it sets the building in motion, starting with the building's foundation; it transfers the motion throughout the rest of the building and it begins to vibrate in a complex manner that possesses frequency content. The building's vibrations tend to the center around one particular frequency, which is known as its natural or fundamental frequency and is simply inverse to the natural period. The relationship between frequency f and period T can be easily expressed as;

$T = 1 / f$

An approximate time period of regular type of framed buildings can be expresses as;

$T = 0.1\ n$, where n is the number of stories.

It means, shorter a building is, the higher its natural frequency. The taller the building is, lower its natural frequency. The severity of damage depends upon the closeness of frequencies of both the vibrating systems i.e. ground and building. When the frequency contents of the ground motion are centered around the building's natural frequency, we say that the building and the ground motion are in state of quasi-resonance. Quasi resonance tends to increase or amplify the building's response and produce greater damage. Resonance does not occur in earthquake because earthquake motion is not harmonic.



Amplification in structures[43]

1.11 What are characteristics of the building affecting damage during earthquake?

Frequency and Period

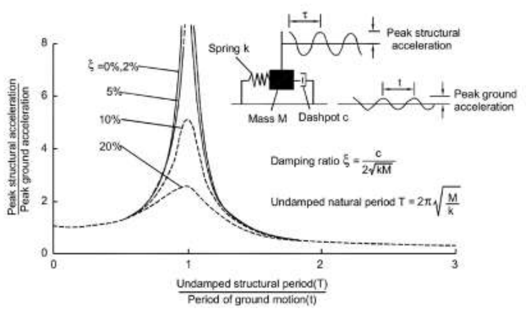
The damage in a building depends primarily upon the frequencies of the input ground motion and the building's natural frequency. When these are close or equal to one another, the building's response reaches a peak level which is called quasi-resonance. In some cases, this dynamic amplification effect can increase the building acceleration to a value two times or more than that of the ground acceleration at the base of the building. Generally, buildings with higher natural frequencies, and a short natural period, tend to suffer higher accelerations but smaller displacement. In the case of buildings with lower natural frequencies, and a long natural period, this is reversed: the buildings will experience lower accelerations but larger displacements.

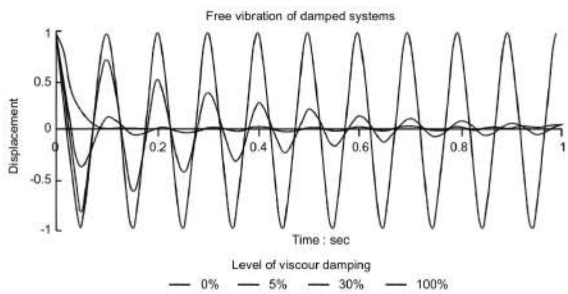
Damping

Damping is the decay of the amplitude of a building's vibrations due to internal friction and the absorption of energy by structural and nonstructural elements. All buildings possess some intrinsic damping. The greater amount of damping reduces amplitude of vibration significantly.

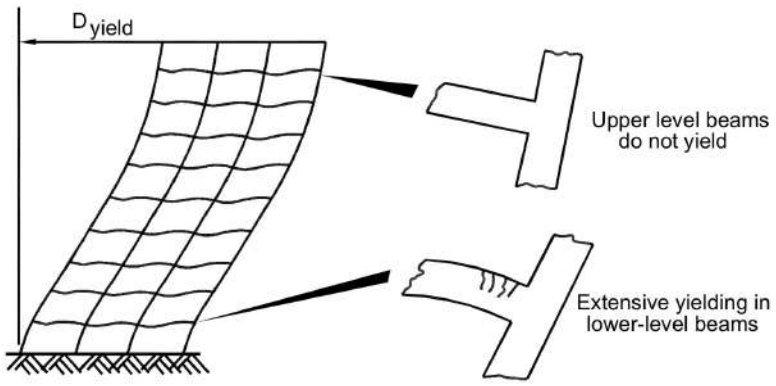
Ductility

The ductility of a structure is in fact one of the most important characteristics affecting its seismic performance. One of the primary tasks of an engineer designing an earthquake resistant building is to ensure that the building should possess enough ductility to withstand the size and types of earthquakes. Ductility leads to energy dissipation in the structure due to inelastic behavior.





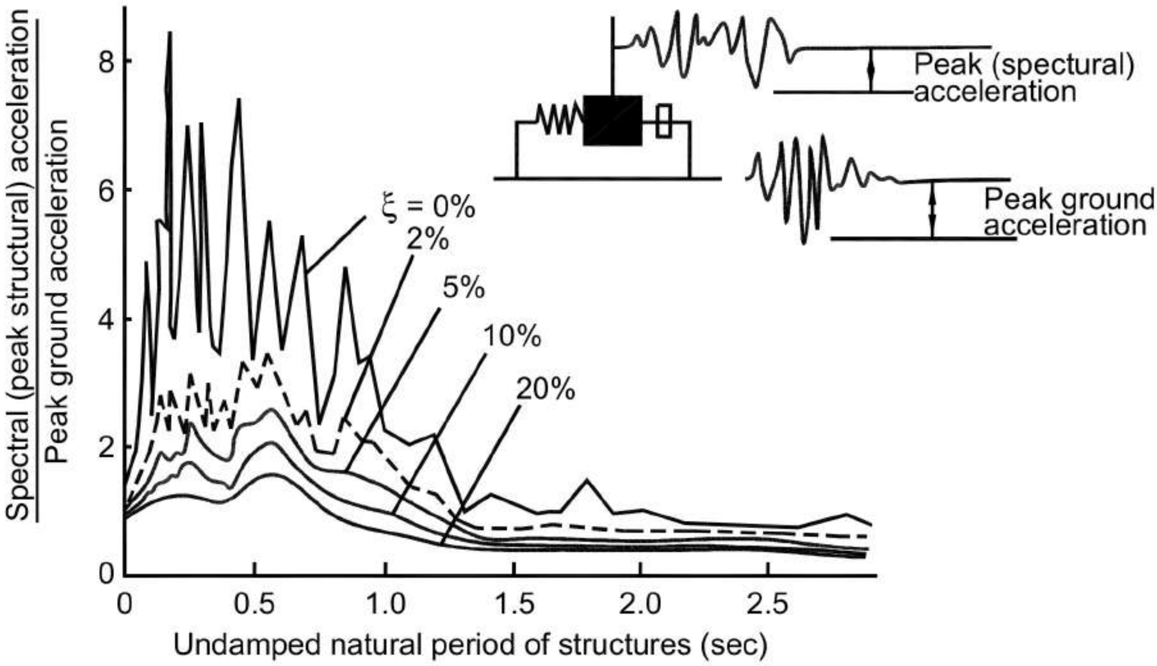
Elastic properties of a building (frequency, stiffness & damping)[4]



In- elastic properties of a building (Ductility)[4]

1.12 How do the building characteristics affect the earthquake forces – a concept of design spectra?

A response spectrum is a graph which plots the maximum response values of acceleration, velocity and displacement against period or frequency. Response spectra are very important "tools" in earthquake engineering. It indicates how the response characteristics vary with building frequency and period: as building period lengthens, accelerations decrease and displacement increases. On the other hand, buildings with shorter periods undergo higher accelerations but smaller displacements. The response spectra also provide some indication about relation of accelerations with frequency characteristics, identifying the frequencies at which a building will undergo peak accelerations. It is a very important step in designing the building to resist earthquakes.

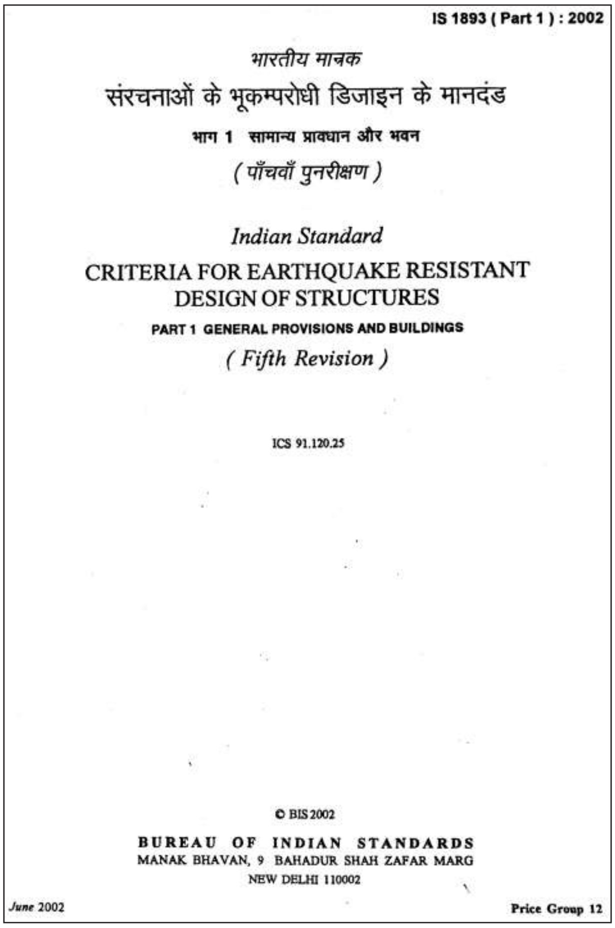


Concept of response spectra[4]

1.13 Can earthquake forces acting on the building be estimated?

Yes. This is remitted in most codes of practice for regular low-to-medium rise buildings and calculated as a function of the parameters as given below;

S.No.	Parameters	Symbol in IS: 1893 (Part 1): 2002
1.	Seismic intensity factor	Z, The probable intensity of seismic motion depending upon Geographical location in which building is located (the seismic areas are defined by seismic zoning maps included in the codes)
2.	Site factor	Depending on the nature of soil layers or foundation Soils, in the form of response spectra of different type of soils
3.	Intended use, which influence acceptable level of damage	I, Importance factor
4.	Structural form, which influence the available ductility	R, Response Reduction factor
5.	Weight of structure and contents	W, Full characteristics dead load plus reduced characteristics live load
6.	First mode period of structure	T, depends upon the mass and stiffness of the building
7.	Spectral acceleration $\left(\frac{S_a}{g}\right)$	Depends upon the T



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भारतीय मानक

संरचनाओं के भूकम्परोधी डिजाइन के मानदंड

भाग 1 सामान्य प्रावधान और भवन

(पाँचवाँ पुनरीक्षण)

Indian Standard

CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

PART 1 GENERAL PROVISIONS AND BUILDINGS

(Fifth Revision)

ICS 91.120.25

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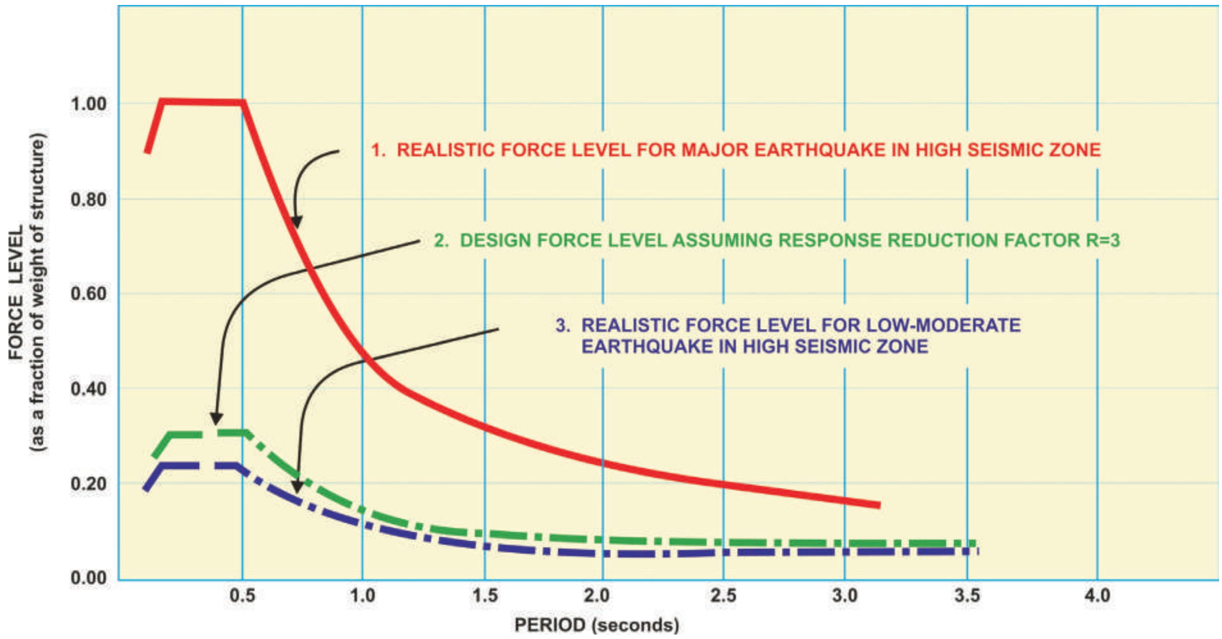
NEW DELHI 110002

June 2002 Price Group 12

IS: 1893 (Part 1) – 2002 "Criteria for earthquake resistant design of structures"

1.14 What is the concept of earthquake resistant design and how to achieve it?

Buildings subjected to earthquake forces generated by strong earthquake motion can be easily analyzed and designed. The concept of earthquake resistant design of structure is some what different from the conventional design approach. In this seismic design approach, the maximum earthquake force can be generated during the strong seismic event at the site called maximum considered earthquake (MCE). But it is neither economically feasible nor justified to design a structure for MCE type of earthquake loading since it has rare possibility of occurrence during the lifetime of the structure as well as quite large in magnitude. This force has reduced to Design Basis Earthquake called (DBE). In other words, we compromise to design the structure elastically only up to the level of DBE and rest of the forces will be resisted by providing ductility in the structure. Therefore, in earthquake design consideration the **strength** and **ductility** are the main two attributes of design.



Concept of design spectra

1.15 What are the other effects of earthquakes responsible for causing damage to buildings?

The other effects of earthquakes are liquefaction, landslide, and less commonly tsunamis.

Liquefaction

Liquefaction is a physical process that takes place during some earthquakes leading to ground failure. As a result, clay-free soil deposits, sands and silts, temporarily lose strength and behave as viscous fluids rather than as solids. Generally, younger and looser the sediment and the higher the water table, the more susceptible a soil is to liquefaction. Liquefaction causes three types of ground failure: lateral spreads, flow failures, and loss of bearing strength. It enhances ground settlement and sometimes generates sand boils (fountains of water and sediment emanating from the pressurized liquefied zone). Sand boils can cause local flooding and the deposition or accumulation of silt.

Landslides

Past experiences have shown that several types of landslides take place in conjunction with earthquakes. The most abundant types are rock falls and slides of rock fragments that form on steep slopes. Shallow debris slides forming on steep slopes and soil and rock slumps and block slides forming on moderate to steep slopes also take place.

Tsunamis

Tsunamis are water waves that are caused by sudden vertical movement of a large area of the sea floor during an undersea earthquake. Tsunamis and earthquake ground shaking differ in their destructive characteristics. Ground shaking causes destruction mainly in the vicinity of the causative fault, but tsunamis cause destruction both locally and at far distant locations from the area of tsunami generation.

Landslide

Liquefaction

Tsunami

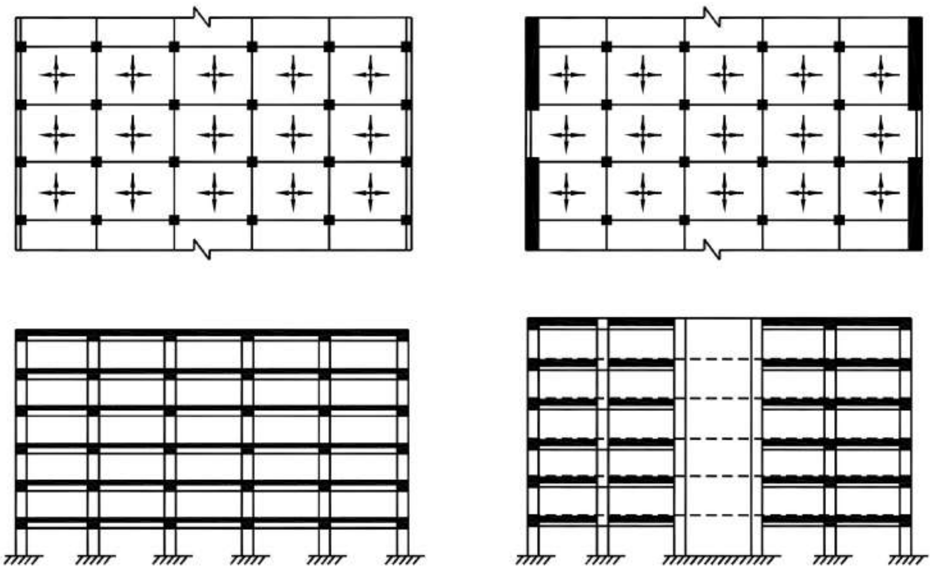
Other effect of earthquakes

Chapter 2

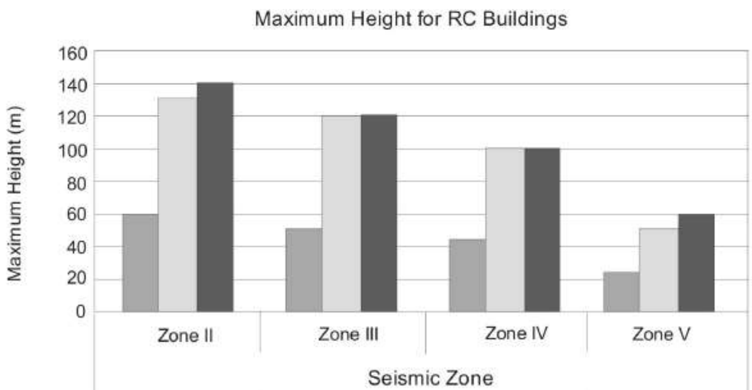
Planning and Design Considerations for Seismically Safe Construction of RC Buildings

2.1 What type of the structural system is suitable for earthquake resistant design of multi-storied RC buildings?

The selection of the load resisting structural system is the most important factor for seismic resistance. It must be of closed loops, enable to transfer all the vertical and horizontal forces to the ground. Three major types of lateral force resisting systems have been suggested namely, moment resisting RC frame system; reinforced concrete frame with shear wall system. They are further subdivided according to type of construction and construction material used. The choice of the system to be adopted for multistoried building depends on the height of building. The moment resisting RC frame systems have several seismic deficiencies like certain zone of failures such as at beam-column joints, or column ends, shear failure, anchorage failure and splice failure of reinforcing bars, short columns and soft or weak stories. The RC frame with shear wall system often proves to be a better choice to resist lateral loads since shear walls are the seismic collapse insurance of a building.



Moment resisting frame and frame with shear wall building system

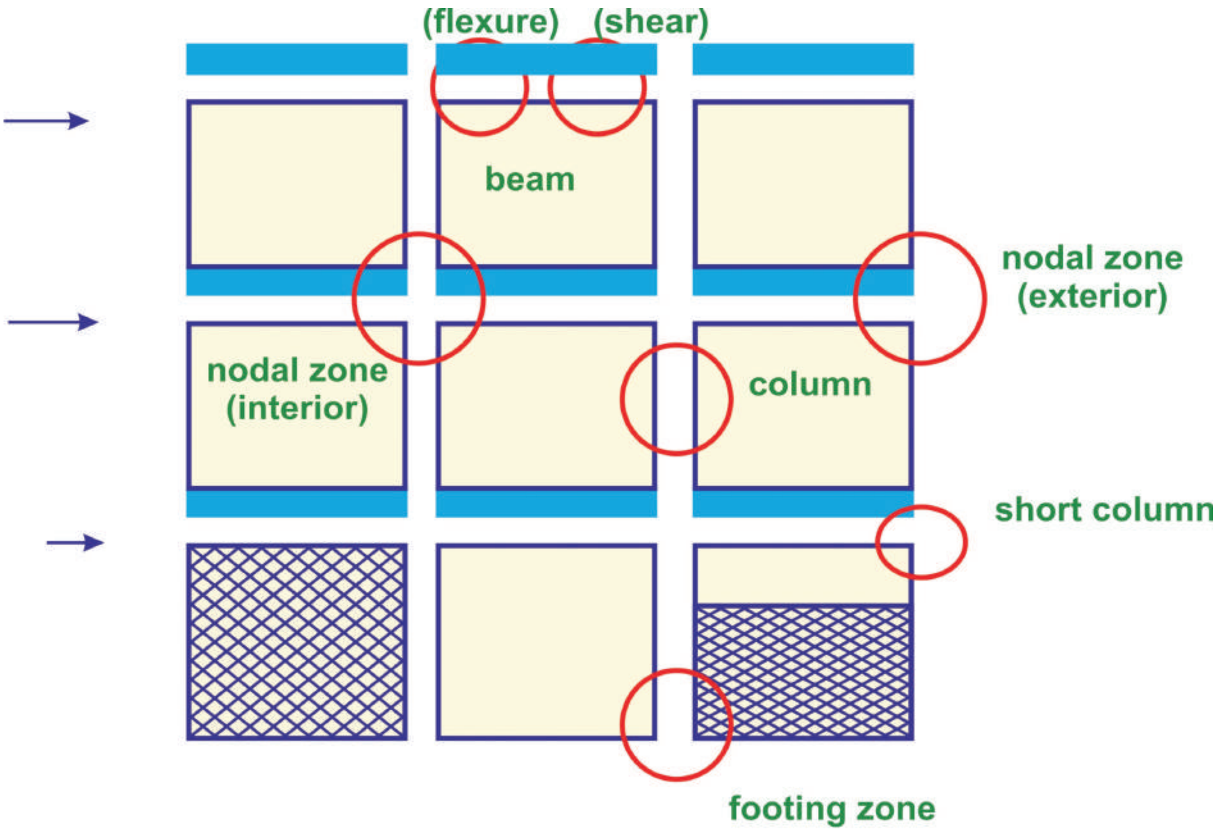


	Zone II	Zone III	Zone IV	Zone V
RC Frame	60	55	45	25
RC Frame and Shear wall	130	120	100	50
RC Shear wall/Coupled Shear wall system	140	120	100	60

Maximum height of different building system in different seismic zones

2.2 How do the moments resisting framing system resist the earthquakes?

Moment resisting frames are preferred architecturally for being unobtrusive and resist vertical and lateral loads through bending. The three main structural components of the resisting frame are column, beam and a rigid beam-column joint besides foundation. The beam and columns ends are the potential regions of failure because moments are maximum at the ends of these components. These members are also subjected to high shear forces to resist the bending. There are some special ductile detailing provisions given in the IS code 13920: 1993 to resist the earthquake forces effectively. These are based on the concept of; ensuring flexural yielding prior shear failure, providing strong column - weak beam. Transverse reinforcements are provided especially in plastic hinge region to confine the core concrete, to ensure adequate ductility, preventing buckling of longitudinal reinforcement and avoiding failure at splicing etc. Capacity design concepts are employed to ensure plastic hinges to be formed in beam ends rather in columns.

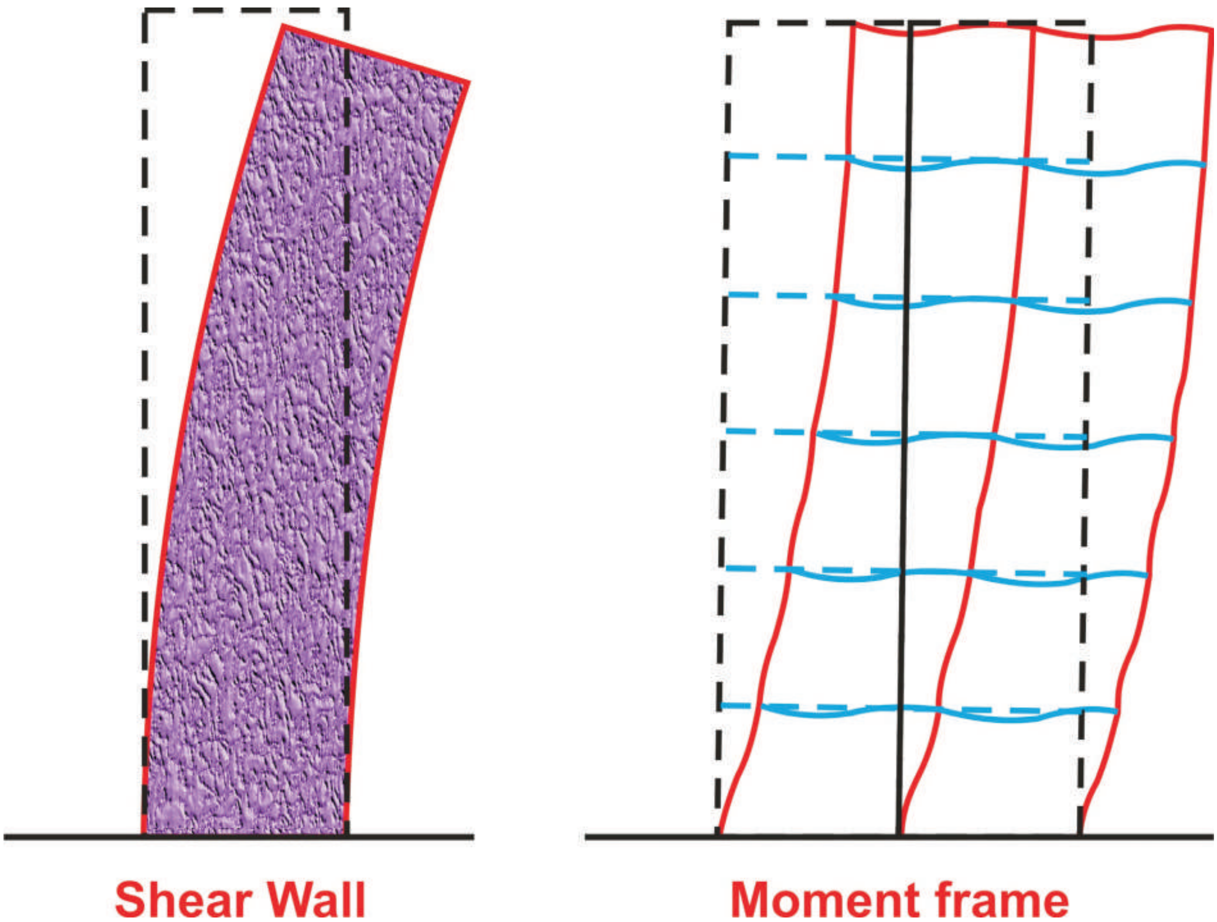


Topics in global behaviour of frames

Potential failure of region in a moment resisting framing system[37]

2.3 What is the dual system for a multi-storied building and how does it differ from moment resisting frame system?

In this system vertical loads are resisted by frames but lateral loads are resisted by both frame and RC wall system. The two systems resist the total design lateral force in proportion to their lateral stiffness. The moment resisting frames resist at least 25% of seismic base shear. In general, a dual system has much more efficient system than moment resisting frame because the inter storey deflection or drift is much less. In dual system, the shear wall resists most of the forces near the base of the building while frame resists most of the upper forces including inertia forces from the wall itself. As a result, the frame exhibits a small variation in storey shear between the first and last floors. Additionally, a secondary lateral support system is also available. This system is somewhat less restrictive architecturally.

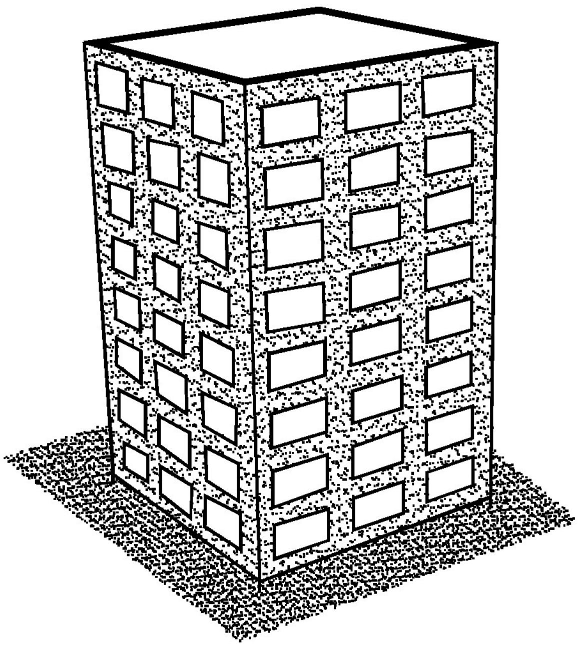


Interaction mechanism between shear wall and frame in a dual system

2.4 What are the guiding principles to design an efficient structure to perform well during an earthquake?

A close coordination is expected between the architect and the engineer to ensure a good outcome, guaranteeing structural safety, reducing vulnerability, and limiting costs. Both partners contribute to their respective expertise. The architect deals primarily with the aesthetic and functional design, while the engineer deals with safety, efficiency and economy of the structure. When a building is designed, a number of parameters are to be considered:

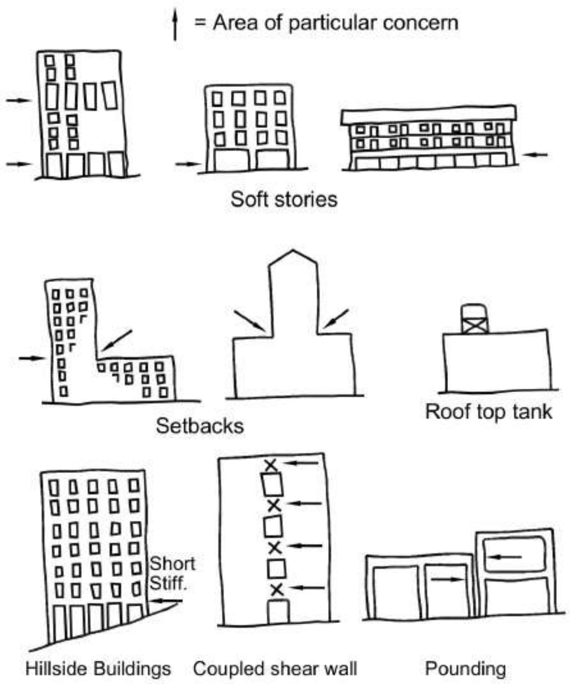
- Provision of a clear and direct load path for transmission of forces from the top of a building to its foundations
- Uniform and symmetrical distribution of mass, strength and stiffness in plan and elevation to avoid the horizontal (torsion) and vertical (soft or weak storey) irregularities
- Regularity in plan, irregular or asymmetrical plan shapes such as L or T configuration may be improved by dividing the building with separation joint to achieve compact, rectangular shapes
- Provision of seismic resistance in both principal axes of a building
- Rigid floor diaphragm at each storey level, to distribute seismic inertia loads at each floor level back to the main vertical seismic resisting elements, such as walls or frames
- A redundant structure is preferred, since such type of structure provides more than one load path available to transmit seismic force, so that if a particular load path becomes damaged or degraded in strength or stiffness during earthquake, another is available to provide backup.
- Appropriate connection between foundation and superstructure is desired to ensure that the whole building is subjected to same excitation.



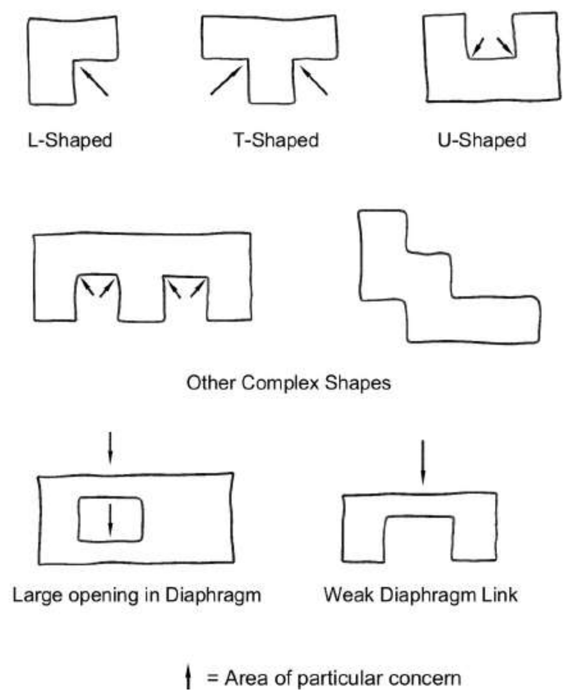
***A well balanced moment resisting frame system*[41]**

2.5 How does the configuration of an RC moment resisting framing system makes it more vulnerable for earthquake forces?

Building configuration is the first important aspect for a sound and robust seismic design of a building since most of the undesirable effects are developed in the building due to undesirable configuration. Building configuration may be regular or irregular in terms of size and shape, arrangement of structural elements and mass. Regular building configurations are almost symmetrical (in plan and elevation) about the axes having uniform distribution of the lateral force. An asymmetrical building with discontinuity in geometry, mass, or load resisting elements is called irregular. These irregularities may create interruption of force flow and stress concentrations and torsional problem. Therefore, we have to take care to identify structural irregularities, and to quantify their damage potential with a reasonable degree of precision to ensure adequate seismic resistance either by modification or strengthening. However, it is practically impossible, and usually not necessary, to avoid these irregularities completely.



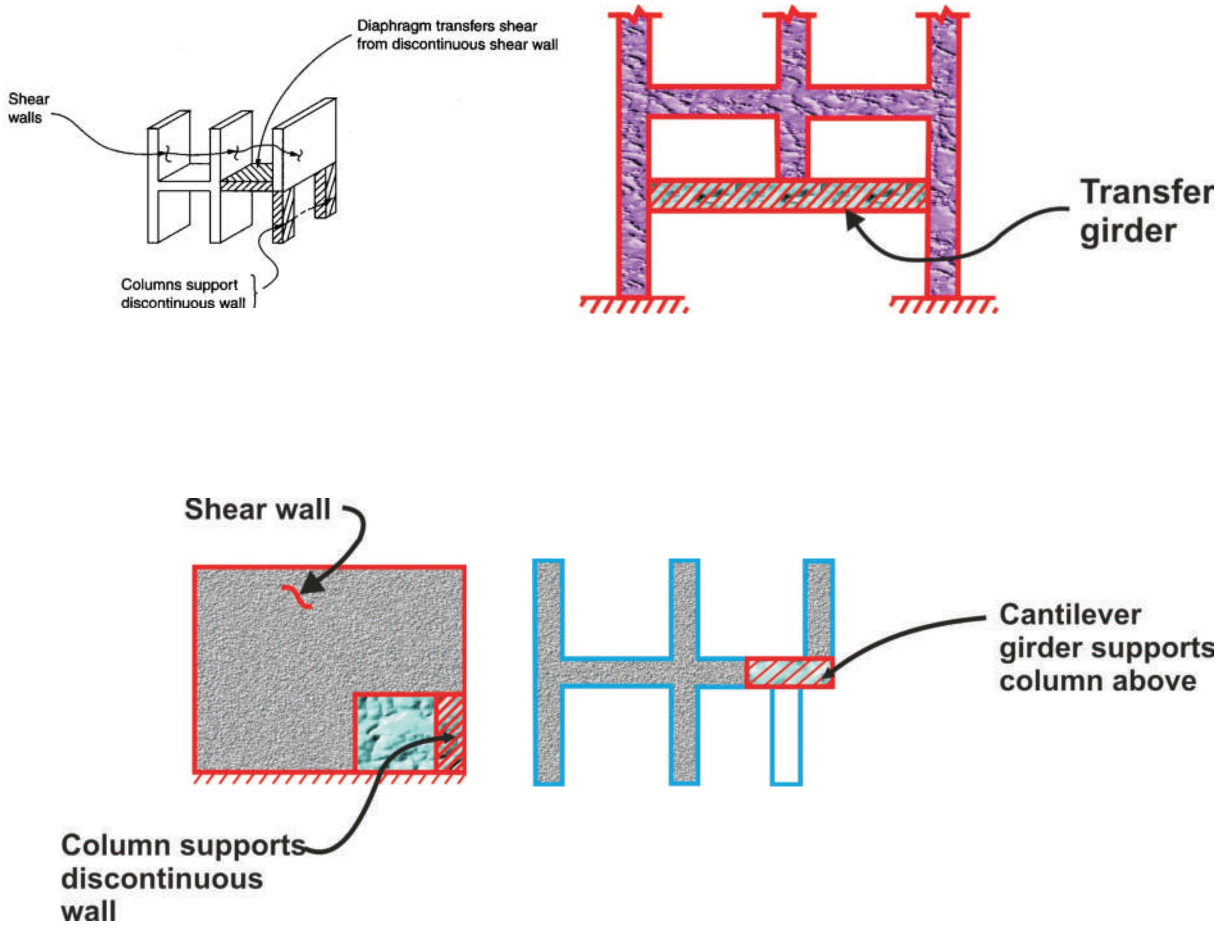
Irregularities in elevation



Irregularities in plan

2.6 How do the discontinues in load path affect the performance of an RC building during earthquake?

Structural damage takes place due to discontinuities/irregularities in the load path or load transfer. Hence, all lateral load-resisting systems, like cores, structural walls or frames should run without interruption from the foundations to the top of the building. These irregularities arise floating box type situation. The most critical region of damage is connecting element (link between discontinuous columns to lower level column) and lower level columns. Therefore, the primary concern in load path irregularities is to consider the strength of lower level columns and that of connecting beams that support the load of discontinuous frame.

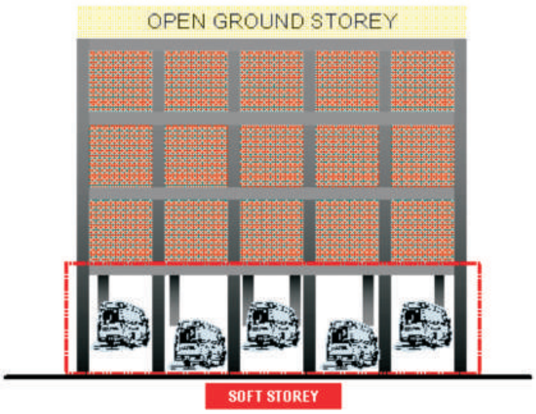


Vertical discontinuities in load path[14]

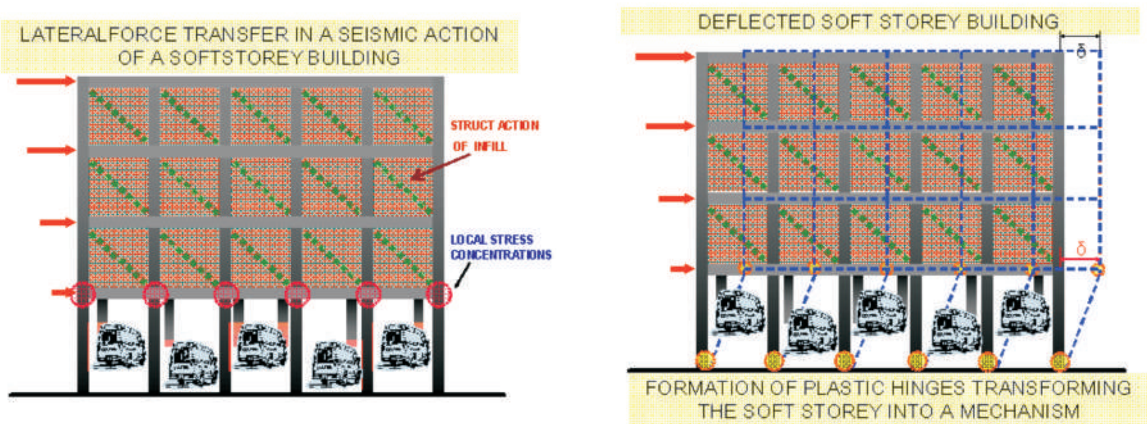
2.7 What is soft storey and how its effect can be minimized during earthquake?

Soft stories at the first-floor level are especially common in multi-story residential buildings in urban areas with an open space for commercial facilities or garages. In the upper floors, the nonstructural masonry partitions and walls are not isolated from the moment frames, creating a large block of masonry that moves more like a rigid body. The taller columns exaggerate the softness of the story compared to the upper floors.

During an earthquake, a soft story may increase significant deformation demands putting the entire burden of energy dissipation on the first-story structural elements, as opposed to distributing the burden along the entire height of the building. The collapse of the soft first-story structure is caused by brittle shear failure in the first story columns. This type of failure is also influenced by torsion, excessive mass on upper floors, P-D effects and lack of ductility in the soft storey. The soft stories deserve a special consideration in analysis and design. The resistance and ductility must be improved in this type of construction.



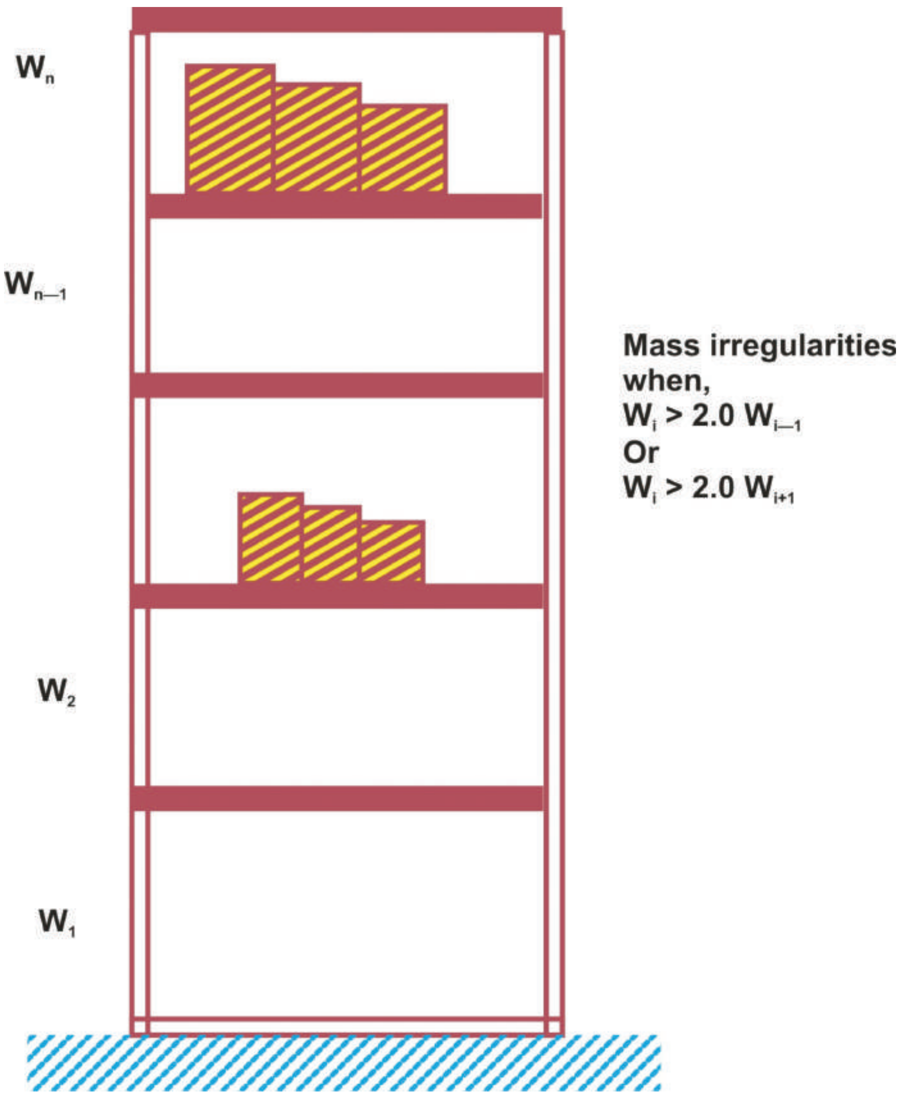
Creation of a soft storey



Potential failure regions and failure mechanism in a soft storey building

2.8 What is mass irregularity in multi-storied RC buildings and what are the possible ways to reduce it?

Mass irregularity exists where the effective mass of any story is more than 200% of the effective mass of an adjacent story. The centre of gravity of lateral forces is shifted above the base in the case of heavy masses in upper floors resulting in large bending moments. For example in case of a six-storey construction comparing seismic effect caused by a certain P weight, placed at the fifth level and, then, at the first level of the same construction; in the case of P at the fifth level, the overturning moments become 25 times greater and shear affects from level 1 to 5 while P- placed at the first level has shear affects 5 times lesser at the first level only. Moreover, excess mass can also lead to increase in lateral inertia forces, reduced ductility of vertical load resisting elements, and increased tendency towards collapse due to *P-delta* effect. Therefore, it is very important to place archives, swimming pools, or rooms containing heavy equipment in lower levels, in order to minimize seismic effects. Where mass irregularities exist, lateral-force resisting elements must be checked using a dynamic analysis for a more realistic lateral load distribution.

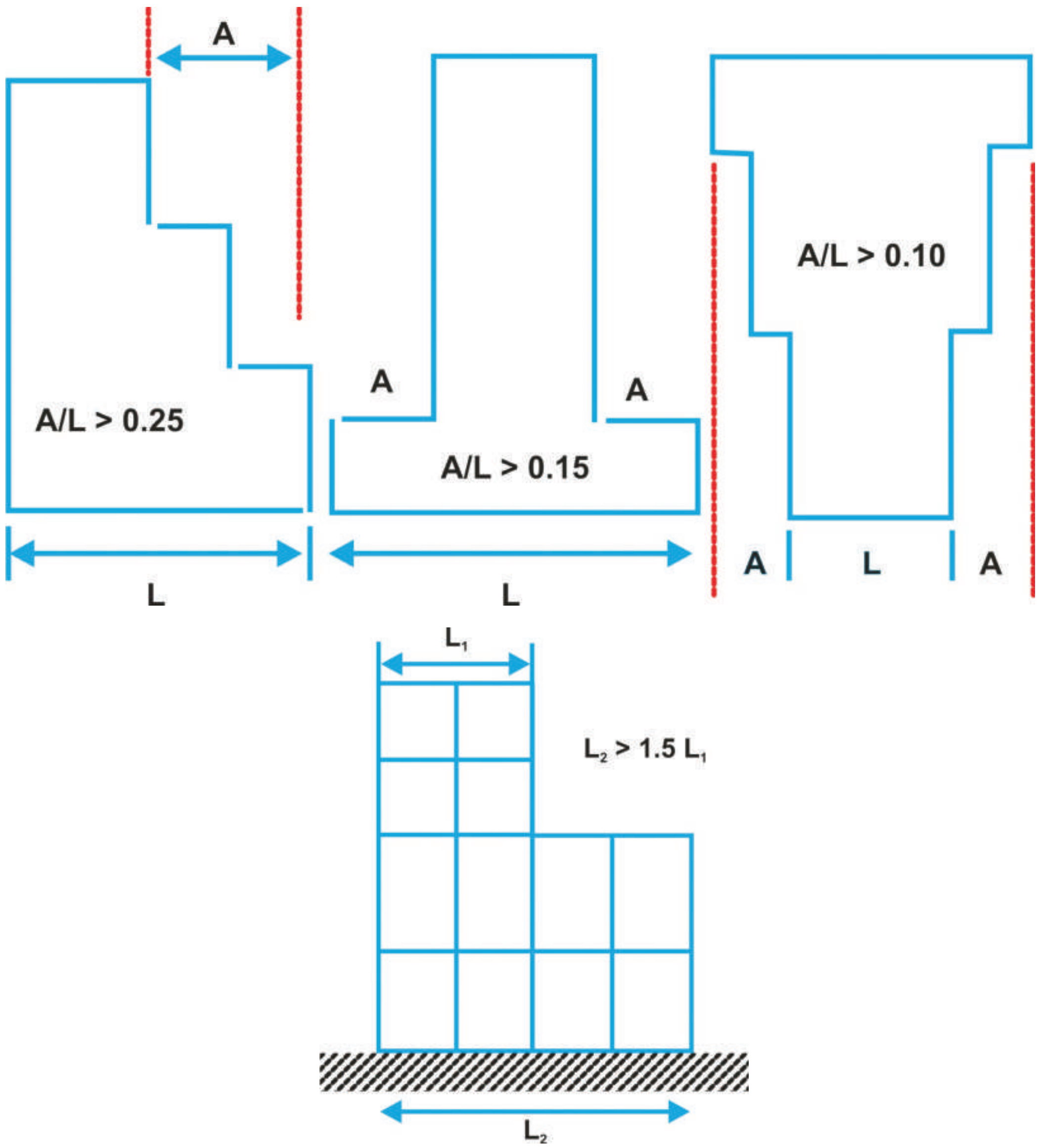


Mass irregularities in RC multi-storied reinforced concrete building[14]

2.9 What is vertical setback and how it can be avoided?

A vertical setback is a geometric irregularity in a vertical plane. The setback can also be visualised as a vertical re-entrant corner. Structure may be called regular if set-back at any floor is not greater than 20% of the dimension of the plan. If the set-back does not preserve symmetry, in each face, the sum of the set-backs at all storeys should not exceed 30 % of the plan dimension of the first storey, and the individual set-back should not exceed 10% of the dimension of the plan.

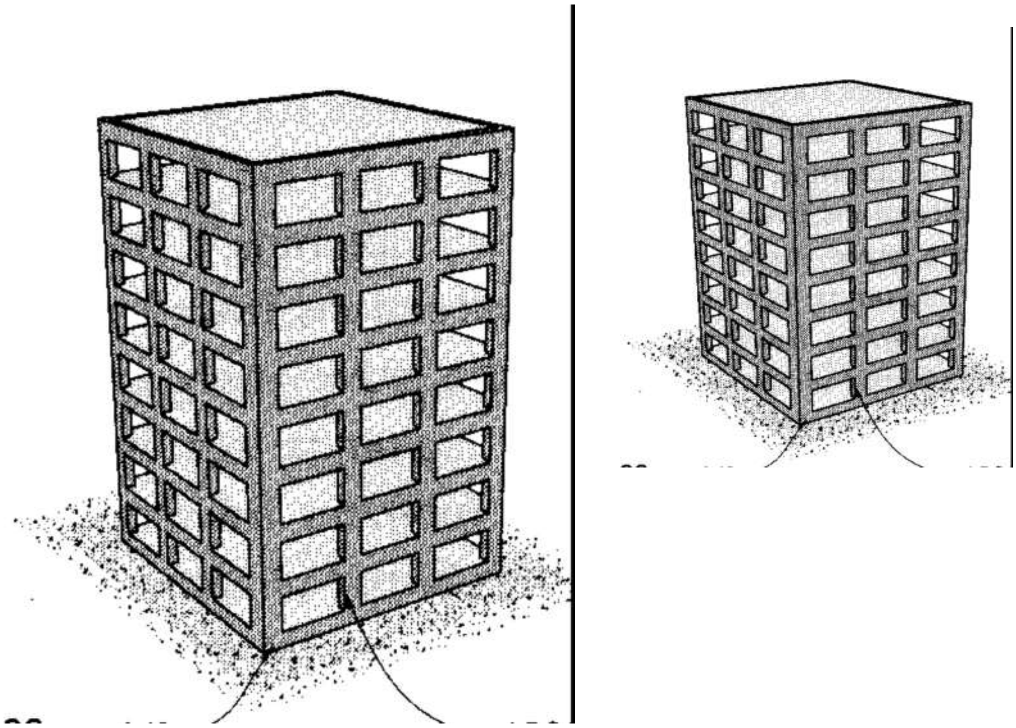
The general solution of a setback problem is total seismic separation in plan through separation section, so that portions of the building are free to vibrate independently. When the building is not separated, check the lateral-force-resisting elements using a dynamic analysis.



Condition for vertical setback in a RC building[14]

2.10 Why the pounding in two RC buildings occur and how it can be avoided?

Pounding damage is caused by hitting of two buildings constructed in close proximity. It may result in irregular response of adjacent buildings of different heights due to different dynamic characteristics. If two buildings of same floor heights are built without separation the damage may not be serious. If the floors of adjacent buildings are at different elevations, the floor of each structure can act like rams, battering the columns of the other building. If one of the buildings is higher than the other, the lower building can act as a base for the upper part of the higher building; the lower building receives an unexpected large lateral load while the higher building suffers from a major stiffness discontinuity at the level of the top of the lower building. Damage due to pounding can be minimized by drift control, building separation, and aligning floors in adjacent buildings.

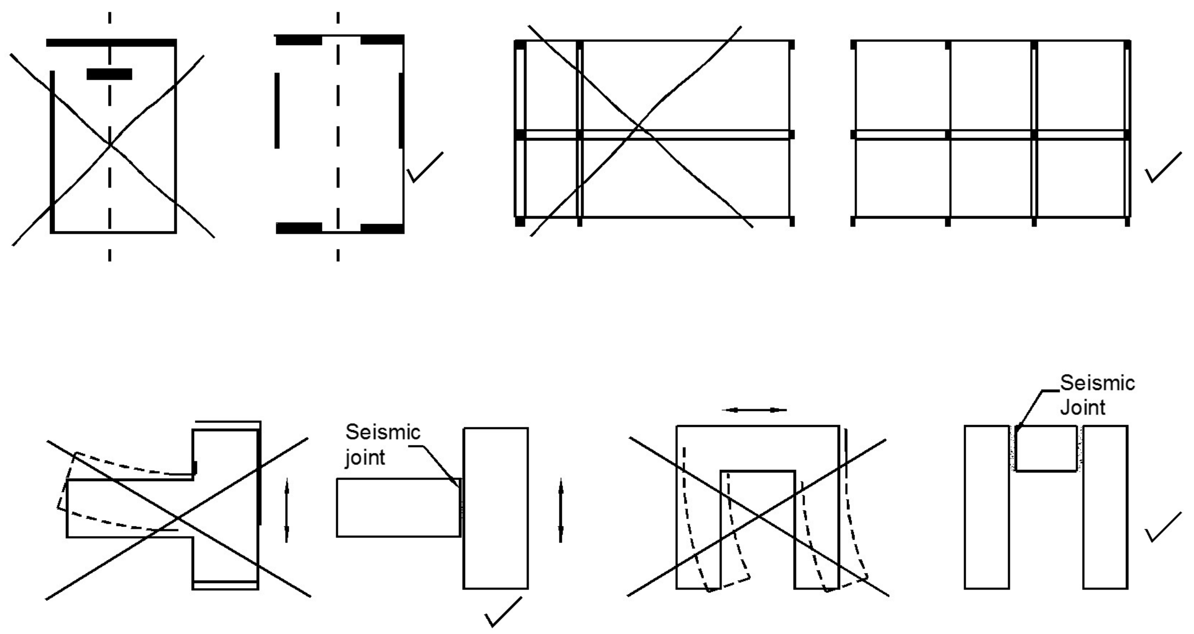


Pounding/hammering in two multistoried RC buildings

2.11 What is a horizontal irregularity in a building and how it influences performance of building?

The eccentricity between the centers of mass and resistance causes torsional moments during an earthquake and results in larger damage in members away from the centre of resistance. For example, structural walls are placed on one side of a building while the other side has open frames. The structural wall is effective to reduce the lateral deformation and resist large horizontal forces but it becomes more effective when it is symmetrically distributed in plan.

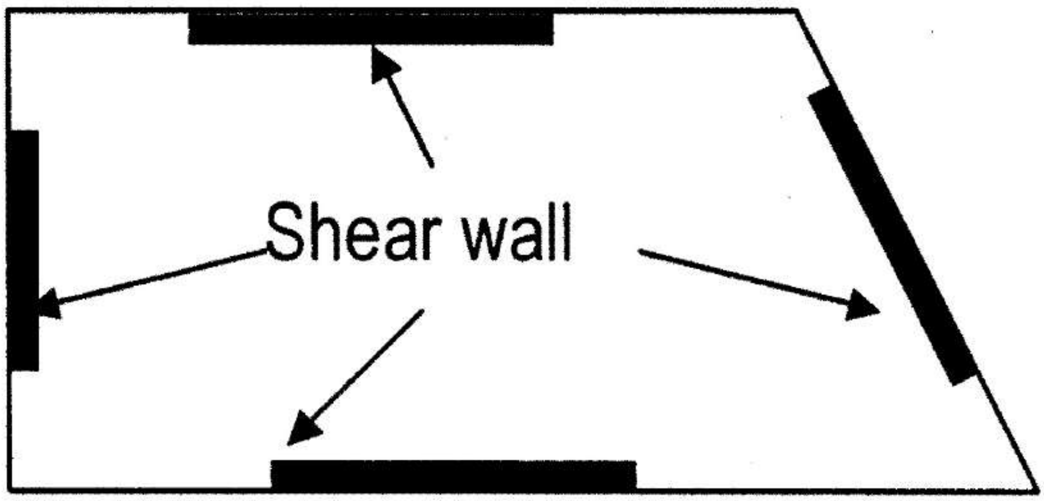
In shape of L, T, H, +, or combination of these shapes damage occurs due to lack of tensile capacity and force concentration. The re-entrant corners of the buildings are subjected to two types of problems. The first is that they tend to produce variations of rigidity causing differential motions between different parts of the building. The second problem is torsion. To avoid this type of damage, either provide a separation joint between two wings of buildings or tie the building together strongly in the system of stress concentration and locate resistance elements to increase the tensile capacity at re-entrant corner.



Horizontal irregularities and re-entrant corners in the buildings

2.12 What are nonparallel systems?

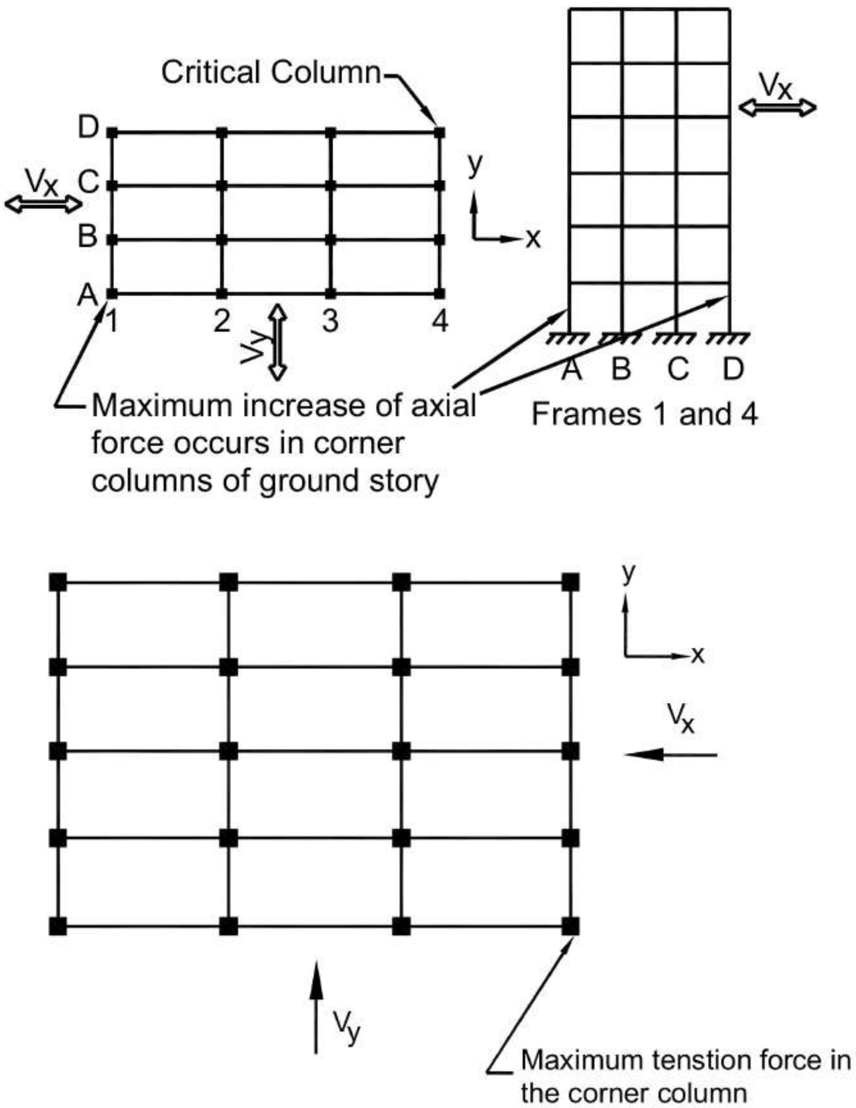
If vertical load resisting elements are not parallel or symmetrical about the major orthogonal axes of the lateral-force resisting system, it is called nonparallel systems. This results in a high probability of torsional forces under ground motion, because the centre of mass and resistance does not coincide. It is often exaggerated in the triangular or wedge shaped buildings resulting from street inter-sections at an acute angle. The narrower portion of the building will tend to be more flexible than the wider ones, which will increase the tendency of torsion. To design these types of buildings, special care must be exercised to reduce the effect of torsion or to increase torsional resistance of the narrow parts of the building.



Non-parallel system in an RC building[14]

2.13 Why corner columns/ corner buildings are more susceptible to earthquake forces?

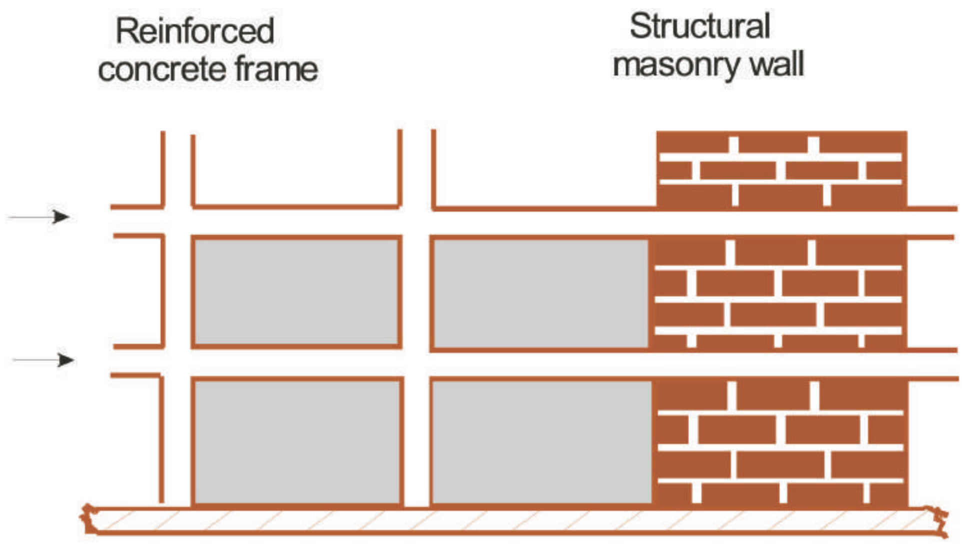
Corner buildings are typically constructed with solid walls perpendicular to the streets with framed window openings parallel to the streets. In these buildings the centre of mass and the centre of stiffness are severely different and this imposes excessive torsional demand on corner buildings causing damage to them.



Corner columns are subjected to maximum axial and tension force due to overturning

2.14 Why the mixed construction system with RC columns and masonry walls is not advisable for earthquake forces?

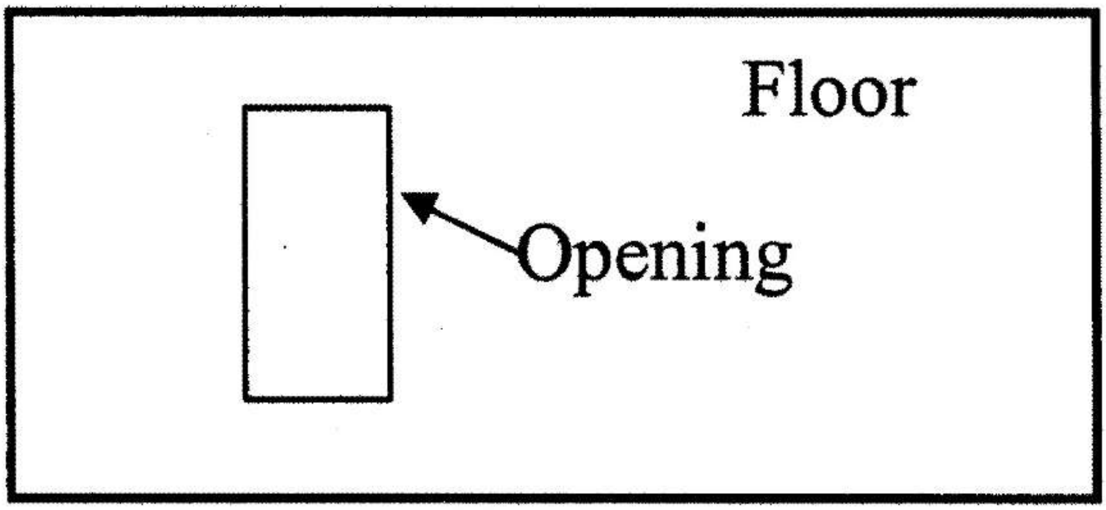
Mixed structural systems with concrete and structural masonry walls behave very unfavorably during earthquakes. The columns in combination with the slabs or beams form frames, which have a substantially smaller horizontal stiffness than the masonry walls. The earthquake actions are therefore carried to a large extent by the masonry walls. When masonry walls fail due to the seismic actions or deflections, they can no longer carry the gravity loads, which usually lead to a total collapse of the building. Mixed systems of columns and structural masonry walls must therefore be absolutely avoided.



A mixed system (RC frame with structural masonry wall)

2.15 What is diaphragm discontinuity and how it influences the seismic performance of a building?

The diaphragm is a horizontal resistant element that transfers forces between vertical resistance elements. The diaphragm discontinuity may occur with abrupt variations in stiffness, including those having cut-out or open areas greater than 50% of the gross enclosed diaphragm area, or change in effective diaphragm stiffness of more than 50 % from one storey to the next. The diaphragm acts as a horizontal beam, and its edge acts as flanges. It is obvious that opening cut in tension flange of a beam will seriously weaken its load carrying capacity. In a number of buildings there has been evidence of roof diaphragms, which is caused by tearing of the diaphragm.



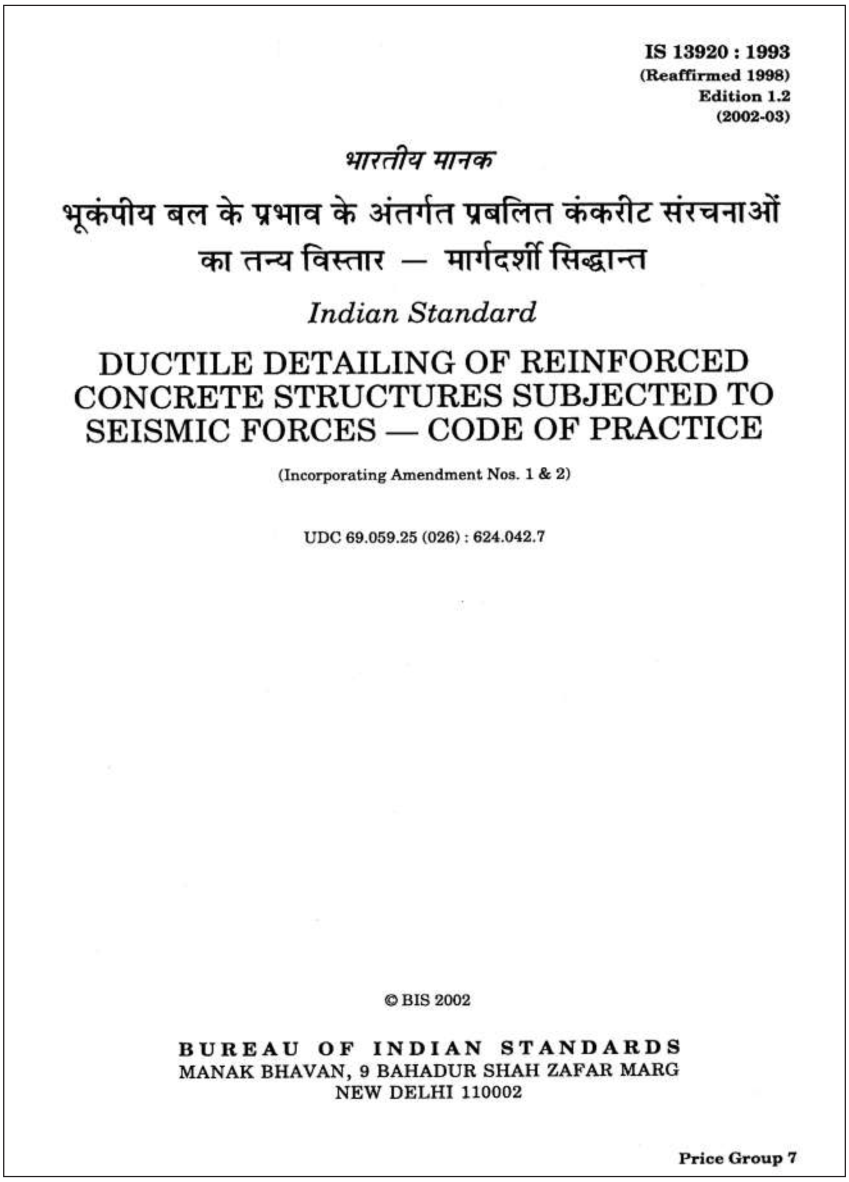
Large openings in the diaphragm reduce the load carrying capacity [14]

Chapter 3

Proportioning and Detailing Considerations in Building Components during Earthquakes

3.1 Why the proportioning and detailing of building components are more important for an efficient seismic design?

Ductility is an important feature to achieve seismically efficient design of RC buildings. It is responsible for redistribution and reduction of internal actions and dissipation of energy in controlled manner at appropriate locations. There always has remained a need to pay attention to proportioning and detailing of each component of the building. In case of multistoried RC buildings the seismic damage is expected to occur in beams. Damage to columns that support the building gravity load is suppressed. This concept of strong columns and weak beams can be achieved by capacity based design. The potential plastic hinge regions or structural fuse usually at ends of beams where the energy is dissipated is to be considered by ensuring a controlled damage in the form of ductility. Provision for proportioning and detailing requirement of various structural members used in multi-storied RC buildings are provided in IS: 13920: 1993.



IS 13920 : 1993
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Edition 1.2
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भारतीय मानक

भूकंपीय बल के प्रभाव के अंतर्गत प्रबलित कंकरीट संरचनाओं का तन्य विस्तार — मार्गदर्शी सिद्धान्त

Indian Standard

DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

(Incorporating Amendment Nos. 1 & 2)

UDC 69.059.25 (026) : 624.042.7

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NEW DELHI 110002

Price Group 7

Proportioning and detailing of building components as per IS 13920: 1993

3.2 What ductile detailing is required in beams of a multi-storied reinforced concrete building?

General Provisions and ductile detailing requirements in the special moment resisting frame members of a reinforced concrete building are given in IS: 13920: 1993.

Design of Flexural Member (Factored axial stress less than *0.1 f_{ck}*)

Proportioning requirements

- Beam width-to-depth ratio ≥ 0.3
- Beam Width $\geq$ 250mm or $\geq$ width of supporting member (on plane perpendicular to bean axis) plus distance on each side of not greater than $\frac{3}{4}$ of overall beam depth
- Beam Width $\leq (b_c + h_w)$ or $\leq 2b_c$, Where bc largest cross sectional dimension of column perpendicular to beam and hw depth of beam and centroidal axes of beam and column must mot be more than $(b_c/4)$
- Effective depth $\geq \frac{1}{4}$ (Beam Clear Span)

Detailing requirements

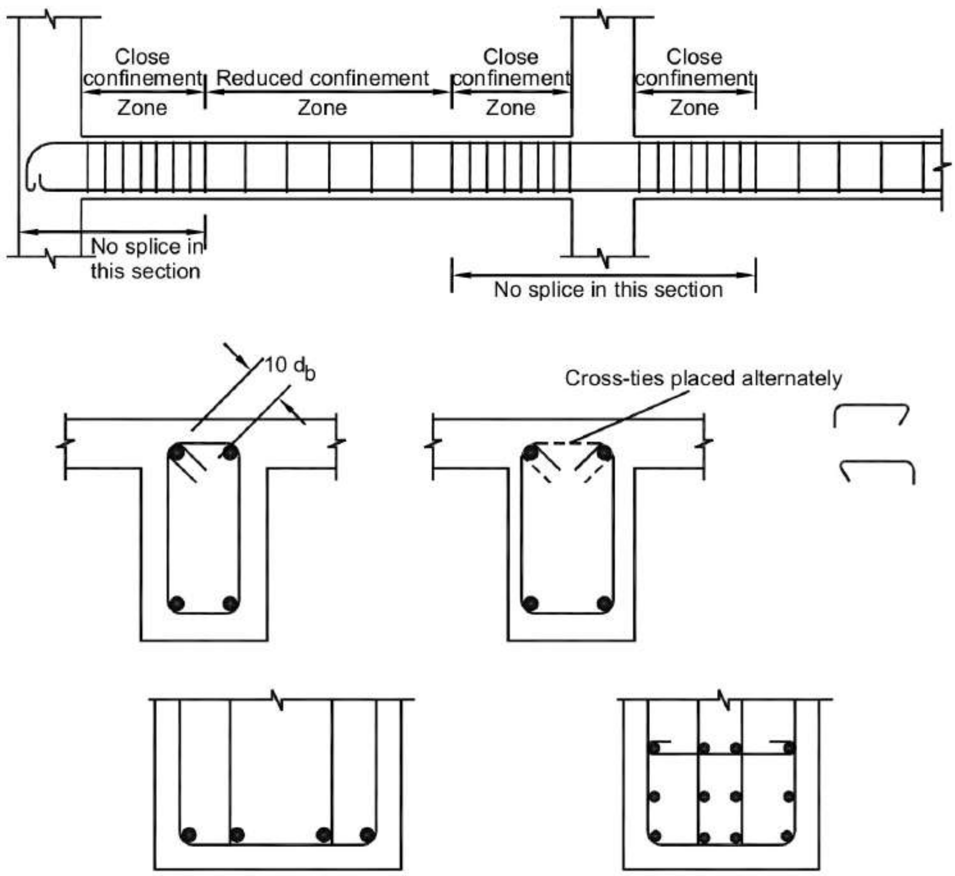
Longitudinal reinforcement or Main Reinforcement

- Tension steel ratio $\rho_{\min} \leq 0.24\sqrt{f_{ck}}/f_y$ or $1.38/f_y$ where f_y is the yield strength of main reinforcement
- Maximum steel ratio at any section, $\rho_{\max} = 2.5$ % (Ratio of reinforcement is normalized to $b_w d$ where b_w is the beam width and d is effective depth)
- Top and bottom reinforcement shall consist at least 2 bars throughout the member length
- The positive steel at a joint face must be at least equal to half the negative steel at that face.
- Positive and negative strength elsewhere in the beam $\geq$ 25% maximum bending strength at joints
- The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a quarter length of the member or anticipated plastic hinge length (c) within 2 times member depth from joint face.
- Not more than 50 percent of the bars shall be spliced at one section.
- The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at spacing not exceeding 150 mm.

Continued...

Transverse or Shear reinforcement requirement

- In the special confinement zone, the spacing of hoops over a length of 2d at either end of a beam shall not exceed
  - d/4,
  - 8 times the diameter of the smallest longitudinal bar
  - 24 times the diameters of hoop bars
  - However it need not be less than 100 mm.
- The first hoop shall be at a distance not exceeding 50 mm from the joint face. Elsewhere the beam shall have vertical hoops at a spacing not exceeding d/2.
- Transverse reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10-diameter extension (but not < 75 mm) at each end that is embedded in the confined core.
- In compelling circumstances, it may also be made up of two pieces of reinforcement; a U stirrup with a 135° hook and a 10-diameter extension (but not < 75 mm) at each end, embedded in the confined core and cross tie. A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars.
- The minimum diameter of the bar forming a hoop shall be 6 mm. However, in beams with clear span exceeding 5 m, the minimum bar diameter shall be 8 mm.

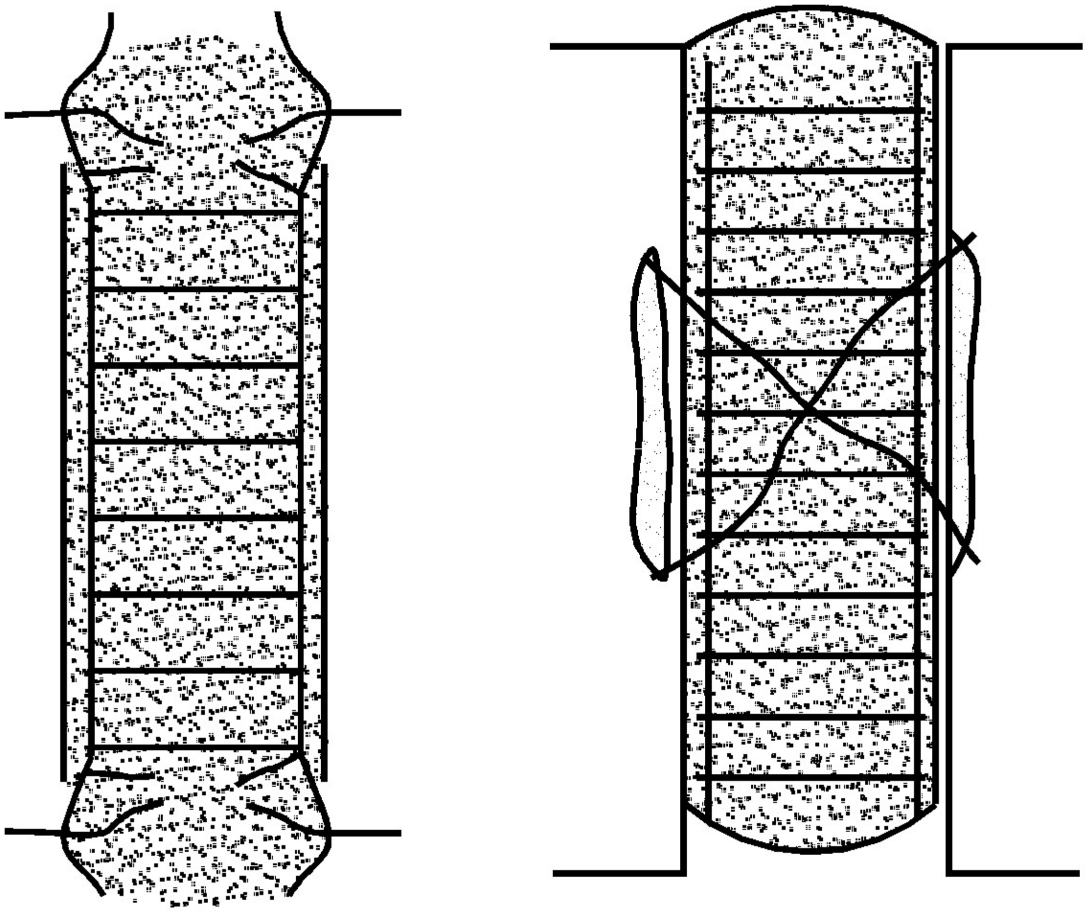


Details of reinforcement in beams [4]

3.3 What type of damage is possible in columns of a multi-storied RC building and how it can be avoided?

A reinforced concrete member normally fails after the yielding of longitudinal reinforcement and when the concrete fails in compression. It is normally called flexural compression failure. The deformation capacity of a column is influenced by the level of axial force in the column and the amount of lateral reinforcement provided in the region of plastic deformation. During an earthquake, exterior columns, especially corner ones, are subjected to varying axial force due to the overturning moment of a structure. The lateral confining reinforcement can delay the crushing failure of concrete under high compression stresses.

Another common mode of failure in column is shear or brittle mode of failure. Shear force causes tensile stress in concrete. Brittle shear failure occurs in the diagonal tension mode when minimum amount of lateral reinforcement (size, spacing and strength of shear reinforcement) is not provided in the member. Columns in building with small and widely spaced ties or inadequate shear resistance, perform very poorly.



Flexure and shear mode of failure in columns[28]

3.4 What proportioning and detailing considerations are required in column to avoid flexure and shear failure?

General Provisions and ductile detailing requirements in the special moment resisting frame members of a reinforced concrete building are given in IS: 13920: 1993.

Column subjected to bending and axial load (IS 13920:1993 specification will applicable if axial stress > 0.1 f_{ck})

- Minimum dimension of the member < 250mm
- Shortest cross-section dimension / Perpendicular dimension < 0.4

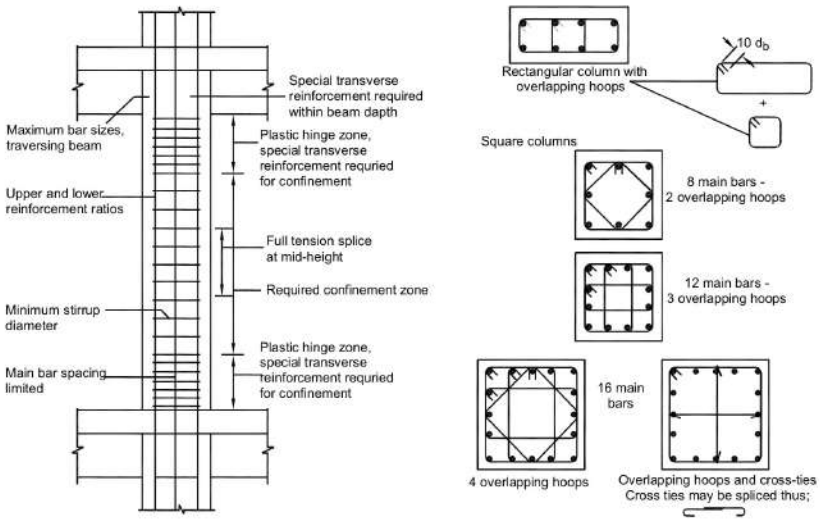
Longitudinal (Vertical) Reinforcement

- Lap splices only in central half portion of the member
- Hoops over the entire splice length at a spacing < 150mm
- Not more than 50 % bar shall spliced at one section

Transverse (shear) Reinforcement

- Hoop requirement as per ***Figure 7A in IS 13920: 1993***
- If the length of hoop > 300mm a cross tie shall be provided as shown in Figure 7B or detailed as Figure 7C.
- Hoop spacing shall not to exceed half the least lateral dimension of column i.e. 300/2=150mm
- The design shear force for column shall be maximum of (a) and b).
  a) Calculated factored shear force as per analyses or
  b) Shear force calculated on the basis of moment capacity of the beams

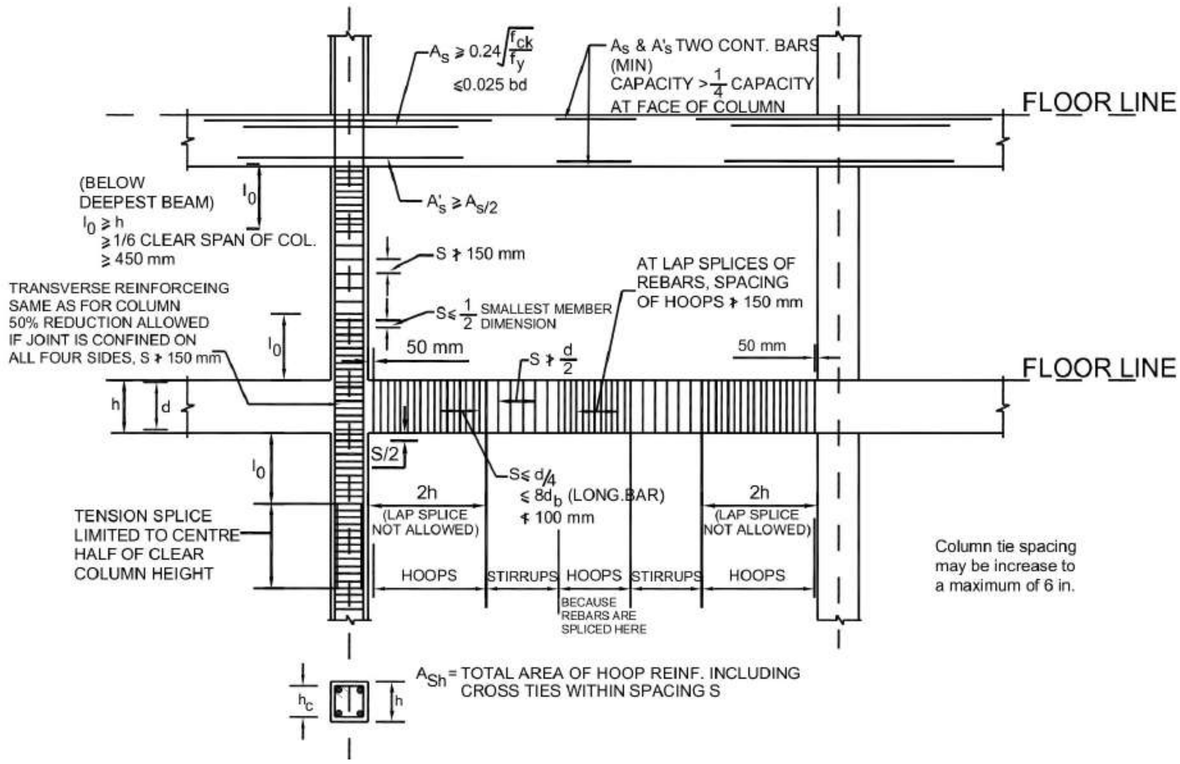
- The spacing shall be lesser of
  a) 0.75 d = 0.75 x 480 = 360mm
  b) 300mm (7.3.3)



Details of reinforcement in beams[4]

Special Confining Reinforcement

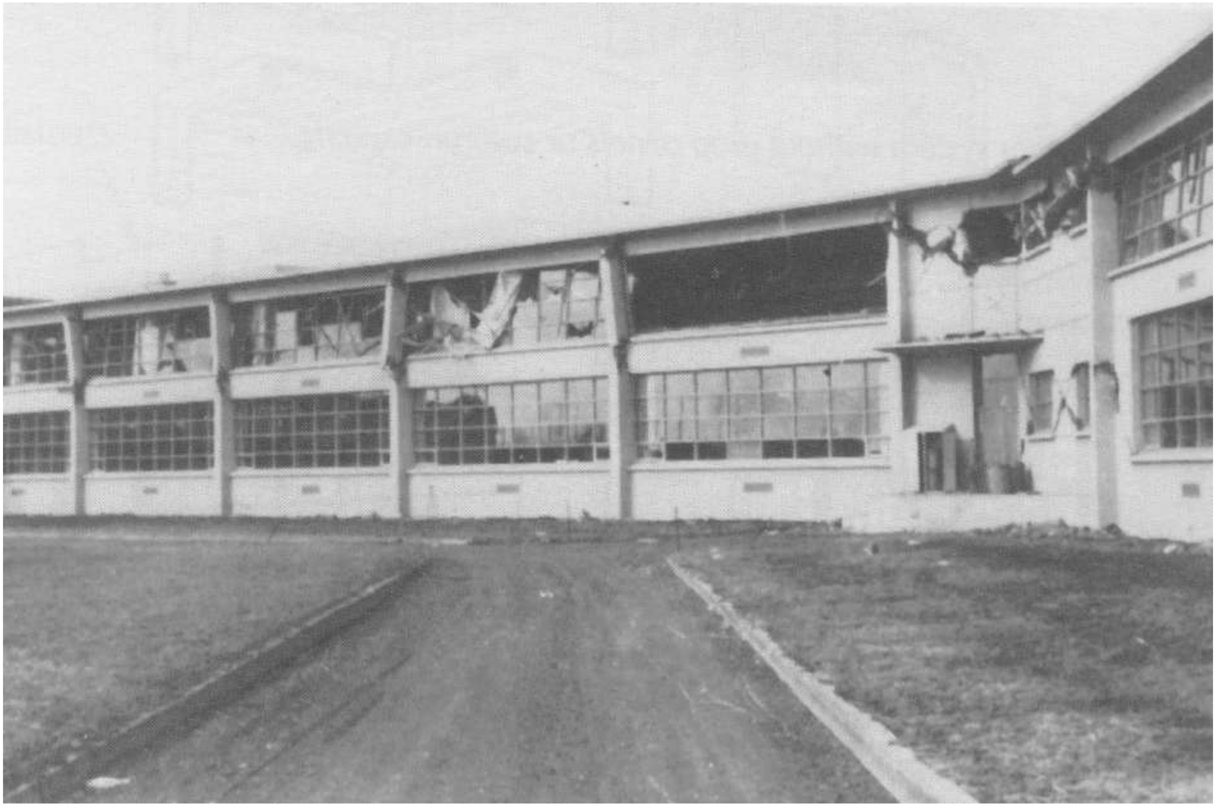
- Special confining reinforcement will provide over a length of l_0 towards the mid span of column
- $l_0 \leq$
- The spacing of hoop shall not exceed
- $S_{max} \geq \begin{cases} 1/4(\text{minimum member dimensions}) \\ \text{should not be less than 75mm} \\ \text{should not be greater than 100mm} \end{cases}$
- Minimum area of cross section of the bar forming hoop is
- $A_{sh} = 0.18\ Sh\ f_{ck}/f_y\ (A_g/\ A_k - 1.0)$



Details of reinforcement in building frame

3.5 Why the strong column- weak beam design is preferred in reinforced concrete buildings?

In most frame structures, the beams have remained strong and elastic, the columns comparatively weaker and failure has occurred in the form of compression crushing, plastic hinging, or shear failure. Whenever damage develops in columns, strength and stiffness degradation will be further aggravated by the presence of axial forces. Excessive column damage means loss of lateral load resistance and also loss of gravity load resistance. Hence, the strong-column weak-beam alternative is promoted by building codes.



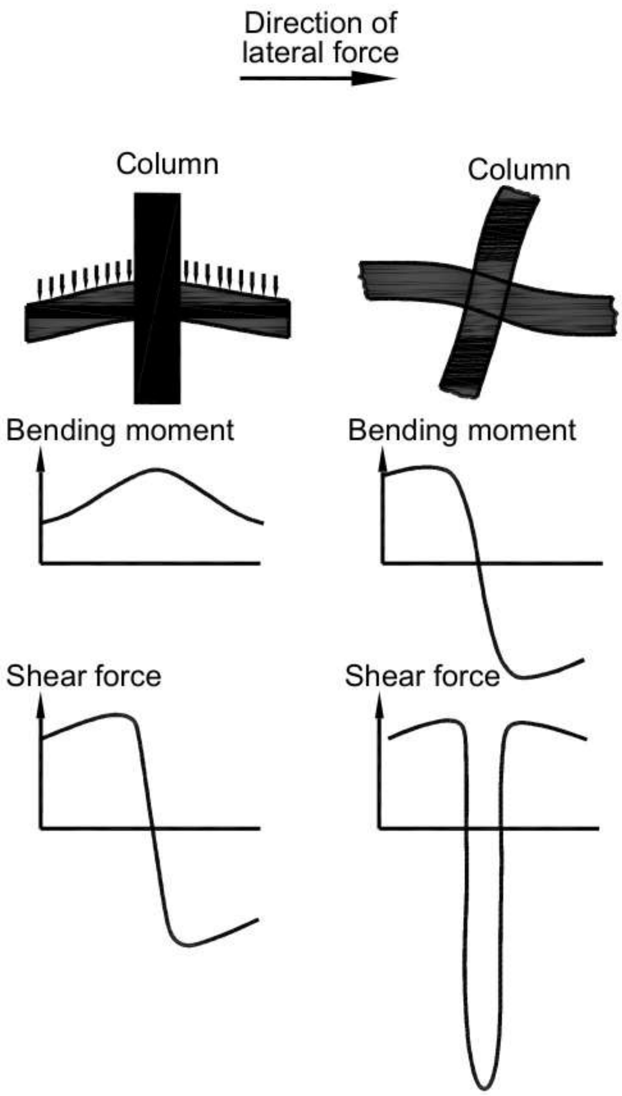
***Failure of concrete columns with top and bottom hinging*[31]**

3.6 Why the beam - column joints are vulnerable to earthquake forces?

The beam- column joints are heavily stressed; the shear stress is many times greater than those in a frame subjected solely to gravity loads. If the beam longitudinal reinforcement is not fully anchored in a beam-column joint, the bar may pull out from the joint; e.g., beam bottom reinforcement. Moreover, wide flexural cracks develop at the beam end and attribute to the slip of beam reinforcement.

Detailing requirement at the joints of frames

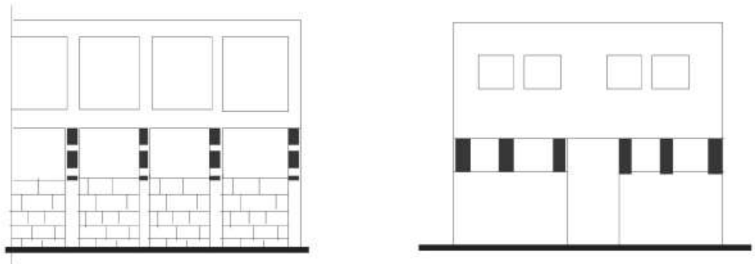
- The special confining reinforcement as required at the end of column shall be provided through the joint as well, unless the joint is confined by 8.2
- A joint which has beam framing in all vertical faces with beam width, is at least $\frac{3}{4}$ of the column width, it may be provided with half the special confining reinforcement required at the end of the column. The spacing of hoop shall not exceed 150mm.



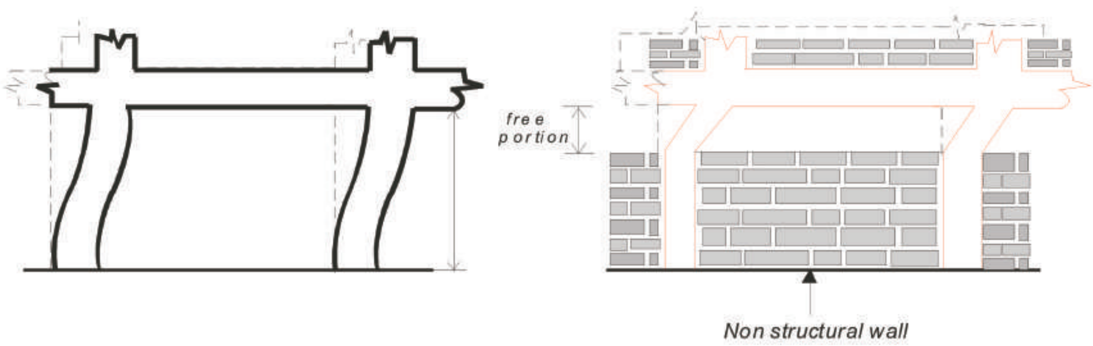
Shear in beam column joint (a) gravity frame and (b) sway frame[4]

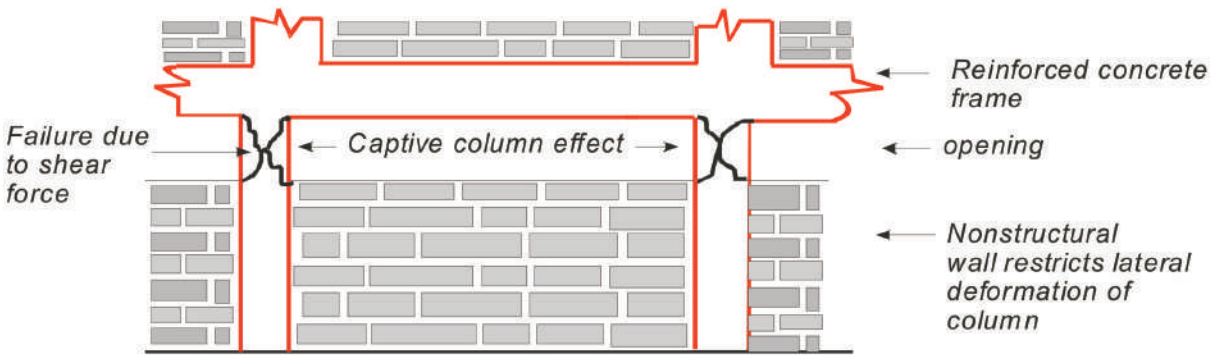
3.7 What are the seismic design deficiencies in short column and how can they be protected?

In a short-column effect the column is kept "captive" and only a fraction of its height can deform laterally, corresponding to the "free" portion. As a result the seismic shear increases inversely proportional to the cube of its height and the column is forced to fail in a non-ductile, shear-governed mechanism. The best solution is to avoid creating such condition or the masonry infill or nonstructural walls must be separated from the frame with joints or gaps between the column and the infill so that the gaps allow the columns to behave flexurally and to deform in a ductile manner. Short column may behave satisfactorily if sufficient shear resistance is provided.



EXAMPLES OF SHORT COLUMNS



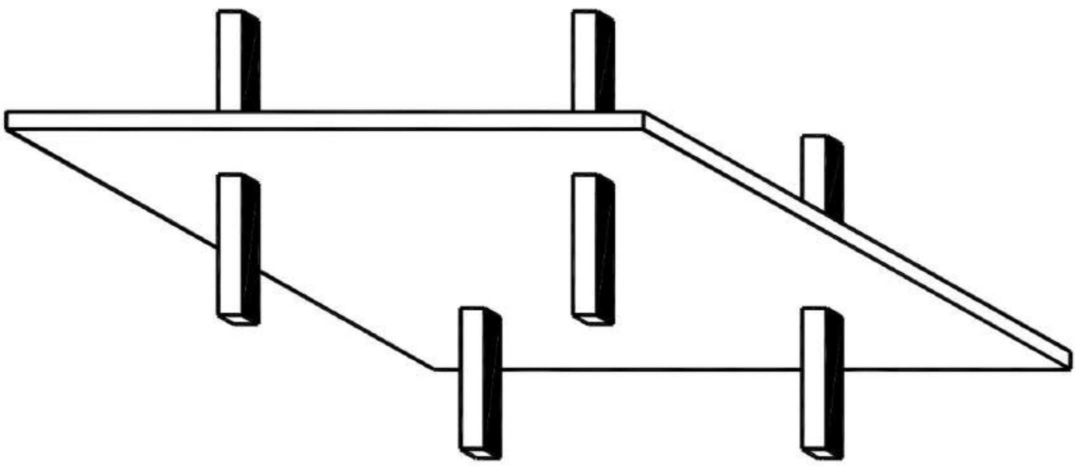


Typical captive-column failure

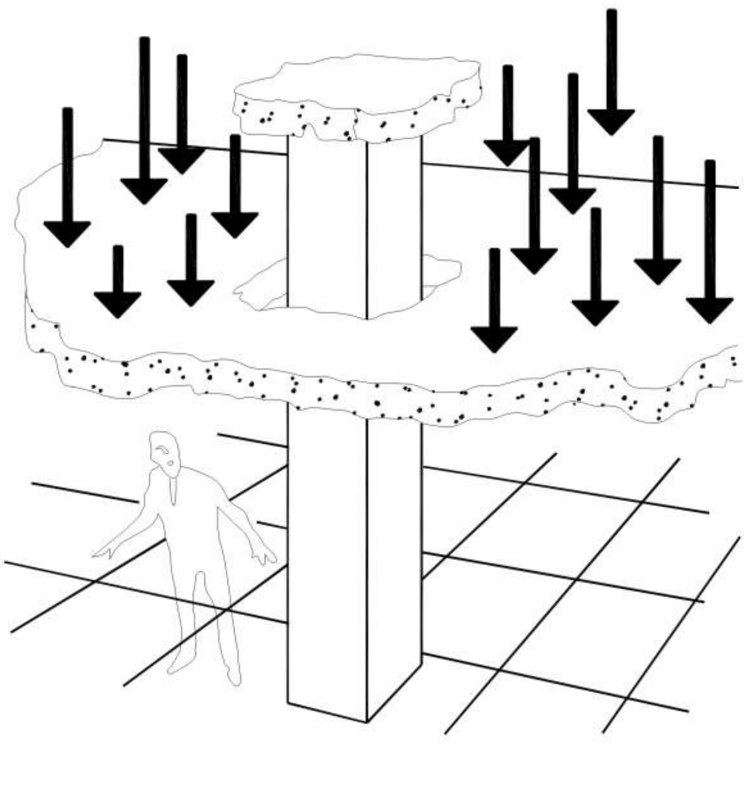
***Failure mechanism of short columns in a building*[40]**

3.8 Is the flat-slab construction advisable in seismic areas?

A flat slab without column capitals is popular in some regions. Its critical part is the vertical shear transfer between the slab and a column. The shear failure at the connection leads to the "the pan-cake collapse", leaving no space between the adjacent floors after the collapse. Serious failure has been observed flat slab construction in past earthquakes.



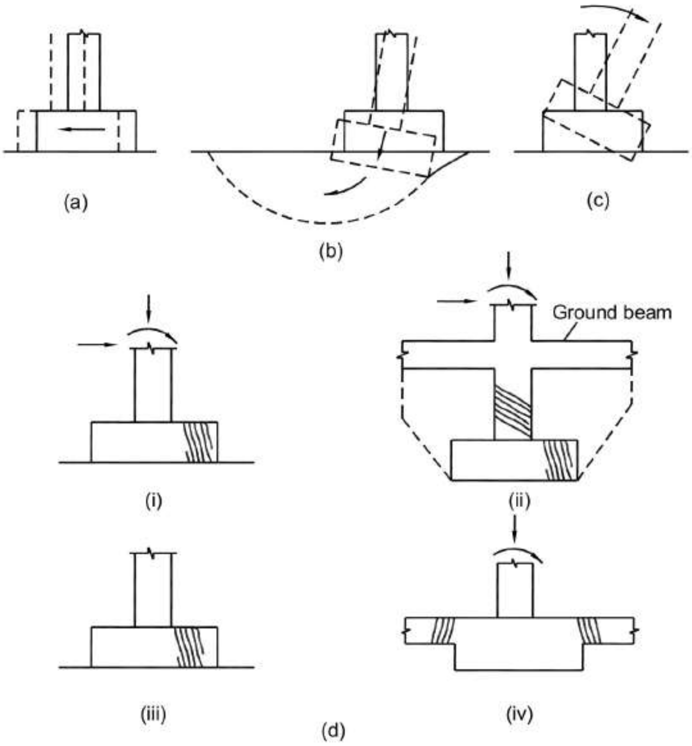
Flat slab floor system without drop panels or column capitals [11]



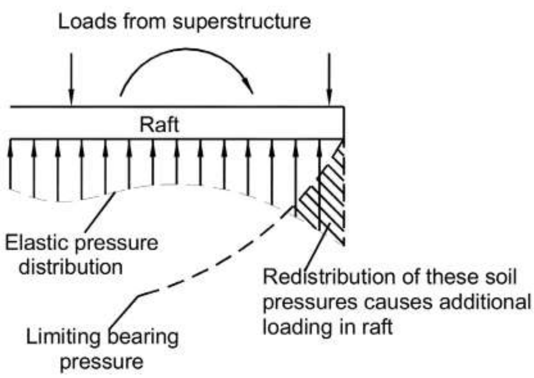
Punching shear failure mode of flat slab system. This can lead to pan-caking of an entire buildings [11]

3.9 Is the foundation failure a threat to a building?

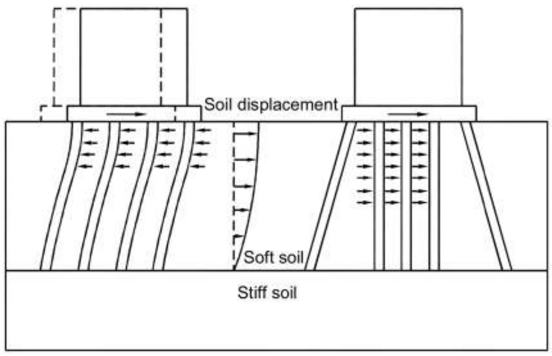
The foundation failure is caused by: a) liquefaction and loss of bearing or tension capacity, b) landslide of slopes, c) fault rupture, d) compaction of soils, and e) differential settlement. It is normally difficult to design and construct safe foundation to meet the ground movement immediately above the fault rupture. The cost of damage investigation and repair work becomes extremely expensive in case of failure of foundation. Therefore, possibility of foundation failure should be reduced.



Mode of failure in pad foundations: (a) sliding failure (b) bearing capacity failure (c) overturning (d) structural failure, where (i) show shear failure in footing, (ii) shows shear failure in stub column, iii) shows bending failure in footing and (iv) shows bending failure in ground beam[4]



Pressure distribution near the edge of a raft under seismic loading[4]



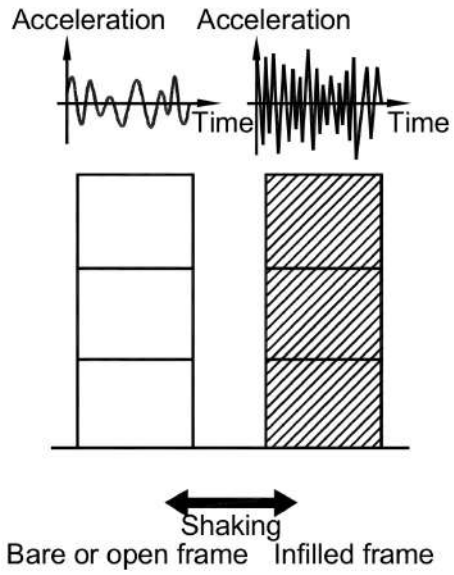
The effect of earthquake on pile group deformations[4]

3.10 What are nonstructural elements in an RC building and why are they seismically vulnerable?

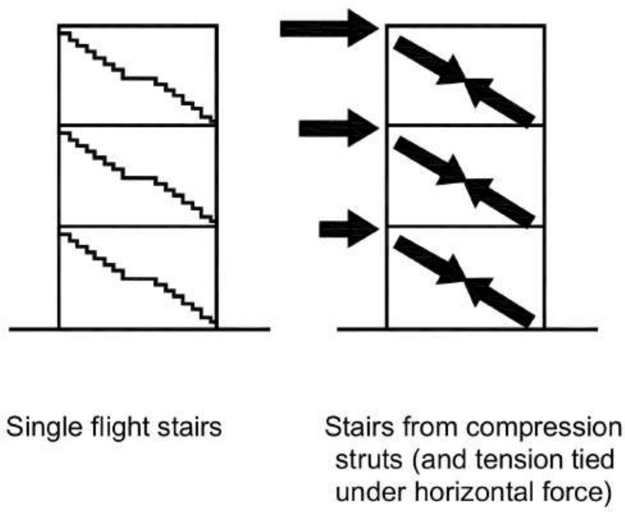
Nonstructural elements are infill wall, partition walls, parapet walls, staircase etc. Damage of nonstructural elements creates a falling hazard for people in, or escaping from the building; Moreover, fallen elements may block evacuation routes in a severely damaged building. The effects of following non-structural elements have to be considered at appropriate stage i.e. planning, analysis, design and construction.

Staircases are normally discarded in structural analysis although they contribute significantly to the stiffness of the framing system. These high-stiffness nonstructural elements cause irregular stiffness distributions in plan or along the height.

In several buildings, the brick veneer is observed nonstructural falling is hazardous to residents fleeing from a building during or after an earthquake or to passers by in the street.



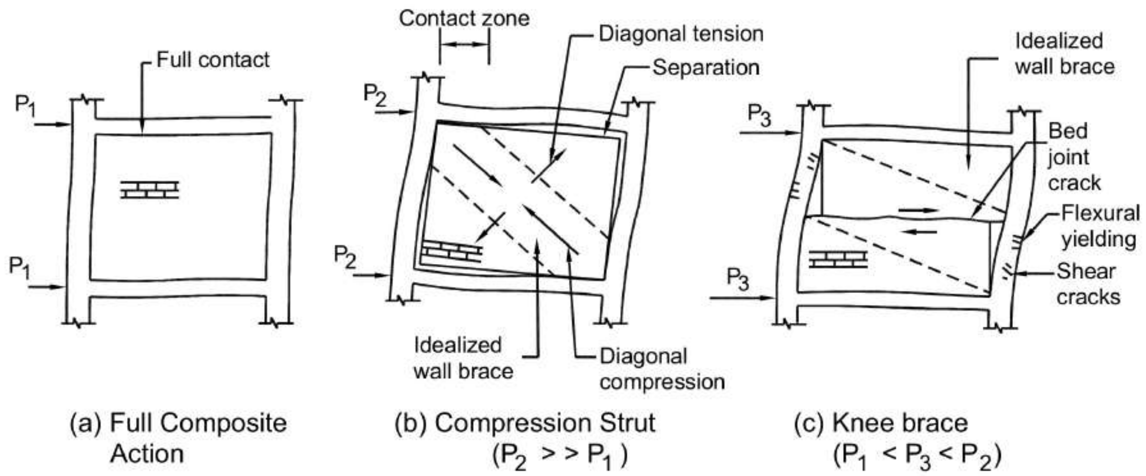
In-filled frame subjected to large acceleration



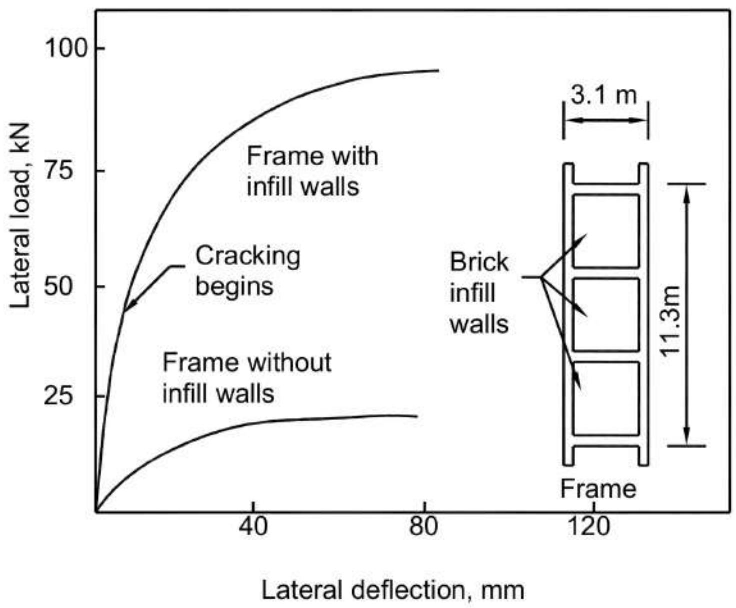
Strut action of stairs[41]

3.11 Why the consideration of infill wall in a multistoried MRF building is necessary for efficient seismic design?

Reinforced concrete frames infilled with masonry, form the structural system of many of these vulnerable buildings. The reinforced concrete frames have typically been designed for gravity loads, and common design practice considers the infill a non-structural components. By neglecting the masonry infill during design of the frame, one is assuming that the final infilled structure will have the same reliability as the frame alone. Such belief is vastly misleading since, infill alters the seismic response drastically and the structure shows the poor performance during seismic events. Therefore, the interaction between frame and infill must be considered.



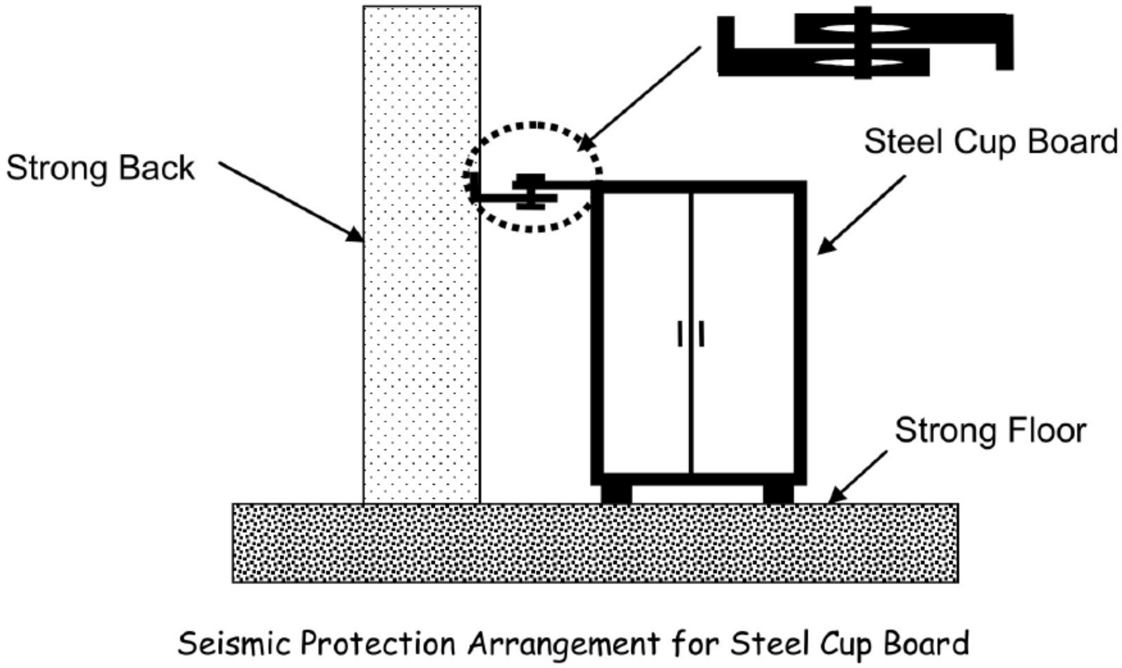
Deformation and failure of infilled frame during earthquake



Load-deformation characteristics of RC frame with and without infill wall[7]

3.12 Why the seismic safety of building content is a must in case of severe earthquakes?

Sometimes the building content may also pose the problem for life safety. Structure should be prevented from collapse. The response of a structure must be controlled to prevent heavy furniture and equipment from overturning on the floor. Hence the contents of a building should be properly fastened to the structure. It was reported in Japan that many persons were killed due to overturning of heavy furniture.



Seismic Protection Arrangement for Steel Cup Board

3.13 What is the influence of incorrect detailing of reinforcement on the seismic performance of a building?

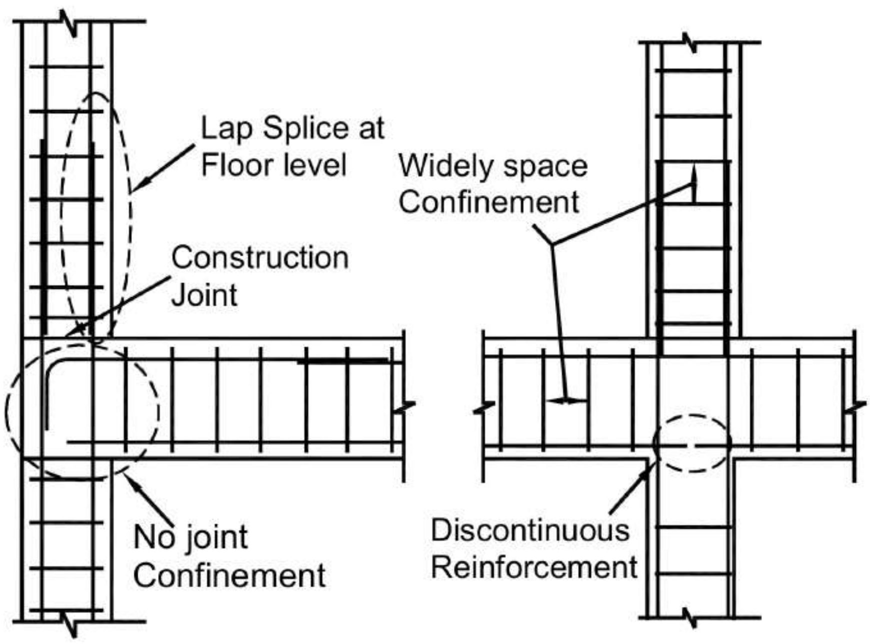
Inadequate transverse reinforcing bars: Transverse reinforcement provides resistance against shear forces and imparts confinement to concrete within. This confinement increases the ultimate strength of concrete, and enables a beam or column to accommodate more damage without failing catastrophically. It is especially important to avoid wide spacing of transverse bars near beam-column joints.

Short overlap lengths at spliced joints: These are a location where one of the reinforcing bars that runs longitudinally along, say, a column ends and overlaps with another that continue farther along the column. If the overlap is too short, the force in one bar cannot be adequately transferred to the next one, which produces a generally unanticipated weak section in the column.

Lack of column confinement in the central part of columns: If the shear reinforcement is not sufficient to prevent shear failure before flexural yielding at the column ends, a brittle failure of column occurs during earthquake.

Lack of column confinement and poor detailing practice: Most of the structural damage observed in frame buildings remains concentrated at column ends. Unfortunately, confinement reinforcement is virtually nonexistent in these members, leading to no ductility.

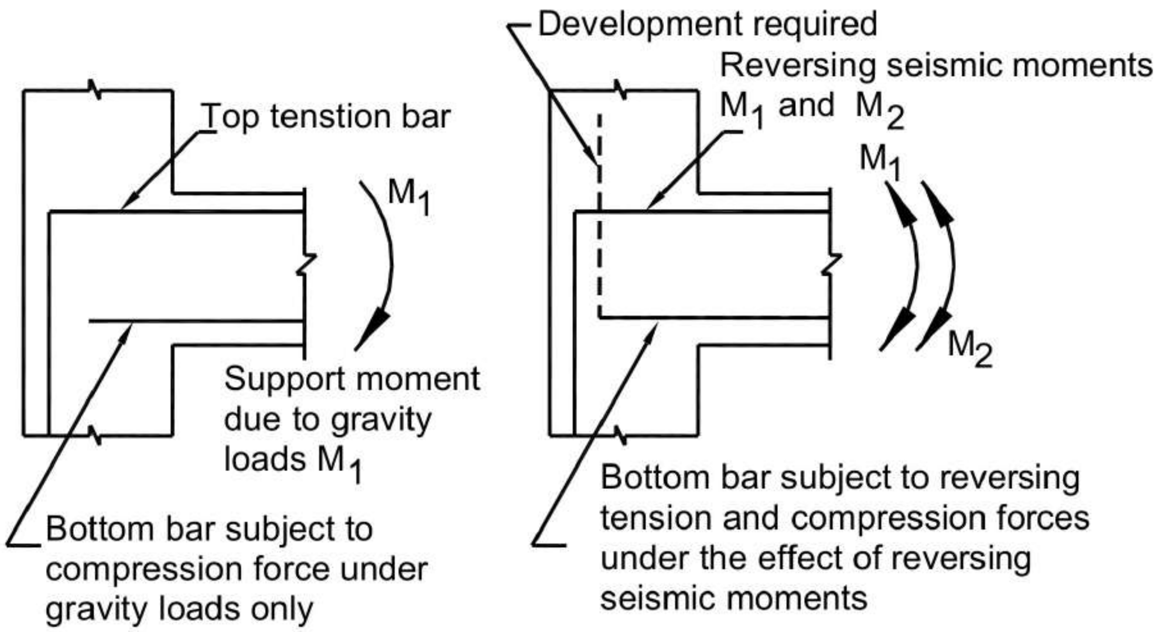
Splice Failure of Longitudinal Reinforcement: Longitudinal reinforcement is spliced in various ways, including lap splices, mechanical splices and welded splices. Splices should be located in a region where tensile stress is low. Splices in older buildings are located in regions of higher stress because the implications for earthquake performance are inadequately understood. Splice failure reduces flexural resistance of the member often before yielding



Incorrect detailing of Reinforcement[42]

Bond Splitting Failure: The bond stress acting on deformed bars cause ring tension to the surrounding concrete. High flexural bond stresses may exist in members with steep moment gradients along their lengths. If the longitudinal reinforcement of a beam or column is not supported by closely spaced stirrups or ties, splitting cracks may develop along the longitudinal reinforcement, especially when the strength of concrete is low, when large diameter longitudinal bars with high strength are used, or when the concrete cover on the deformed bars is thin. These splitting cracks result in loss of bond stress, limiting the lateral load carrying capacity at a small deformation.

The change in spacing of stirrups, wrong anchoring of longitudinal reinforcement of the columns into the beams, insufficient thickness of the concrete cover, inaccurate placing or lack of stirrups in the beam-column joints, bottom reinforcement of a beam often anchored straight in the beam-column joint are common detailing practice in non-seismic regions. These will not allow flexural yielding under earthquake loading.



Forces in the longitudinal bottom bar of the beam subject to gravity loads and reversing earthquake effects

3.14 What is the influence of construction deficiencies on the seismic response of an RC building?

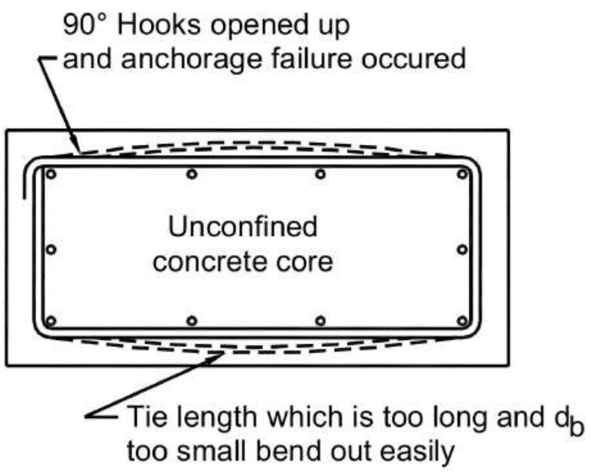
The quality of construction work affects the performance of a building, for example the material strength, the amount of reinforcement, lateral reinforcement, sufficient concrete cover etc.

Sometimes congested reinforcement due to (a) the use of small cross sectional area, (b) the use of lap splicing, and (c) the anchorage of beam reinforcement in the already-congested beam-column joints, may also reduce the seismic capacity of the structure.

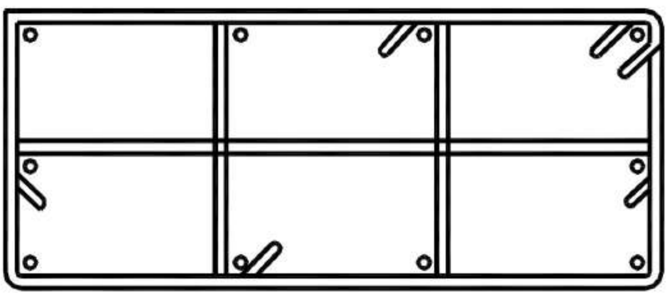
Eccentric Beam Column results in excessive localized damage to the column. It is estimated that more than 10% of the structures have eccentric beam-column joints.

Inadequate shear area in both principal directions of the building, when most of the shear resisting elements such as columns, shear walls are oriented in the same direction, and a few elements are oriented in the perpendicular direction, the structure appears strong in one direction and extremely weak in the perpendicular direction. Therefore, the shear resisting elements must be distributed uniformly to have equal stiffness in both the directions.

Building on Piles - High bending moments combined with axial forces acting at the top of a pile can cause crushing of concrete. Such damage is difficult to identify after an earthquake. Therefore, the flexural capacity of pile must be much higher to the columns so that the plastic hinges must be appeared in column rather than piles.



Deficiencies of tie arrangement caused longitudinal bars to buckles and column to fail



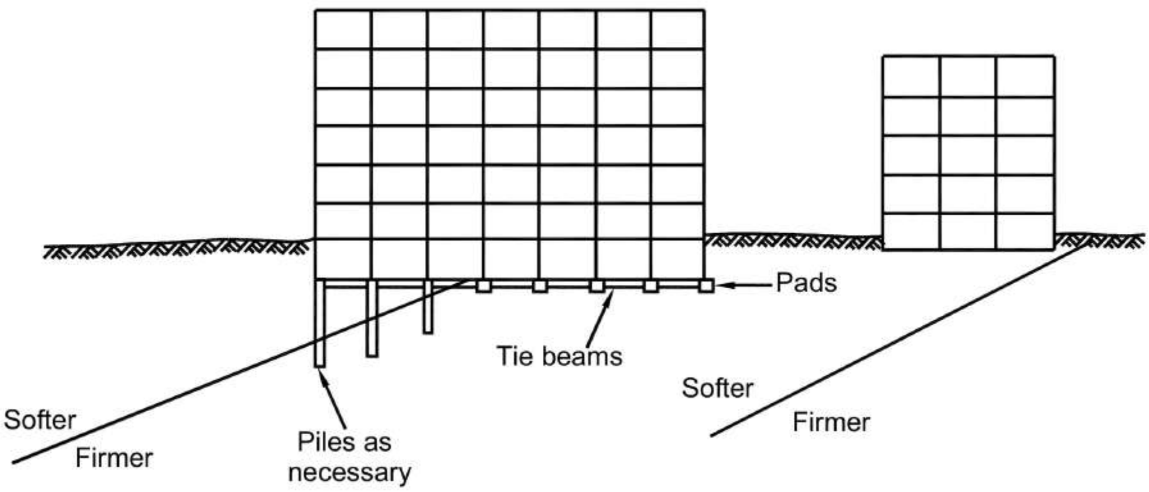
Proper tie arrangement to prevent buckling of longitudinal bars and confine concrete

3.15 How the poor constructional materials affect the performance of an RC building during earthquake?

A building must be constructed using specified materials, the arrangement of reinforcement, the concrete work etc. The quality of materials also affects the building's age. Proper maintenance is also essential. Deterioration of structural materials due to time and aggressive environmental conditions reduce the seismic performance potential of a building. Construction workers must be trained as well as educated to handle right materials and to execute the work properly.

3.16 Does the soil at construction site affect the performance of a building during earthquake?

During the seismic motion, when the radiating earthquake energy passes through the local soil, its effects are magnified, depending on the type of soil. Building and the local soil interact in reacting to the earthquake. This is, of course, reasonable. A building on rock or soft deep deposits cannot behave similar to the one on rocky and stiff soil. Soft and deep deposits magnify earthquake effects. This fact is reflected in seismic codes as the effect of types of soil. The difference between the peak horizontal acceleration at the firm ground (0.034g) and on the soft dried lake bed soil (0.168g) dramatically confirms this observation. The ground motion is amplified in a region of soft soil, such as old river beds or reclaimed land.



***Building on two types of soil requiring precautions against different settlement*[4]**

Chapter 4

Seismic Evaluation and Retrofitting of Existing Reinforced Concrete Buildings

4.1 What is seismic evaluation?

Seismic evaluation may be understood like the diagnosis of a person with or without ailment on which the direction and nature of treatment depends. Seismic evaluation implies determining the capacity of the structures to resist earthquakes.

4.2 Why the seismic evaluation of a building is a must?

The aim of seismic evaluation is to judge the seismic capacity and vulnerability of buildings during earthquake so that the amount of retrofitting may be determined. There are many deficient buildings, not meeting the current seismic requirements that may suffer extensive damage or even collapse if shaken by an earthquake. This may cause injury to occupants or people living in the vicinity. So, it is necessary to identify weak buildings and to evaluate their capacity for future earthquakes. If required, the buildings must be retrofitted.

4.3 What are the procedures for seismic evaluation?

Numerous methods of seismic evaluation of a building are available depending upon the objective of evaluation and the skill of the evaluator ranging from the visual examination to detailed structural analysis. Each has its own advantage, disadvantage and limitations. The most commonly used methods for the seismic evaluation are;

- **Rapid Visual Screening Procedure (RVSP)** – This method is used for short term evaluation of buildings especially at a time when the frightened occupants refuse to re-enter their houses after an earthquake unless they have been assured that the building is safe for future earthquake.

- **Simplified Evaluation Procedure** – ATC–14 (A Handbook for Seismic Evaluation of Existing Buildings and Supporting Documentations) is a document in which a simplified method of seismic evaluation of different types of buildings is recommended. However it has been modified by a number of Indian conditions and requirements and is available in different ways. The purpose of evaluation is to identify the buildings or building components that are risky to human lives.

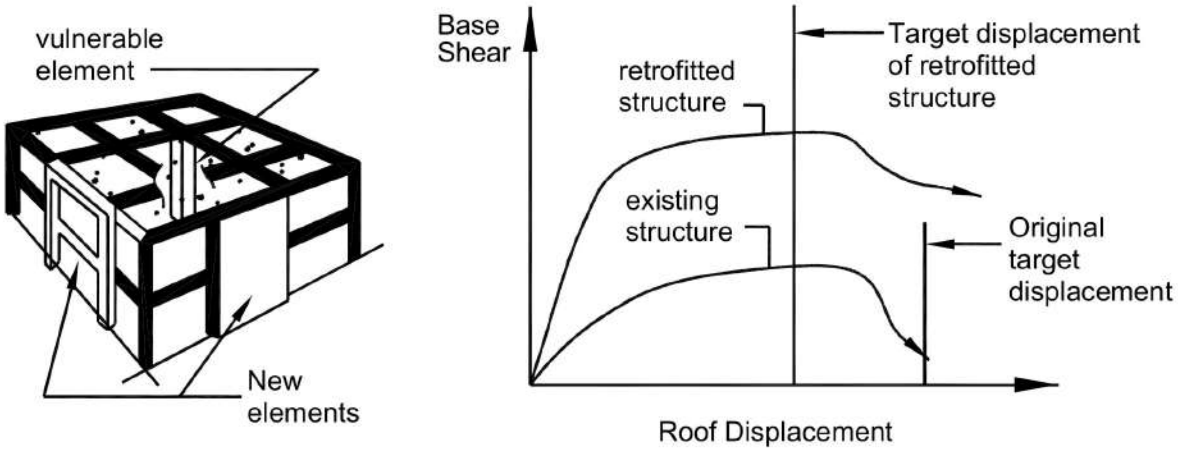
- **Visual Inspection Method** – It is generally used in earthquake damaged buildings. Based on the visual observation and modes of failure of building components, one estimates that the particular building may be used in future or not. Sometimes the help of Non Destructive Testing (NDT) may also be required.

- **Non-linear Static Pushover Procedure** – Non-linear Static Pushover Procedure is used by the design engineers to evaluate the seismic capacity of the building in terms of strength and ductility. It is very helpful to determine the amount of retrofitting required and to further re-evaluate the building after the retrofitting.

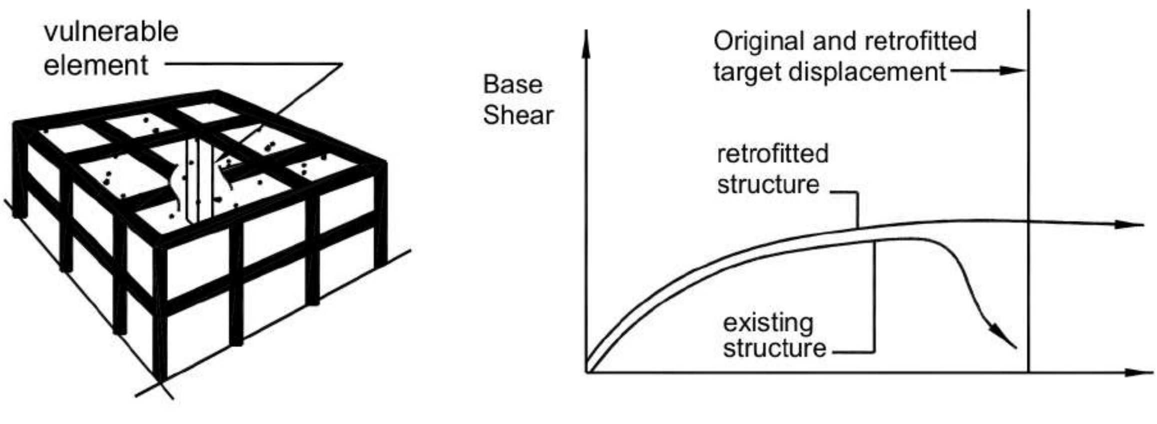
- **Non-linear Dynamic Time History Analysis** – This is a method to evaluate building considering dynamic behavior and non-linear material properties. But it requires huge computational effort and time with competent professional softwares. Therefore, most of the design engineers are generally not adopting this method of evaluation.

4.4 Why seismic retrofitting is required?

Past earthquakes have manifested that buildings with proper design and construction have borne the seismic shocks without collapse. But the structures either old or constructed without seismic design techniques have undergone serious damage or even collapse with an irreparable loss of innumerable lives. It has been seriously studied that if such buildings are modified to earthquake resistant structures by employing retrofitting techniques, they may be safely reused with no hazard to property and life safety. This also proves to be a better option catering to the economic considerations and immediate shelter problems rather than replacement of buildings. Moreover, retrofitting of buildings is generally more economical as compared to demolition and reconstruction even in the case of severe structural damage.



Global modification of the seismic deficient system



Local modification of the seismic deficient system [39, 31]

4.5 How the retrofitting is defined?

Seismic retrofitting includes concepts like system behavior improvement, components repair/strengthening up to expected performance i.e. minimum required strength and acceptable damage from an earthquake. Various terms like repair, strengthening, retrofitting, remolding, rehabilitation, reconstruction etc. are freely employed with a marginal difference;

Repairing Repairing suggests reconstruction or renewal of any part of a damaged or deteriorated building providing the same level of strength and ductility as was prior to the damage. Sometimes, Repair is also related to the seismic resistance of the building to its pre-earthquake state

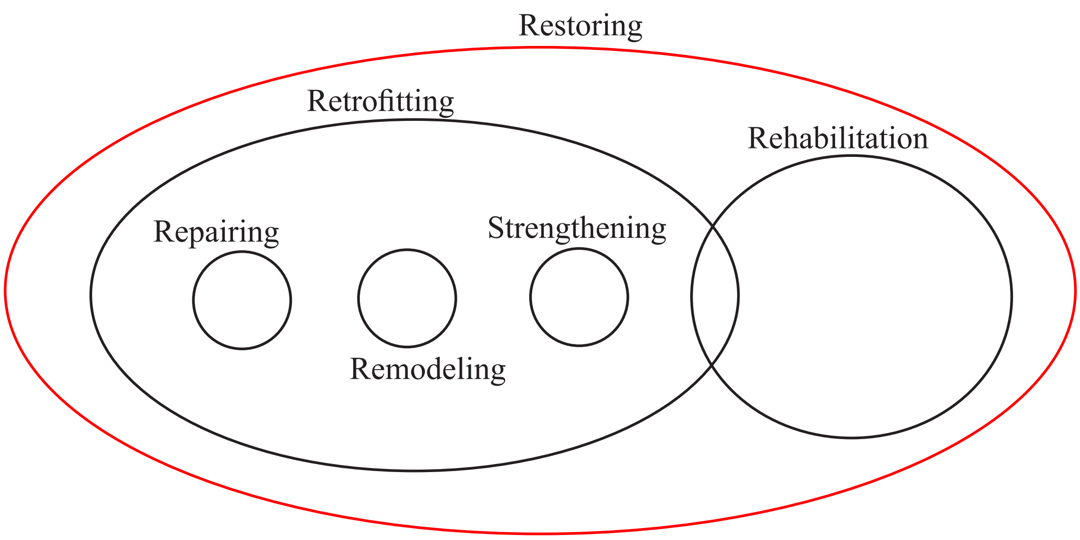
Retrofitting Retrofitting includes upgrading earthquake resistance of either an existing seismically deficient building or earthquake damaged building up to the level of the present day codes by appropriate techniques. Retrofitting also incorporates upgrading of certain building system, such as mechanical, electrical, or structural, to improve performance, function, or appearance.

Remolding Remolding means reconstruction or renewal of any part of an existing building owing to change of usage or occupancy

Rehabilitation Rehabilitation encompasses reconstruction or renewal of an earthquake damaged building to provide the level of function, prior to the damage. It also refers to increasing the seismic resistance of an existing seismically deficient building.

Restoration Rehabilitation of buildings in a certain area may be described as restoration

Strengthening Reconstruction or renewal of any part of an existing building to provide better structural capacity i.e. higher strength and ductility than the original building, is taken as strengthening. Sometimes the term strengthening and retrofitting are used simultaneously.



Van diagram for various terms used in retrofitting[6]

4.6 What are the causes that make a building seismically deficient?

For existing seismic deficient buildings

- The buildings have been designed according to a seismic code, but the code has been upgraded in the following years
- changes in codes over the past 50 years show that the design force levels have increased with each revision and the detailing requirements have been made more stringent
- Buildings are designed to meet the modern seismic codes, but the deficiencies exist in the design and /or construction;
- Existing reinforced concrete (RC) frame buildings with non-ductile detailing
- Essential buildings must be strengthened like hospitals, school & colleges, historical monuments and architectural buildings;
- Important buildings whose service is assumed to be essential even just after an earthquake;
- Buildings, the use of which has changed through the years;
- Buildings those are expanded, renovated or rebuilt.

For earthquake damaged buildings

- Immediate and long terms safety of the occupants since all the damaged buildings can not be replaced or rebuilt
- Economic consideration since the retrofitting, in general, is more cost efficient than reconstruction of buildings
- Important, historical, heritage buildings should be preserved at any cost

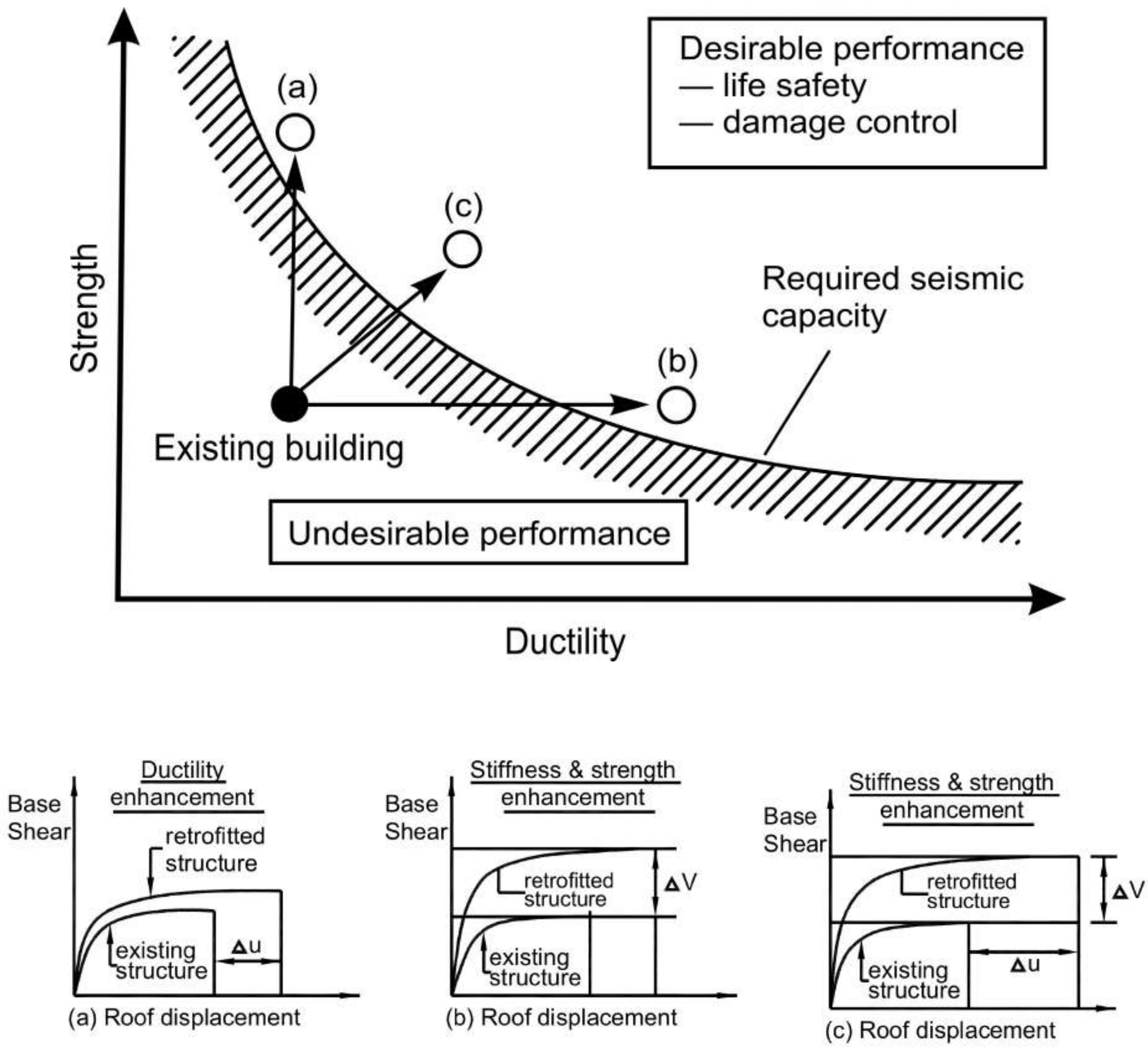
4.7 What are the problems associated with retrofitting?

The problems faced by a structural or field engineer in case of earthquake vulnerable or earthquake damaged buildings are

- Methods of seismic assessment or evaluation of existing buildings' capacity
- To obtain sufficient records of buildings such as architectural and structural drawings, structural design calculations, material properties, details of foundation and geo-technical reports, records of at least natural period of the buildings
- Retrofitting and issues of their structural safety
- Guidelines or Codes of Practice on retrofitting
- Issues related to costs, invasiveness and the requirement of specialist knowledge
- Socio-economic issues such as aesthetic and psychological assumptions
- Cost vs. importance of the structure, especially in the case that the building is of cultural and/or historical interest
- The available workmanship and the level of quality control
- The duration of work/disruption of use and the disruption to occupants
- The functional and aesthetic compatibility of the retrofitting scheme
- Selection of the type and level of intervention
- Repair materials and technology available
- Controlled damage to non-structural components
- Sufficient capacity of the foundation system is essential
- To counter the irregularities of stiffness, strength and ductility

4.8 What are the concepts of retrofitting?

Aim of retrofitting is to (i) upgrade of the lateral strength of the structure (ii) increase the ductility of structure (iii) increase in strength and ductility



Aim of seismic strengthening or retrofitting[39]

4.9 What is the classification of retrofitting techniques?

Two alternative approaches are conceptually adopted and implemented in practice for seismic retrofitting: the first approach focuses on reduction of earthquake induced forces (i.e. modifying the demand) and the second focuses on upgrading the structure to resist earthquake induced forces (i.e. modifying the capacity). While applying the first approach, base isolation or damping devices are commonly applied to the structure. An upgraded structural capacity is achieved either by intervening on specific elements or by changing the load paths within the structure[38].

Structural Level (or Global) Retrofit Methods
Structural-level approach of retrofitting which involves global modifications to the structural system

Conventional Methods (modifying the capacity)

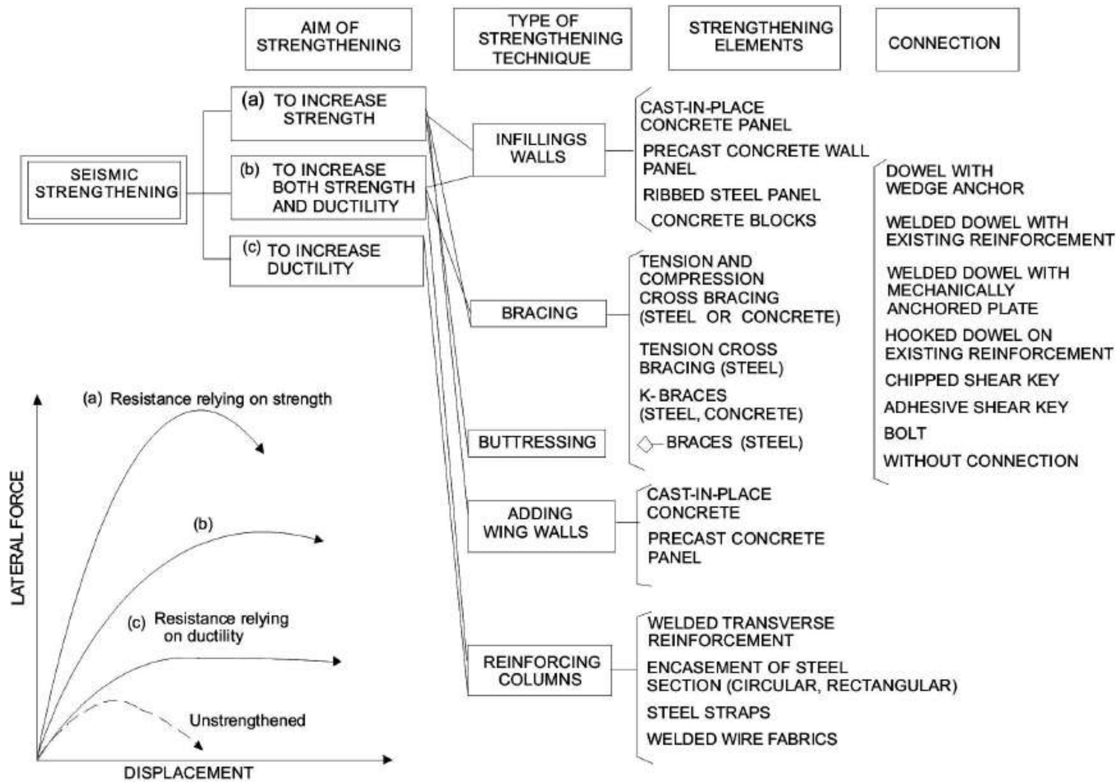
- Adding New Shear Walls into/onto the Existing Frames
- Adding Steel Bracing into/onto the Existing Frame
- Adding Infill Walls into/onto the Existing Frames

Non-Conventional Approach (modifying the demand)

- Seismic Base Isolation
- Supplemental Damping Devices

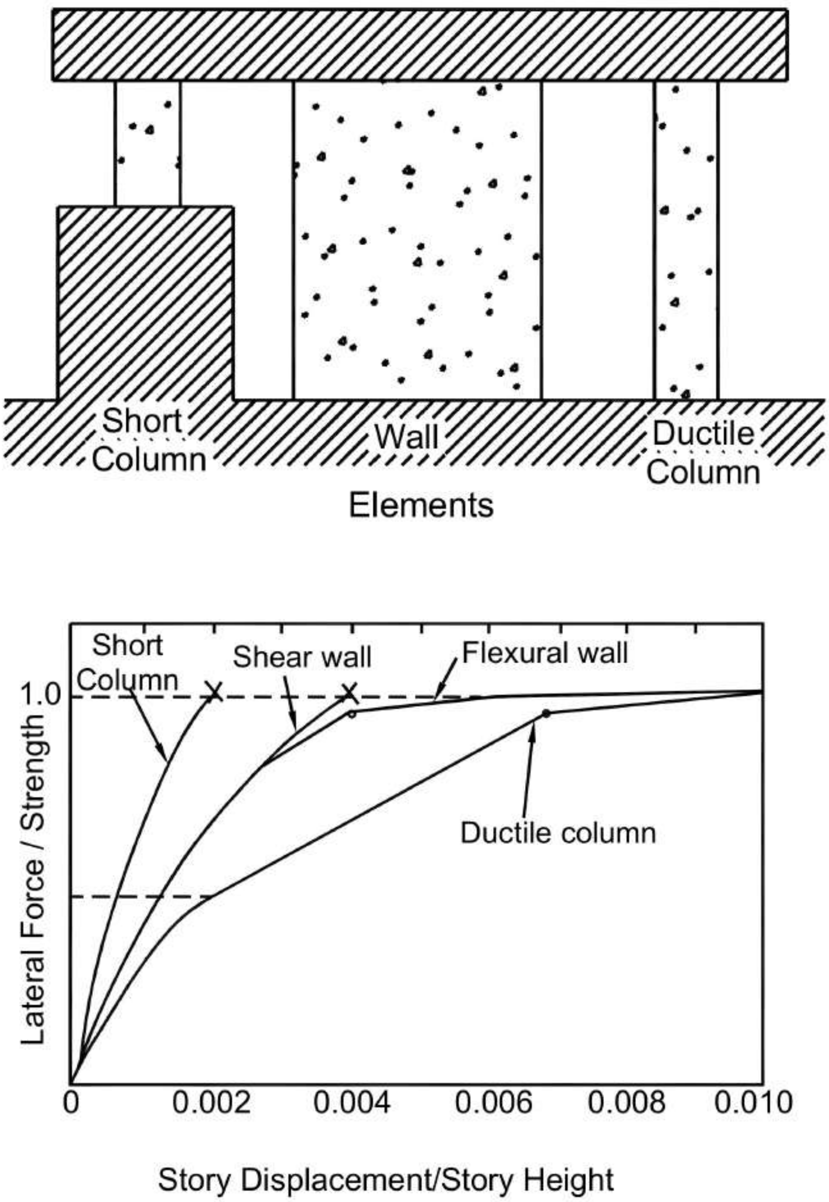
Member Level (or Local) Retrofit Methods

Member level approach of retrofitting or local retrofitting of components with adequate capacities to satisfy their specific limit states. It includes jacketing/confinement of Columns, Beam, Beam-Column Joint, Slab, Foundations etc.



4.10 What are the considerations in retrofitting of buildings?

Seismic retrofitting is constrained to certain areas of an existing building. In such cases, the retrofitted structure is a "behavioral hybrid" system consisting of strong/ductile components (added elements) and of weak/brittle components (elements that are not strengthened). In the event of an earthquake, all components at each floor, retrofitted or not, will undergo the same lateral displacements. While modified or added elements can be designed to sustain these lateral deformations, the remaining non-strengthened elements could still suffer substantial damage. Therefore, it is suggested that the design of retrofitted schemes should be based on drift control rather than on strength consideration alone and should be able to predict initial and final stiffness of the retrofitted structure.



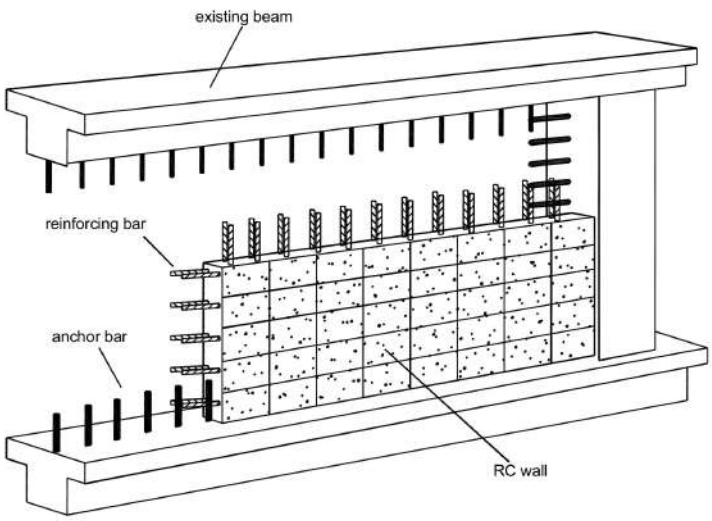
Idealized storey of a building and load-displacement relationship of elements [30,31]

4.11 What are considerations for retrofitting of a building using shear walls into/onto the existing frames?

Addition of new RC walls is the most common method for strengthening of existing structures. Special consideration is needed to the distribution of the walls in plan and elevation (to achieve a regular building configuration), transfer of inertial forces to the walls through floor diaphragms, integration and connection of the wall into the existing frame buildings and transfer of loads to the foundations. Added walls are typically designed and detailed as in new structures which are designed to carry all or most of the lateral force. These systems simply bypass the existing inadequate system and the need to correct all or most of the deficiencies in the old structure. The main difficulty with this retrofitting scheme is that the lateral forces are concentrated in certain areas where shear walls are added and these walls impose large forces on the foundation. Therefore, new foundations or strengthening of the existing foundations may be required to resist the increased overturning moment and the increased dead load of the structure. Foundation intervention is usually costly and quite disruptive, thus rendering the application of this technique is unsuitable for buildings without an existing adequate foundation system. This could be inconvenient in cases where the strengthening project, or when there is not enough or no information on the original foundation design.

Limitations

- Increase in lateral resistance but it is concentrated at a few places
- Increased overturning moment at foundation causes very high uplifting that needs either new foundations or strengthening of the existing foundations
- Increased dead load of the structure
- Excessive destruction at each floor level results in functional disability of the buildings
- Possibilities of adequate attachment between the new walls and the existing structure
- Closing of formerly open spaces can have major negative impact on the interior of the building uses or exterior appearance



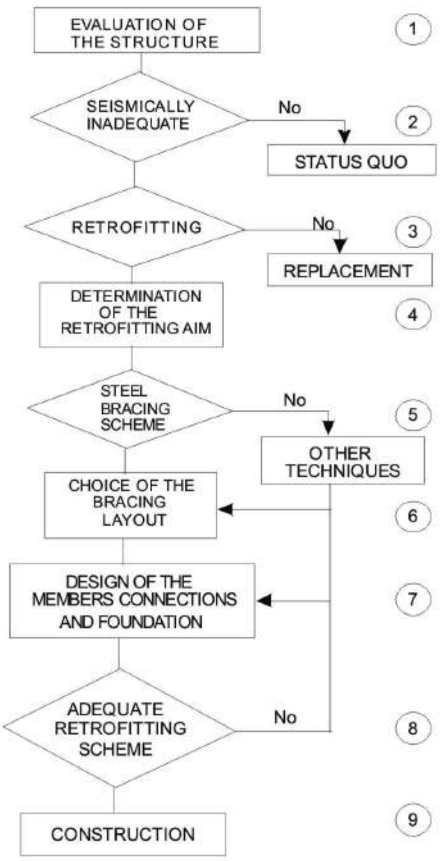
Details for addition of a shear wall[31]

4.12 How far steel bracing is effective as retrofitted technique?

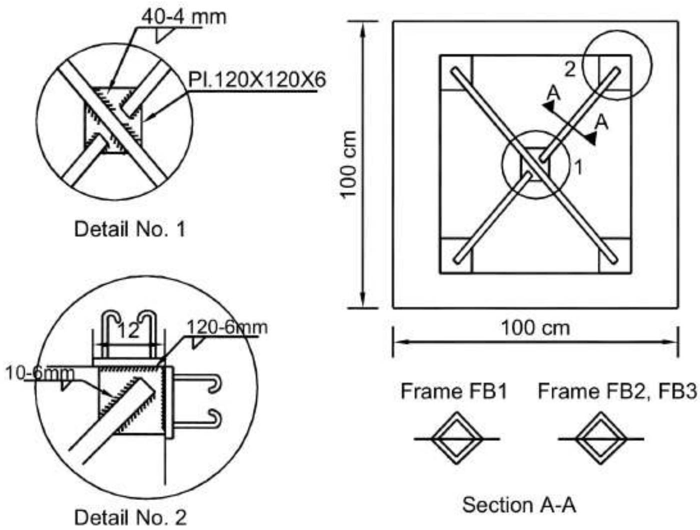
Steel bracing is commonly used in RC framed buildings if heavy reinforced concrete shear walls cannot be supported by existing foundation. Steel bracing can be a very effective method for global strengthening of buildings. Some of the advantages of steel bracing over the shear wall are the ability to accommodate openings, construction work can be performed externally to the building to minimize disruption of the occupants and increase speed of work; small increase in mass, as a result foundation and construction costs may be minimized. But care must be taken to produce a final design that is structurally well balanced i.e. no stiffness irregularity in plan or along the height. Coordination between building owner, engineer and architect is essential to satisfy architectural and functional requirements.

Limitations

- Lack of information on the seismic behavior of the added bracing
- The connection between an existing framing system and added elements should be carefully detailed because forces must be transferred between the existing and added structural system.
- A moderate to high level skilled labor is necessary for construction, due to the need for member fit-up adjustment and welding.
- Close quality control particularly with respect to welding is essential.



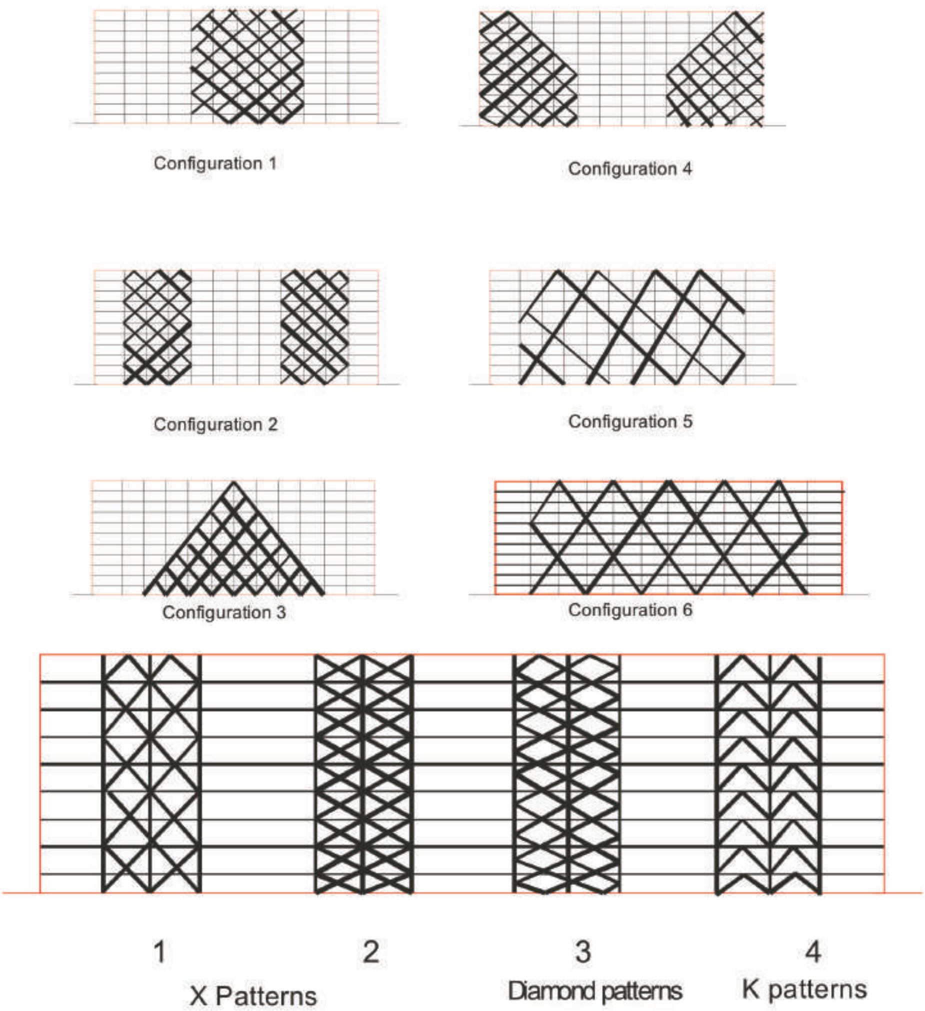
***Flow chart for retrofitting with bracing system*[32]**



***Detail of the X-bracing system and its connection to the RC frame*[32,33]**

The steel bracing may be applied either externally or internally. In the external bracing system, steel bracing systems are attached to the exterior frames which create minimum disruption to the function of the building and its occupants and also provide better feeling of security. In case of internal bracing, the buildings are retrofitted by incorporating a bracing system inside the individual bays of the RC frames. The bracing may be attached to the RC frame either indirectly or directly. Different forms of steel bracing such as X, V and K may be used. Cable braces may also be used on a low-rise structure. The cables could be added quickly and with minimal disruption to the occupants. In addition, the cable braces required minimal modification to existing structure. End blocks are provided to hold the cable anchorages at the foundation level and at the roof.

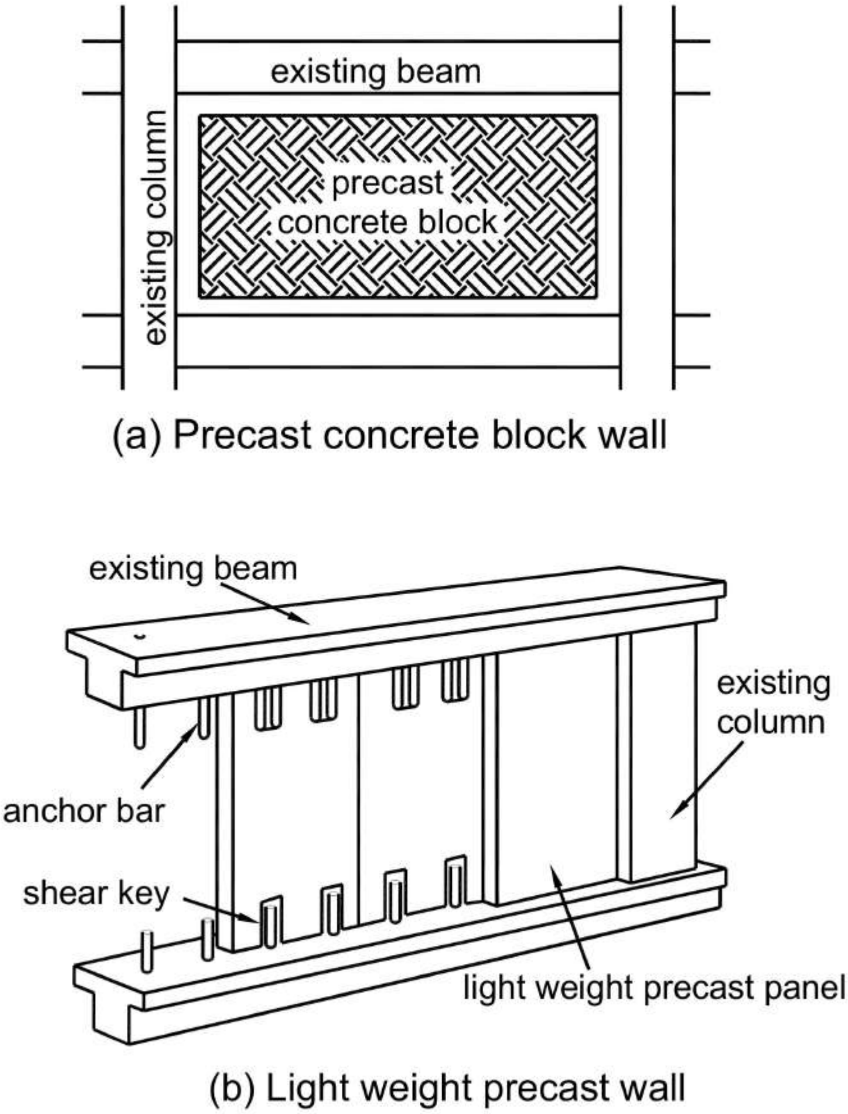
The bracing system should be designed for elastic response, but detailed for ductile behavior. It is desirable to limit the effective slenderness ratio of the braces to 100, and preferably to 80 to limit inelastic buckling therefore it is effective in compression as well as tension. Such bracing members have been shown to exhibit better- absorbing characteristics than braces with higher effective slenderness ratios. Therefore the designer must control the slenderness ratio by selecting the bracing layout and the brace section. Inelastic buckling of the braces is the main problem in achieving good hysteretic ductility.



Bracing Patterns

4.13 What are considerations for retrofitting a building by infill walls into/onto the existing frames?

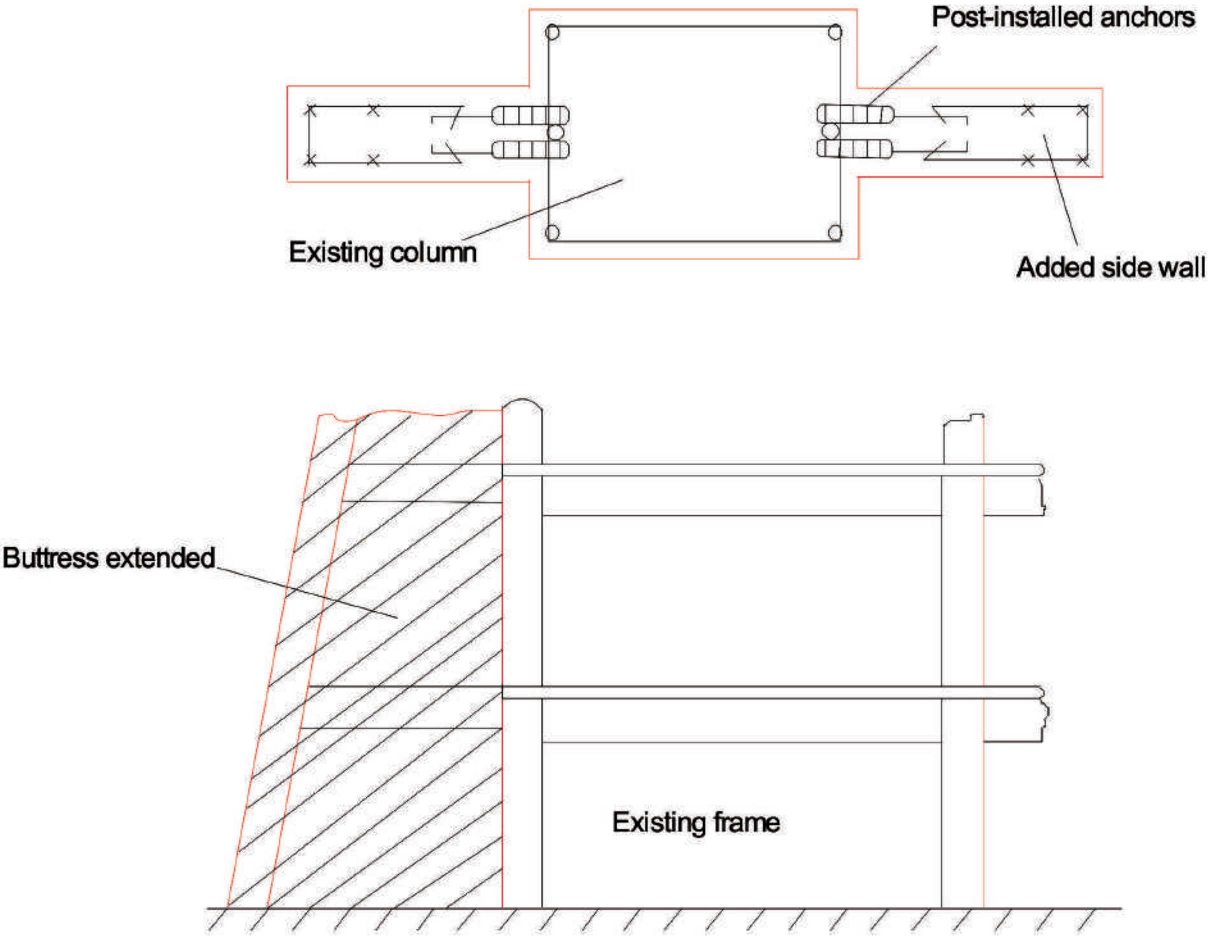
A convenient way to introduce infill walls may be by partial or full infilling of strategically selected bays of the existing frame. The infill wall may be provided in a bay consisting of beams and the two columns and the latter acting as its boundary elements. In case of increasing the capacity of existing infill shotcrete is normally used. Pre-cast panels may also be a good alternative in place of cast – in –place infill walls. These walls, well anchored into the surrounding frame with various types of connections (e.g., shear keys, dowels, chemical anchors etc.), not only increase the lateral stiffness of the building significantly, but also relieve the existing non-ductile frames from being subjected to large lateral force demands.



Addition of precast infill wall and its connection details[31]

4.14 How far external buttresses help to reduce the seismic vulnerability of an RC building?

External buttresses increase the lateral resistance of the structure as a whole. It requires a new foundation system. There are two most intricate problems in retrofitting; (i) the buttress stability (ii) the connections between the buttresses and of the building. The buttress should be connected to the floors and columns at all levels to ensure full interaction and resisting the lateral loads. The connection area will be subjected to unusual levels of stresses that require special attention.



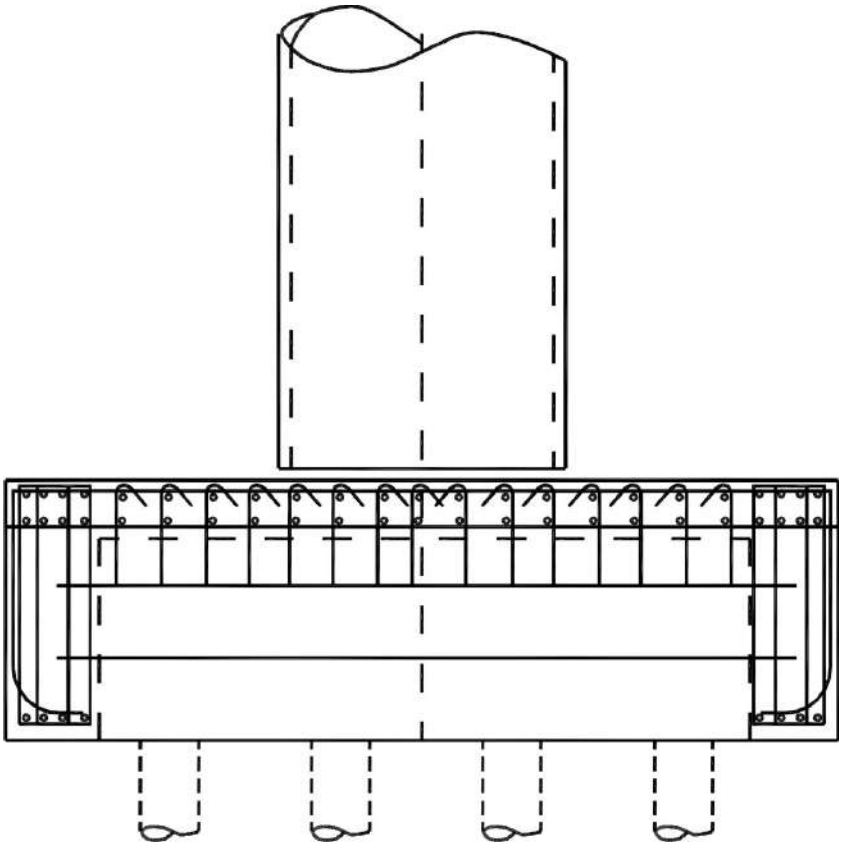
External buttresses to increase the lateral strength of frame[38,31]

4.15 How can seismic capacity of a foundation system be increased?

An existing foundation is strengthened for certain justified reasons (i) seismic upgrading of the super-structure; (ii) structural requirements; (iii) capacity design principles; (iv) parameters such as soil conditions and soil-structure interaction.

Retrofitting strategies aim at strengthening the existing foundation system and/ or adding supplemental foundation elements (footings or piles). Larger spread footings can distribute the load and additional reinforcement can increase their shear and bending resistance. The incorporation of existing footings into grade beams or mats is another option.

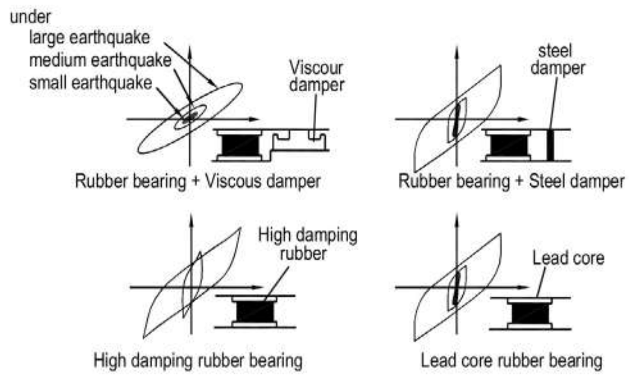
The addition of grade beams or increased size of spread footings usually requires excavation and there are difficulties in pinning or attaching the existing footings to the new elements. Piles may be added to improve the overturning resistance. Adding piles along the perimeter of the building can be an easier task from an economical and constructional point of view. The cost varies depending on the type and the level of intervention. In cases where piles have to be installed in the existing system the cost may dominate the total seismic retrofitting project.



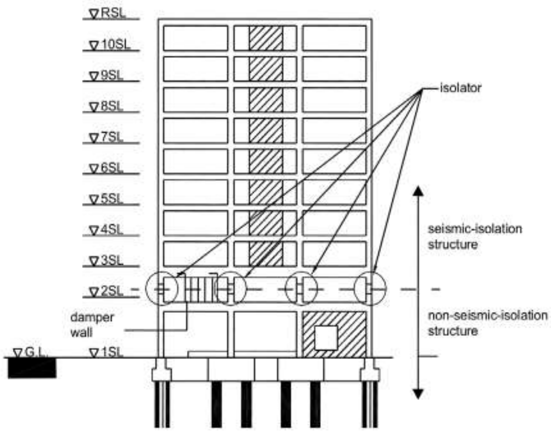
***Strengthening of existing foundation system*[31]**

4.16 What is seismic base isolation technique and how can it be used in retrofitting of buildings?

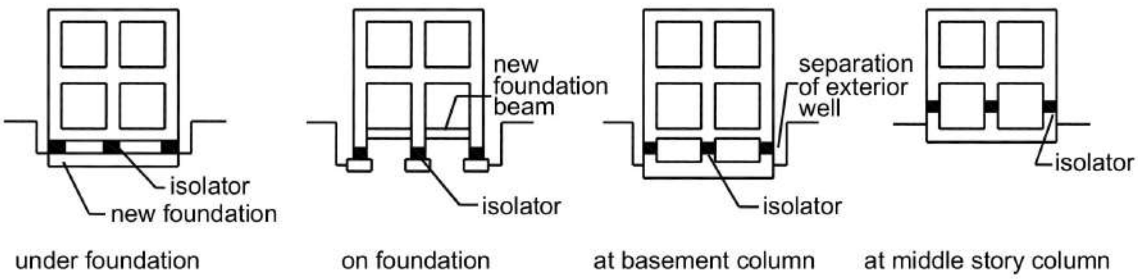
Seismic base isolation is recommended for retrofitting of critical or essential facilities, buildings with expensive and valuable contents and structures. It significantly reduces the seismic impact on the building and assemblies. Seismic isolation involves the insertion of flexible or sliding bearings at one level of a building. The isolation devices are inserted at the bottom or at the top of the first floor columns. Seismic isolation is more appropriate for buildings of historic significance because it may be applied without much disturbance to the historical architectural features. However, inserting an isolator within an existing column is not so simple because of the necessity of cutting the element, temporarily supporting the weight of the above structure, putting in place the isolators and then giving back the load to the column, without causing damages to persons and to structural and non-structural elements



***Isolation bearing and lateral force - displacement relationship*[31]**



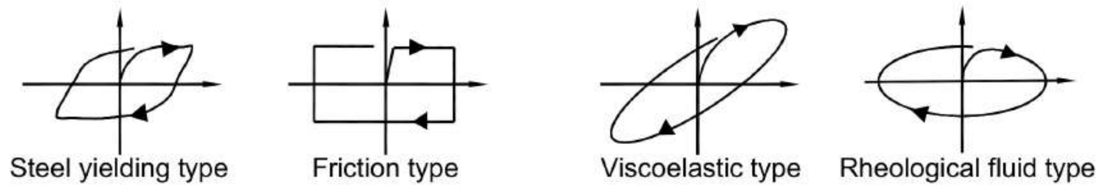
***Insertion of isolation at the intermediate storey*[31]**



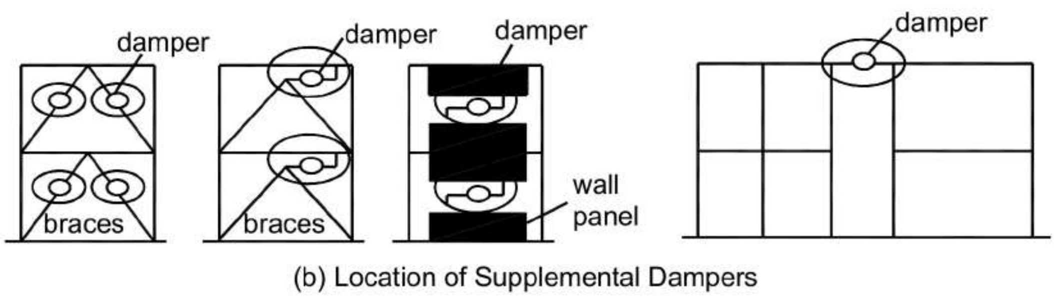
***Possible locations of isolations in a building*[31]**

4.17 What are energy dissipating devices and how they can be used in retrofitting of buildings?

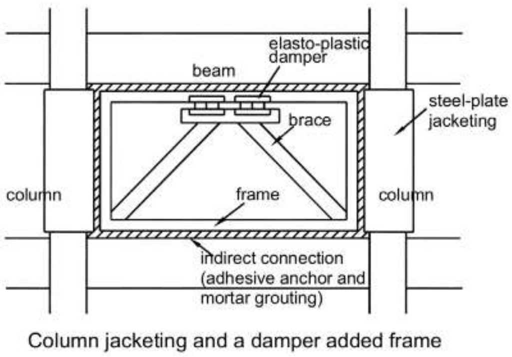
Energy dissipation devices can be inserted in an existing structure to reduce seismic demand through damping. Visco-elastic fluid dampers, Visco-elastic solid dampers, hysteretic energy dissipating dampers and friction dampers are various types of energy dissipating devices. These devices are also called Added Damping and Stiffness (ADAS) structural elements. They are mechanical devices, practically installed in structures in order to (i) substantially increase the overall damping in a structure, and (ii) to increase the overall stiffness of a structure. ADAS elements are ideally installed in flexible moment-frame structural system to economically achieve a moderately stiff and highly damped building system. It should be noted that most energy dissipating devices become effective with deformation in the devices.



Types of dampers and idealized load displacement relationships[31]



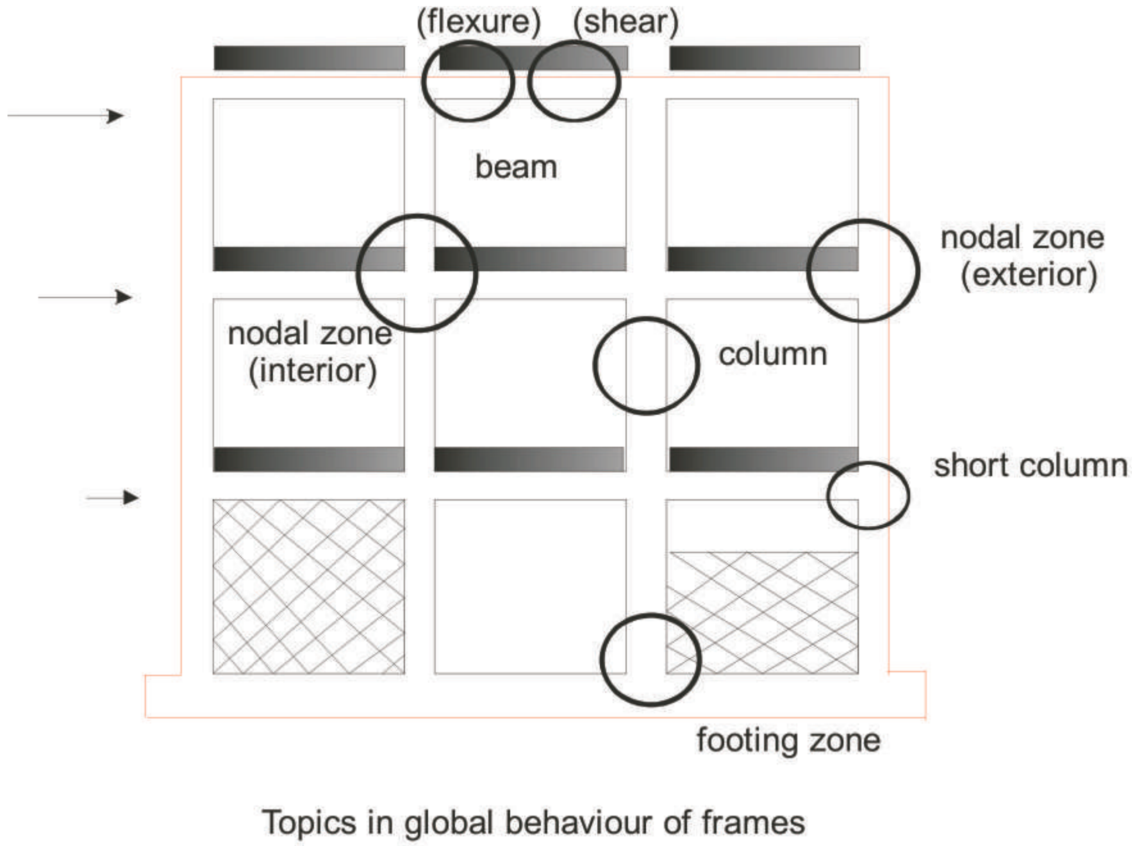
Supplemental damping system used for seismic retrofitting[31]



Column jacketing and a damper added frame[31]

4.18 What are the member level (or local) retrofit methods?

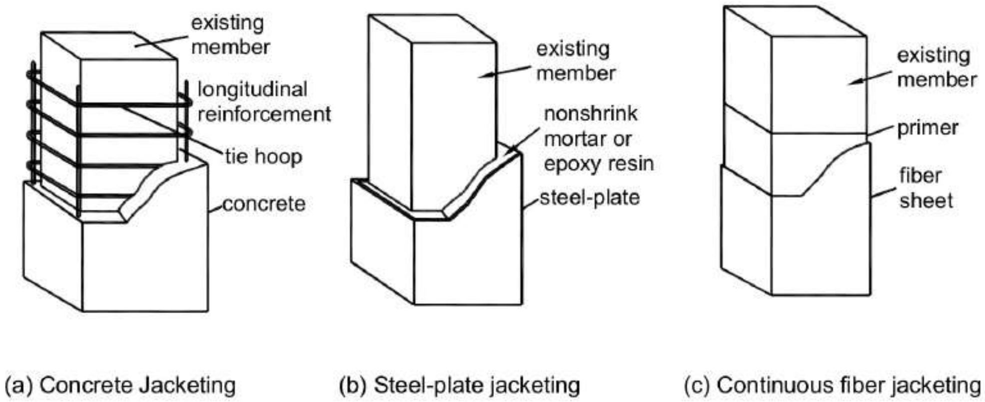
To strengthen individual components, structural members and their connections can be retrofitted and/or strengthened by reinforced concrete or steel jacketing, or by fiber reinforced plastic (FRP) or carbon fiber wrapping called local retrofitting. The local modification of isolated components of the structural and non-structural system aims to increase the deformation capacity of deficient components. Local intervention techniques are applied to a group of members that suffer from structural deficiencies and a combination of these techniques may be used in order to obtain the desired behavior for a seismically designed structure.



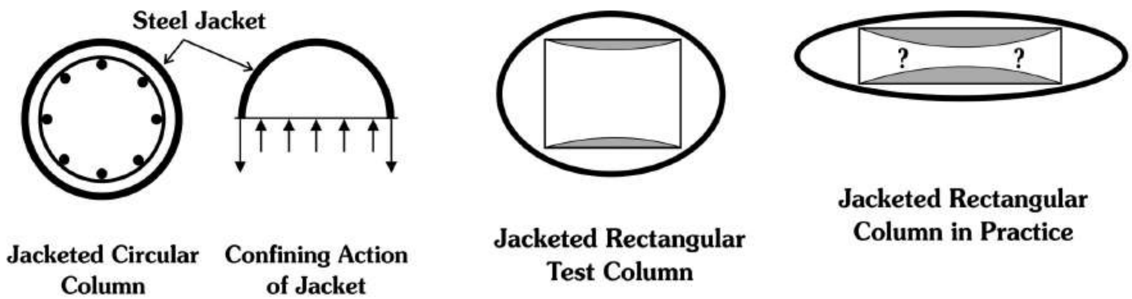
Member (or Local) retrofitting aims to improve the seismic deficiency of a member or connection of frame building[37]

4.19 What is jacketing and how it is effective to increase the strength and ductility of structural members?

Jacketing is most oftenly used and one of the most popular methods for strengthening of a deficient structural member. The most common types of jacketing are reinforced concrete (RC) jacketing, steel jacketing, fiber reinforced polymer (FRP) composite jacketing, jacketing with high tensile materials like carbon fiber, glass fiber etc. The main purposes of jacketing are: (i) to increase concrete confinement by transverse fiber/ reinforcement, especially for circular cross-sectional member (ii) to increase shear strength by transverse fiber/ reinforcement. Transverse fiber should be wrapped all around the entire circumference of the members possessing close loops sufficiently overlapped or welded in order to increase concrete confinement and shear strength. Jacketing of circular cross-section is more effective as compared to rectangular cross-section. Where square or rectangular cross-sections are to be jacketed, circular/oval/elliptical jacketing are most often used and the space between the jacket and column is filled with concrete. Such types of multi-shaped jackets provide a high degree of confinement by virtue of their shape to the splice region proving to be more effective. However, circular and oval jackets may be less desirable due to (i) need of large space, (ii) where an oval or elliptical jacket has sufficient stiffness to confine the concrete along the long dimension of the cross-section, is open to question.



Jacketing techniques[3]

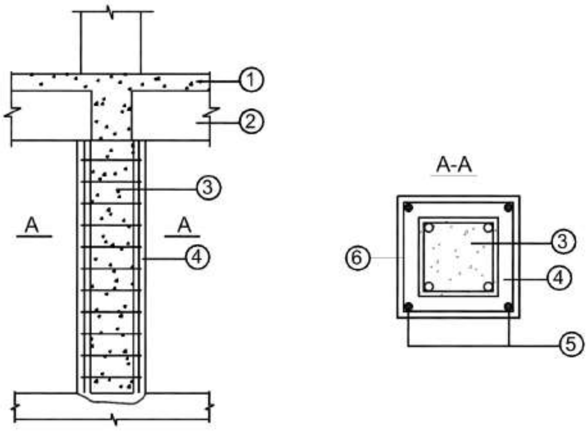


Notes: Shading shows areas of reduced confinement effectiveness

Various shapes of retrofitting jackets[36]

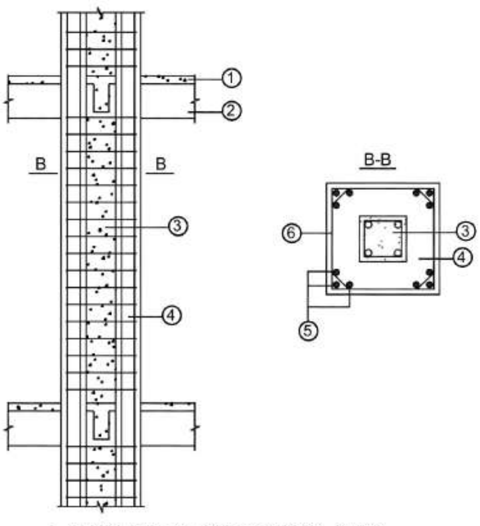
4.20 What are types of reinforced concrete (RC) jacketing of columns?

RC jacketing is applied for the rehabilitation of concrete members. If the longitudinal reinforcement placed in the jacket passes through holes drilled in the slab and new concrete is placed in the beam-column joint, it is categorized as global intervention which improves flexural strength of the column but strength of the beam-column joints remains the same. If the longitudinal reinforcement stops at the floor level then RC jacketing is considered as a member intervention technique which improves the axial and shear strength of the column. It has several advantages like avoiding the concentrations of lateral load resistance; no major changes in the original geometry of building; the original function of the building can be maintained. However there are some disadvantages. The presence of beams may require most of the new longitudinal bars in the jacket to be bundled into the corners of the jacket; it is difficult to provide cross ties for the new longitudinal bars which are not at the corners of the jacket; lack of guidelines.



1 - slab; 2- beam; 3 - existing column; 4 - jacket;
5 - added longitudinal reinforcement; 6 - added ties

***Jacketing of column for increasing shear and axial strength*[26]**

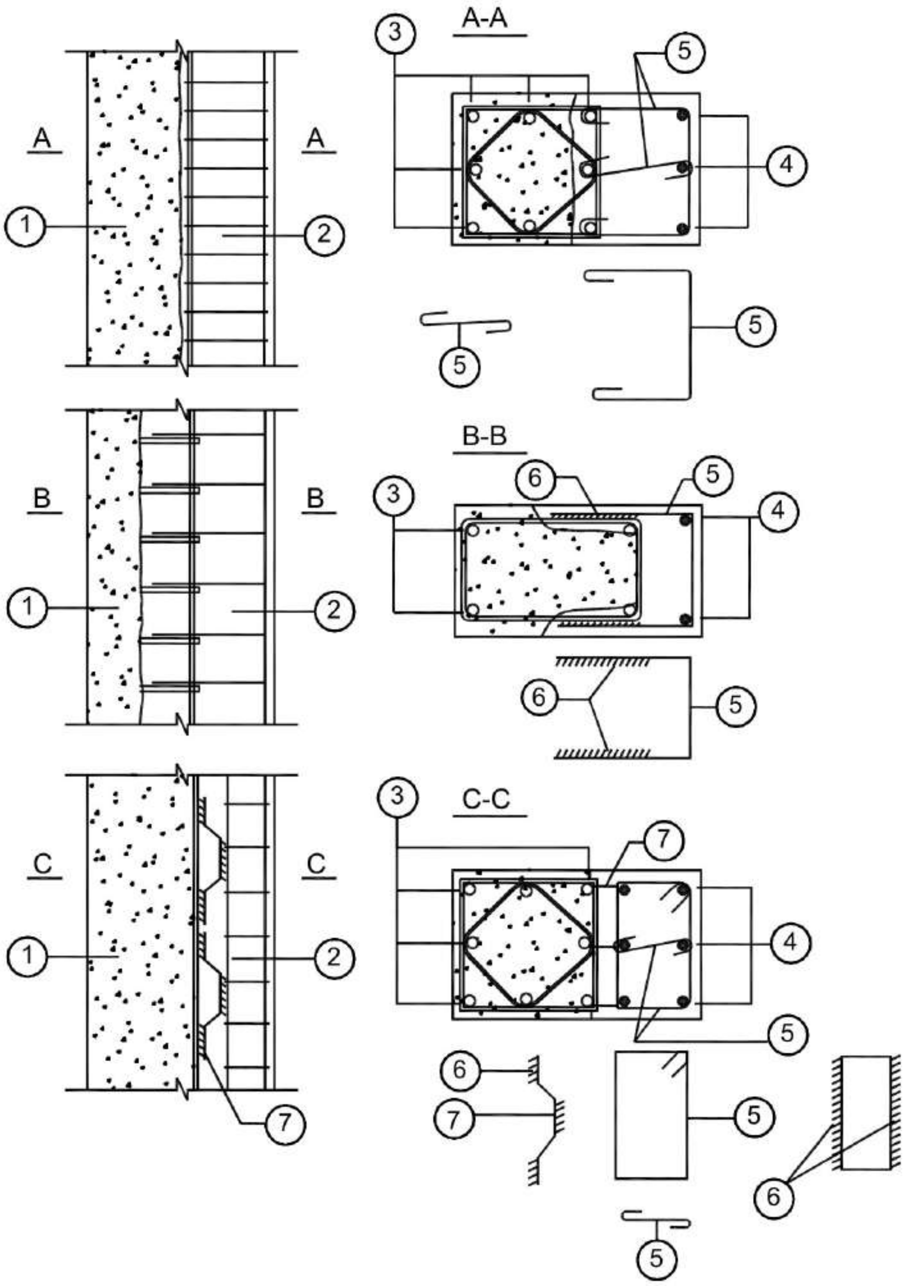


1 - slab; 2- beam; 3 - existing column; 4 - jacket;
5 - added longitudinal reinforcement; 6 - added ties

***Jacketing of column for increasing flexure, shear and axial strength*[26]**

4.21 What are the constructional details of one sided reinforced concrete (RC) jacketing of existing columns?

In case of one sided jacketing of existing column, an adequate connection of new added reinforcement with the existing reinforcement of the column is necessary by providing good detailing and closely spaced, well anchored, additional transverse reinforcement. A number of options are as follows;

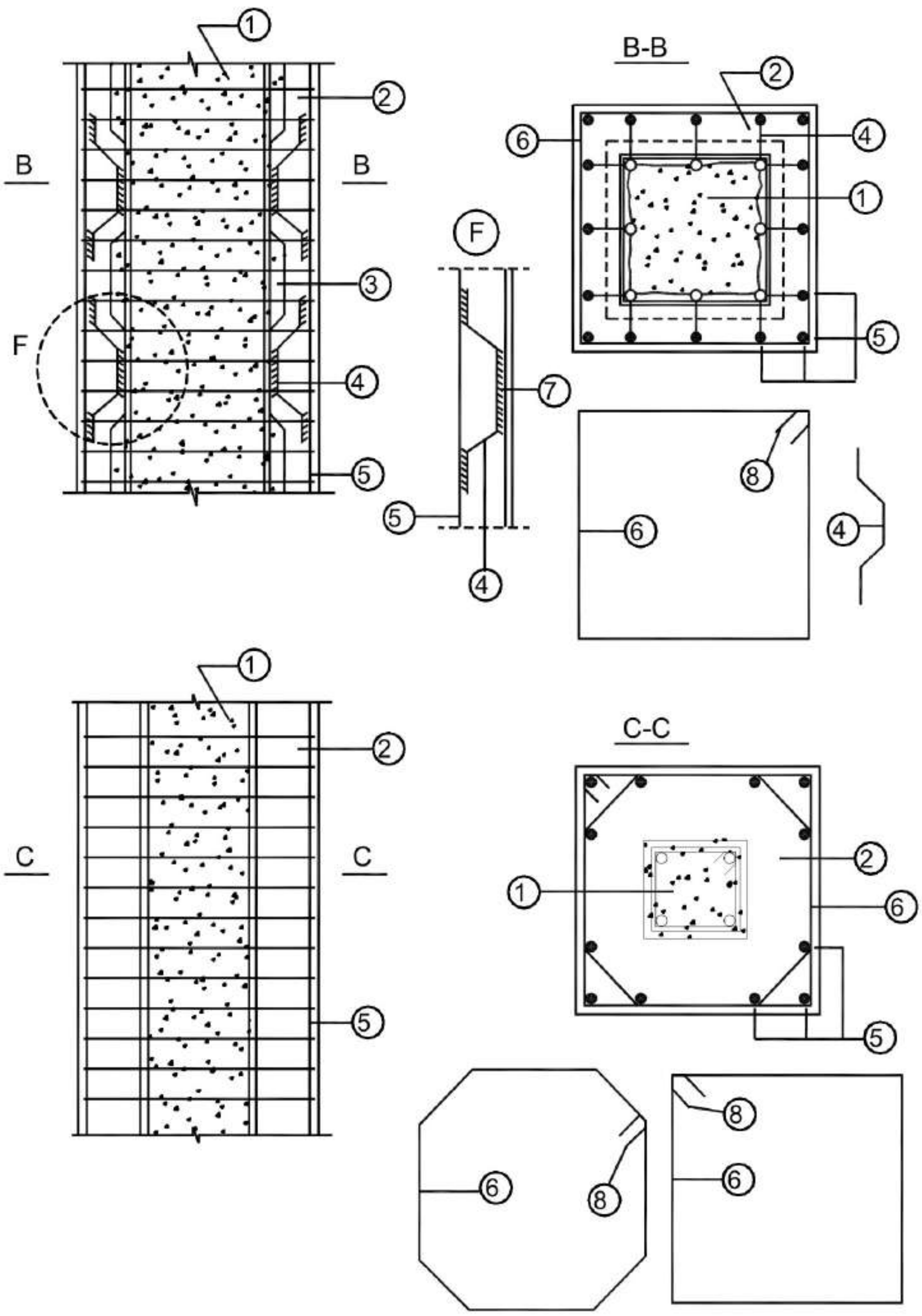


1 - existing column; 2 - jacket; 3 - existing reinforcement;
4 - added longitudinal reinforcement; 5 - added ties;
6 - welding; 7 - bent bars

Details of options in case of one, two or three sided jacketing[26]

4.22 What are the constructional details of four sided reinforced concrete (RC) jacketing of columns?

The details of four sided RC jacketing of an existing column may be provided as follows. However, there are other options may also be possible on the similar fashion.

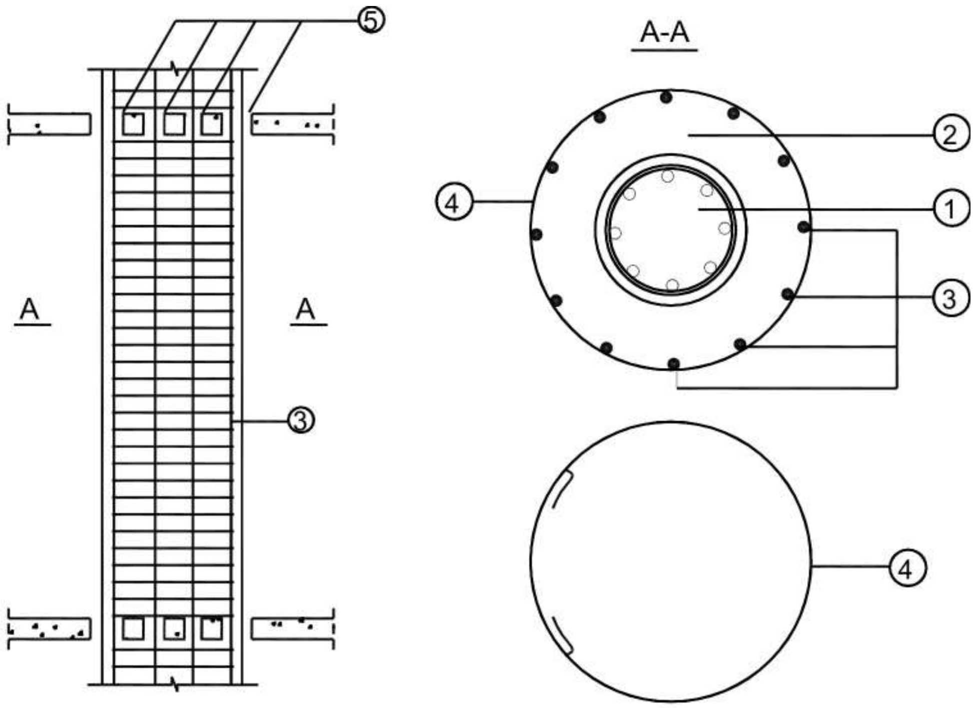


1 - existing column; 2 - jacket; 3 - key; 4 - bent bars; 5 - added reinforcement; 6 - ties; 7 - welding; 8 - alternative corners

Details of options in case of four sided jacketing[26]

4.23 What are the constructional details in reinforced concrete (RC) jacketing of circular columns without beams?

The jacketing of circular columns in case of flat slab (beamless) or has relatively narrow beam, can be achieved by adding longitudinal reinforcement through drilled hole in the slab if possible. Special attention must be paid to the adequate anchorage and splicing of the column ties or hoops.

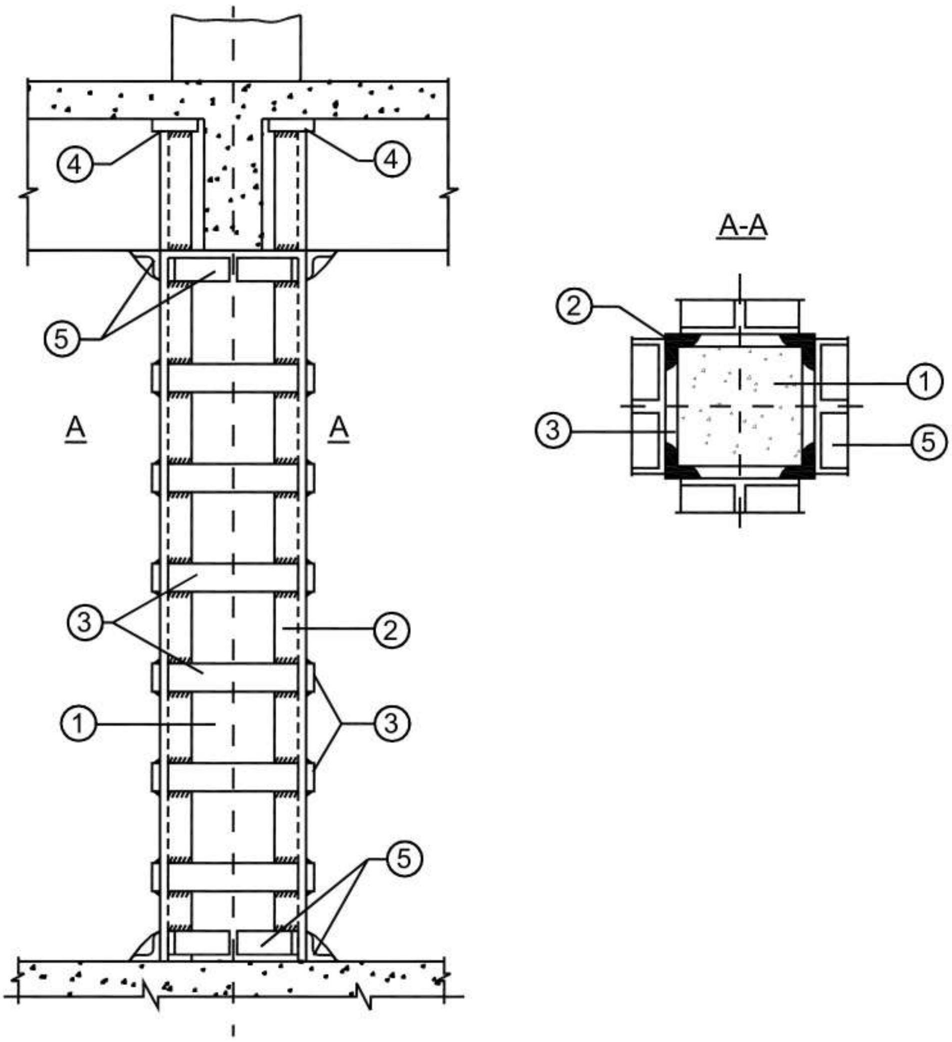


1 - existing column; 2 - jacket; 3 - added reinforcement;
4 - hoop; 5 - drilled holes

Jacketing of column in case of flat slab or relatively narrow beams[26]

4.24 What are the constructional details in steel or strap jacketing of columns?

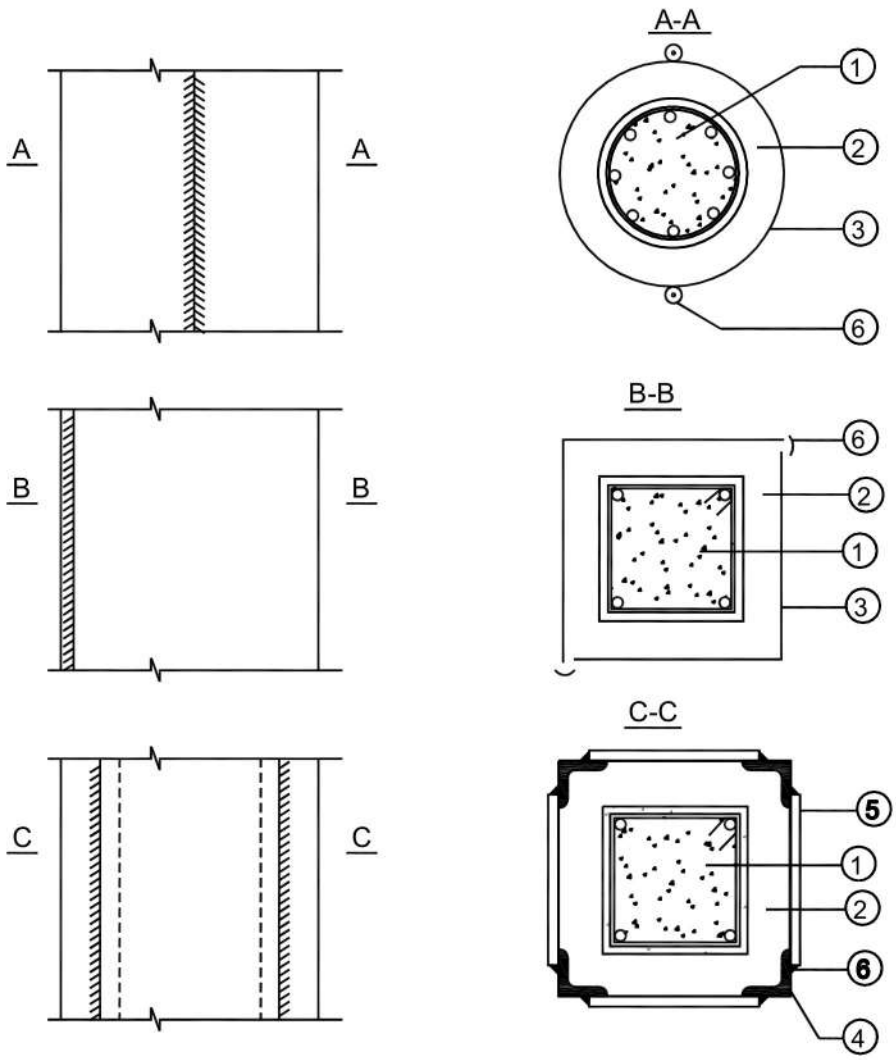
The steel jacketing involves the encasement of the column with thin steel plates placed at a small distance from the column surface, ensuring gap filled with non-shrink grout. An alternative to a complete jacket is the steel cage or steel angles, placed at the corners of the existing cross-section. Space between the steel cage and the existing concrete are usually filled with non-shrink grout. When corrosion or fire protection is required, a grout concrete or shotcrete cover may be provided. The steel jacket is terminated 1.5″ from the top of the footing to avoid possible bearing of the steel jacket against the footing.



1 - existing column; 2 - steel angle profile; 3 - steel plate;
4 - supporting plate; 5 - angle profile

Steel profile jacketing for shear only[26]

The different options for steel jacketing are as follows;



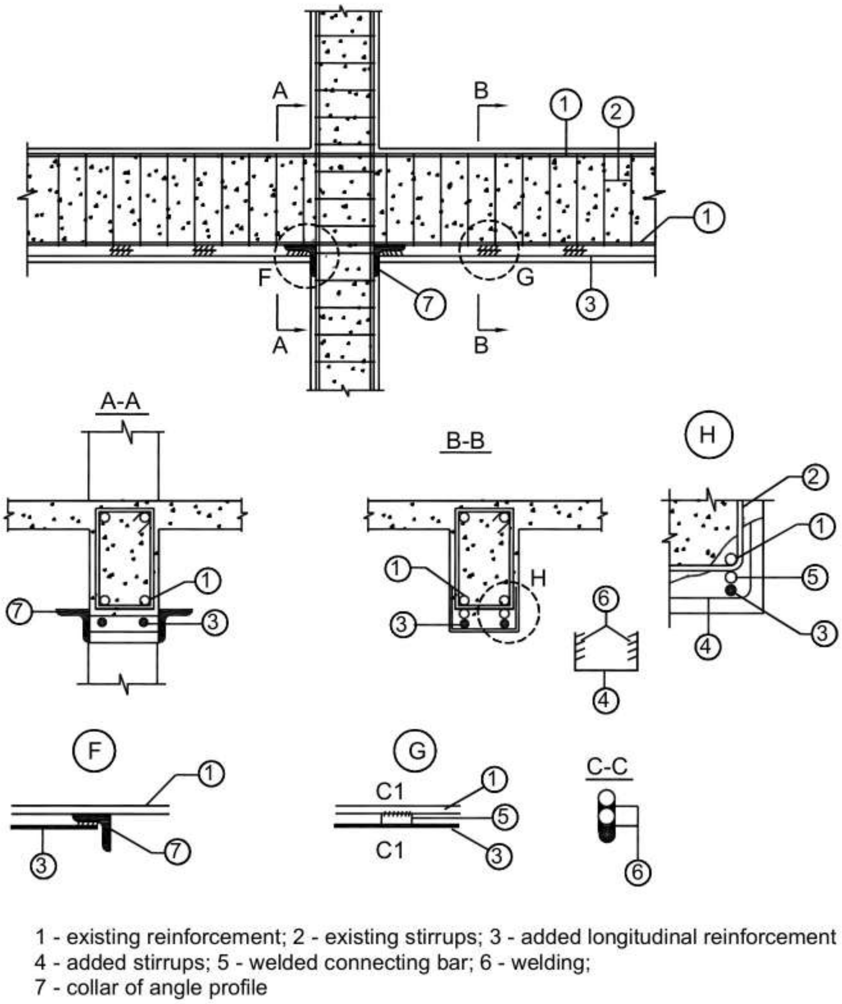
1 - existing column; 2 - new concrete or grout; 3 - steel incasement;
4 - steel angle profiles; 5 - steel plate; 6 - welding

Steel casements retrofit for shear only [26]

4.25 What are the possible ways to carry out beam jacketing?

Jacketing of beams is recommended because it gives continuity to the columns and increases the strength and stiffness of the structure. In the retrofitted structure, flexural resistance must be carefully computed to avoid strong beam-weak column system since there is a strong possibility of change of mode of failure and redistribution of forces. Beam jacketing may be carried out by one-sided jackets or 3 and 4-sided jackets. At several occasions, the slab has been perforated. The beam should be jacketed through its whole length. The reinforcement has also been added to increase beam flexural capacity moderately.

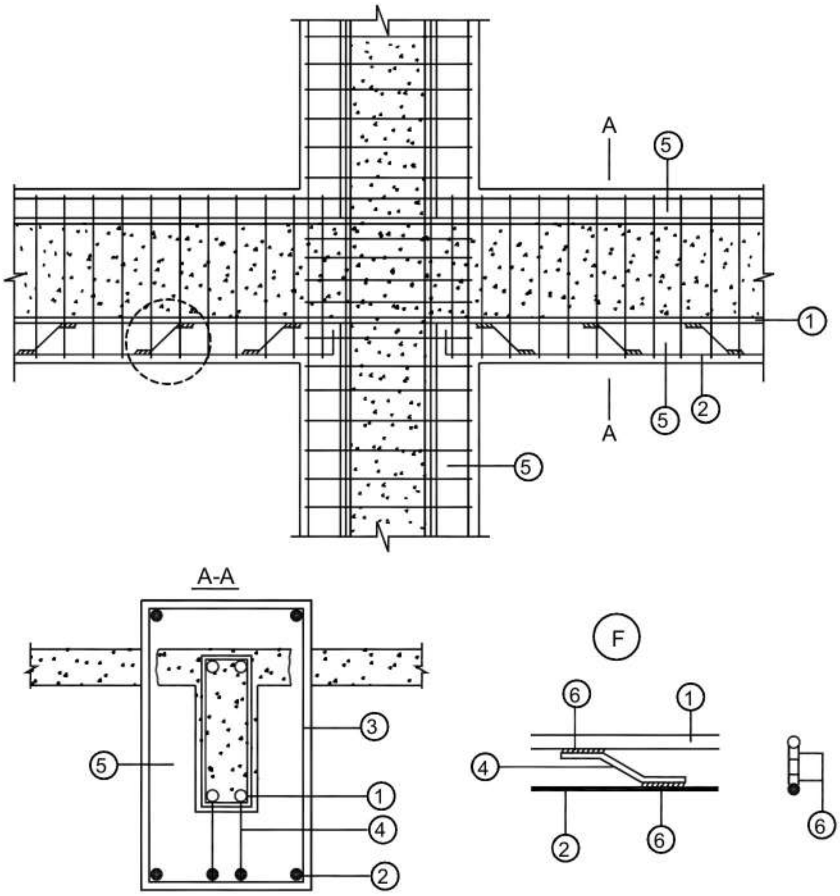
Steel plate adhesion is also used to retrofit beams. It is advisable to use several thin layers instead of one thick plate, to minimize interfacial shear stresses. The execution of the bonding work helps to achieve a composite action between the adherents. Prevention of premature de-bonding or peeling of externally bonded plates is a most critical aspect of designs.



One sided jacketing or adding strength only to beam soffit[26]

4.26 How to carry out four sided RC jacketing of beam?

Four sided jacketing of beam is useful to increase its flexural and shear strength of existing beam. The details for the jacketing are as follows;

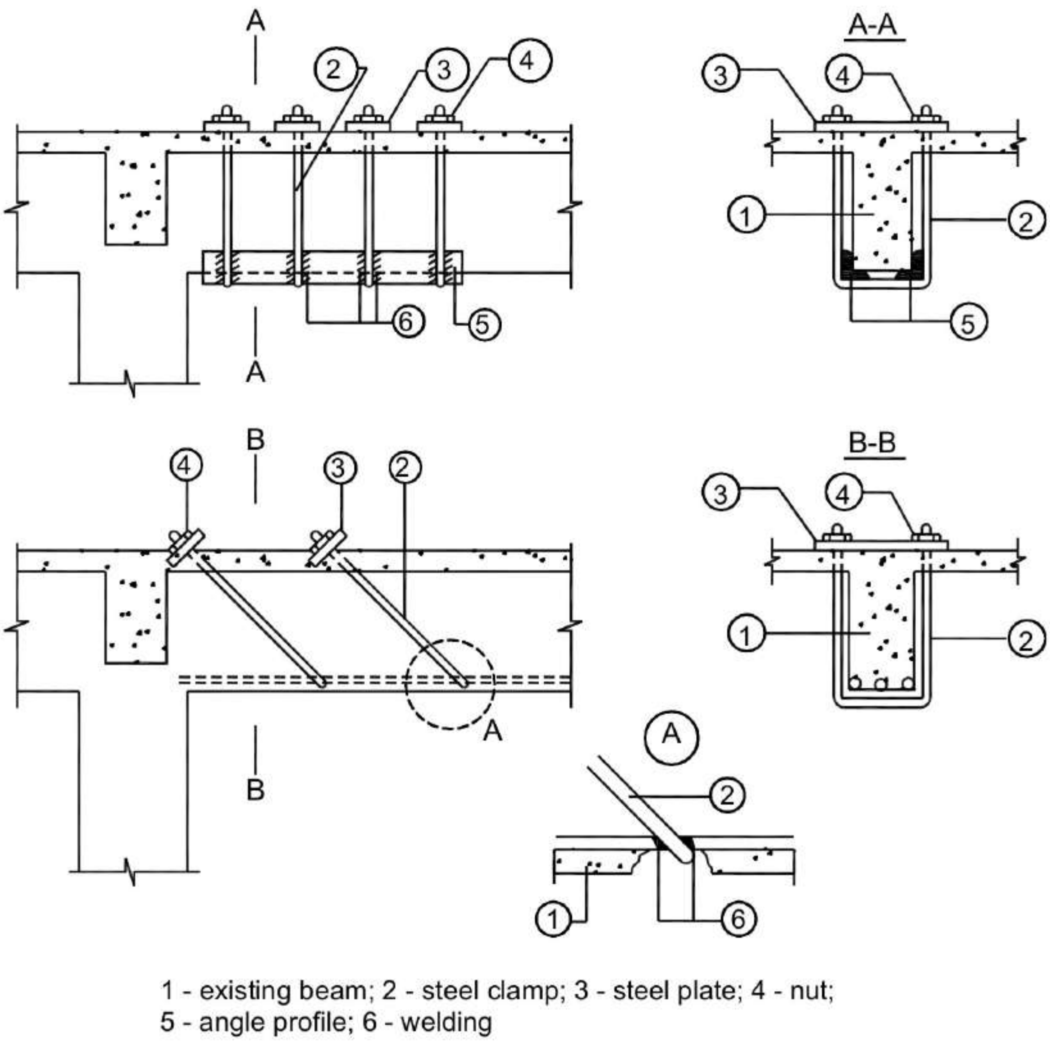


1 - existing reinforcement; 2 - added longitudinal reinforcement; 3 - added stirrups;
4 - welded connecting bar; 5 - comcrrete jacket; 6 - welding

***Four sided jacketing or adding strength to beam in flexure and shear*[26]**

4.27 How to increase the gravity load capacity of existing beam?

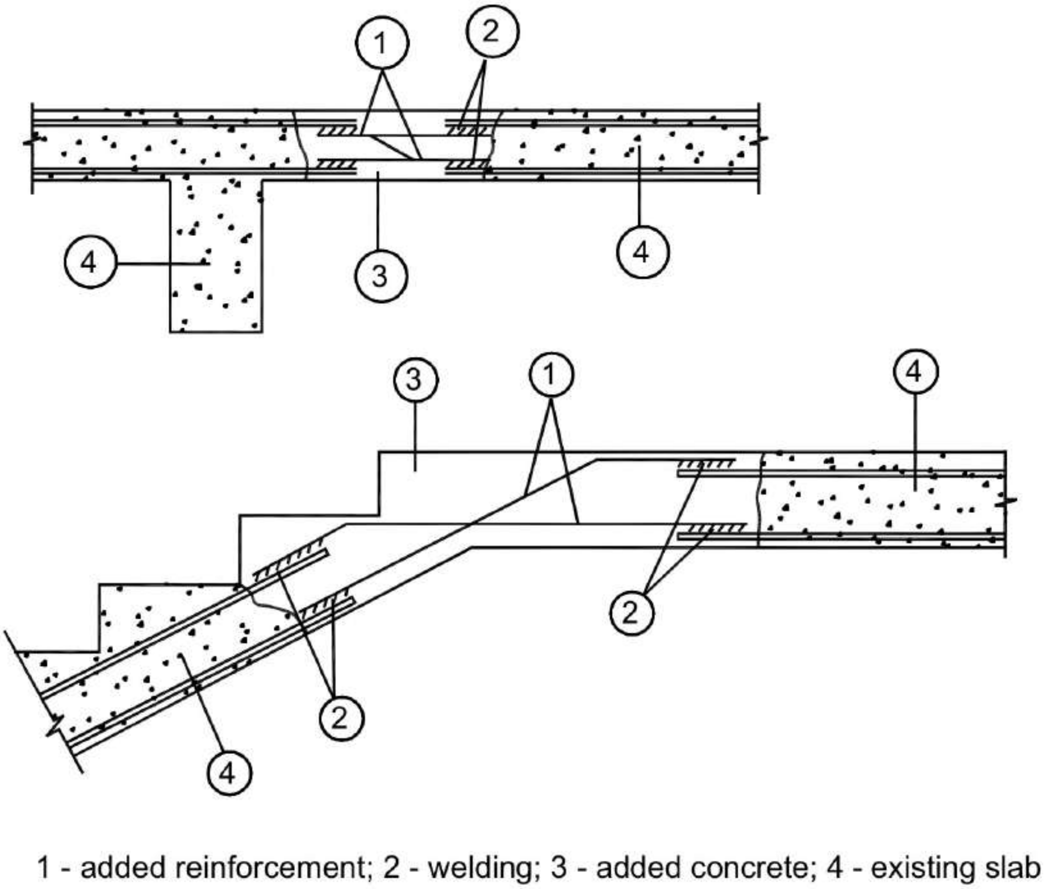
Steel rods can be used to improving the shear resistance of damaged or undamaged beams. It can be performed by vertical external clamps or by diagonal ones as shown below;



***Four sided jacketing or adding strength to beam in flexure and shear*[26]**

4.28 What are the possible ways to carry out jacketing of staircase slab?

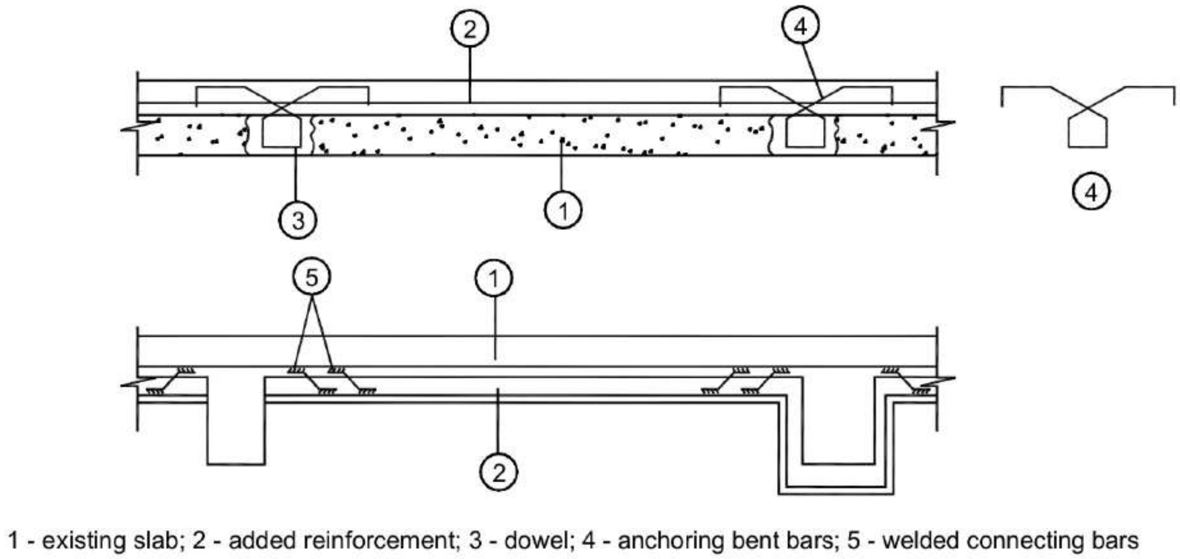
The possible ways to repair the floor slab/staircase slab are as shown below;



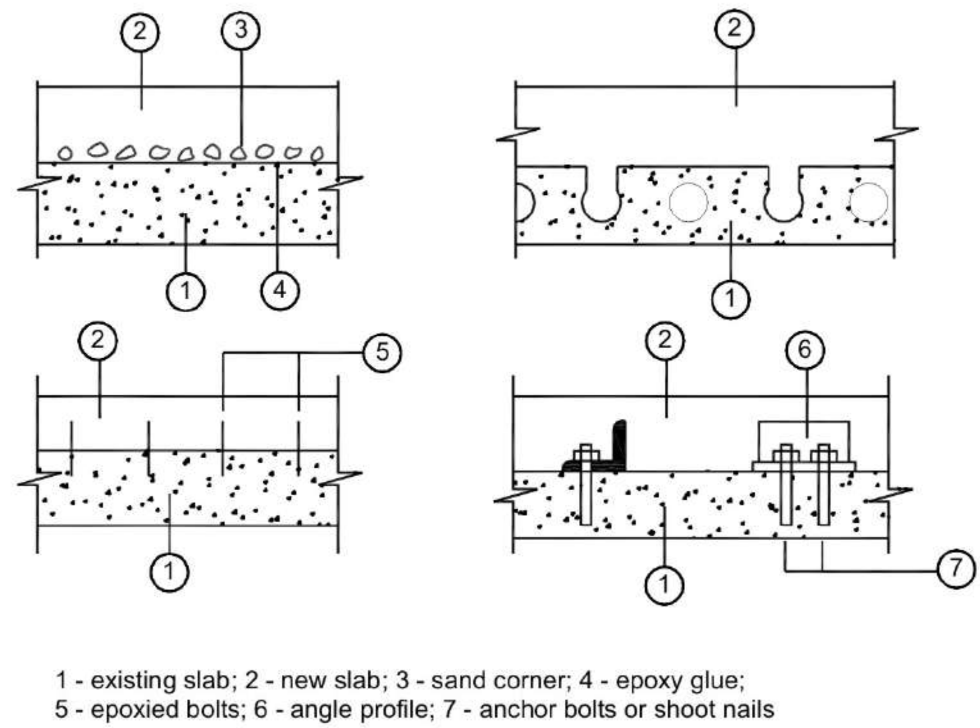
Possible ways to repair the floor slab/staircase slab[26]

4.29 What are the possible ways to increase the thickness of an existing slab?

Possible ways to increase the flexural strength of existing slab by increasing its thickness. Two details may be possible (i) adding of reinforcement from top (ii) adding of reinforcement from bottom. First option is more preferred.



Details for adding of reinforcement in case of existing slab[26]

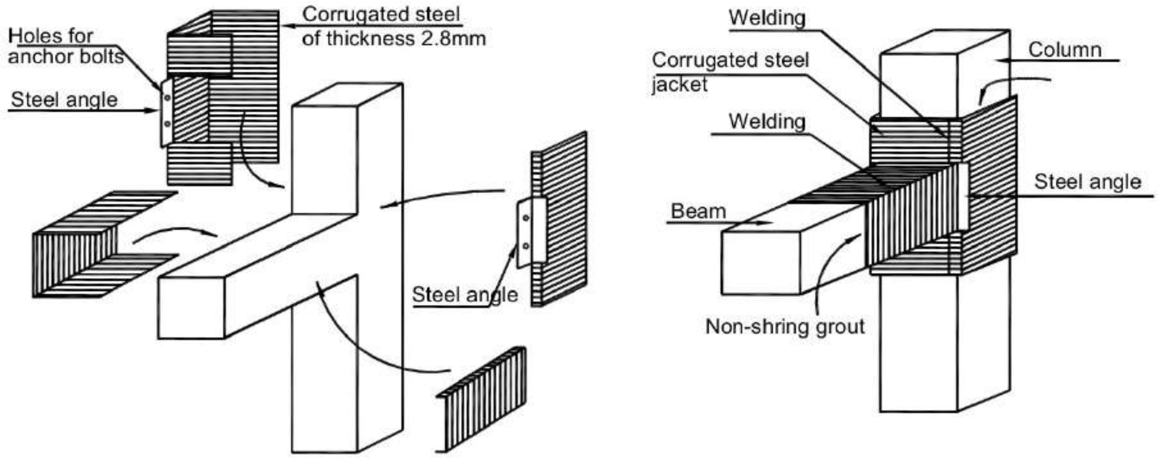


Option for the compatibility of the existing slab and the newly added reinforced concrete[26]

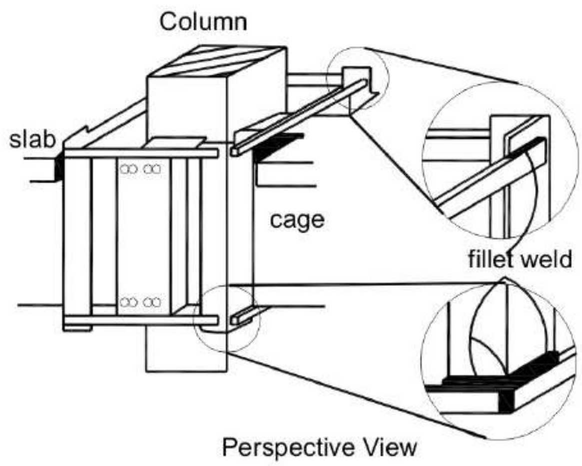
4.30 Is beam – column joint jacketing also possible?

Various configurations of steel jackets, plates, or shapes have been used to increase the strength and ductility of deficient beam-column joints. Steel jackets consist of flat or corrugated steel plates, or rectangular or circular steel tubes prefabricated in parts and welded in place. The jacket is expected to provide lateral confinement and shear resistance to the joint area, thereby adding strength and ductility to the joint. The corrugated steel jackets are constructed in two halves for easy installation. The vertical seams are welded in situ. The gap between the concrete and the steel jacket is then filled with grout to provide continuity between the jacket and the concrete. The corrugated shape is needed to provide confining pressure by passive restraint in the joint region. However, due to lack of space in the joint region it is difficult enough to provide an adequate confinement

Another alternative to strengthen the beam-column joint with a steel cage is welded around the joint after casting the column jacket as shown. This type of jacketing is effective in rehabilitating the joint, with improving the strength, stiffness and energy dissipation characteristics of the existing joint.



Jacketing with corrugated steel plates[34]

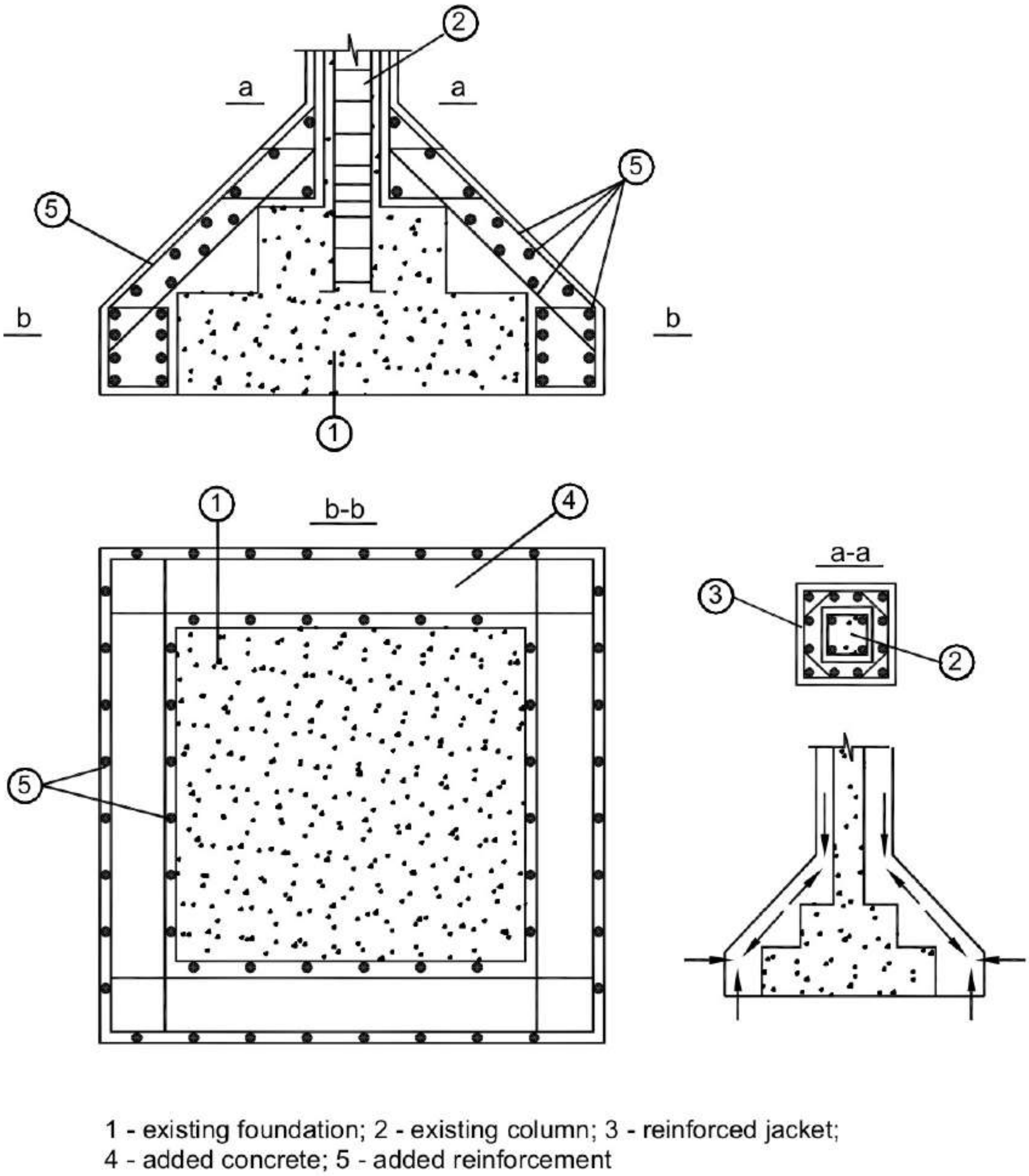


Jacketing with steel cage

Alternative methods for jacketing of beam-column joints[35]

4.31 How to retrofit the existing footing of a retrofitted column?

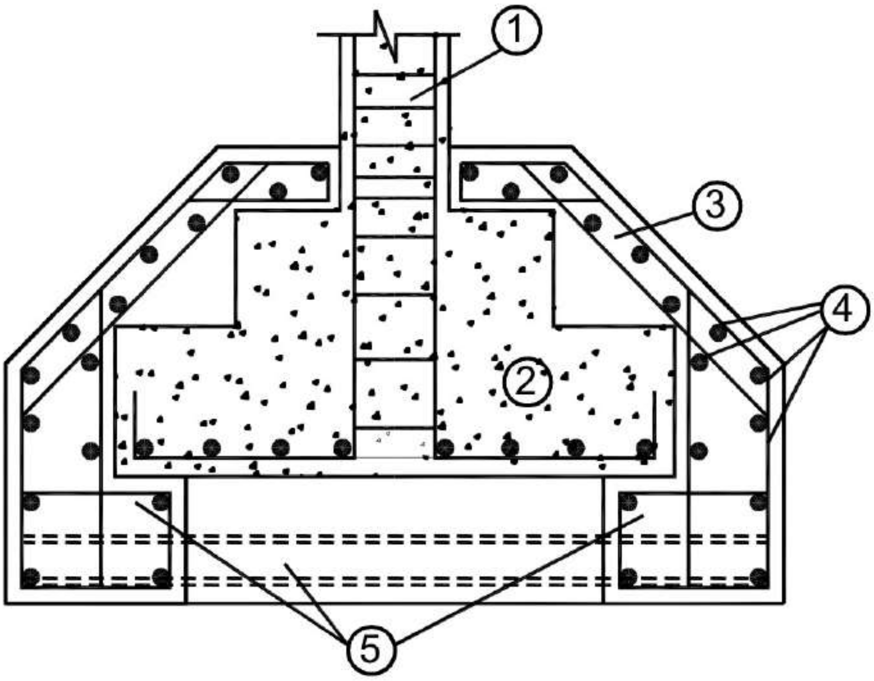
Retrofitting of existing footing depends upon the objective of retrofitting of columns. In first case, when the existing column is retrofitted with the objective to increase the axial, flexural and shear strength, the capacity of the existing footing is also increased so that the plastics hinges may form in retrofitted column. In second case, the existing column is retrofitted with the objective to increase only shear and axial capacity without increasing flexural strength; footing is retrofitted only for the additional load imposed on it. Retrofitting of footing under both the cases is shown below.



Strengthening of footing of a retrofitted column to increase its (a) flexural, shear and axial strength (b) only shear and axial strength[26]

4.32 How to retrofit the existing footing of an un-strengthened column?

In the case of an un-strengthened column, the details of reinforcement are as follows;



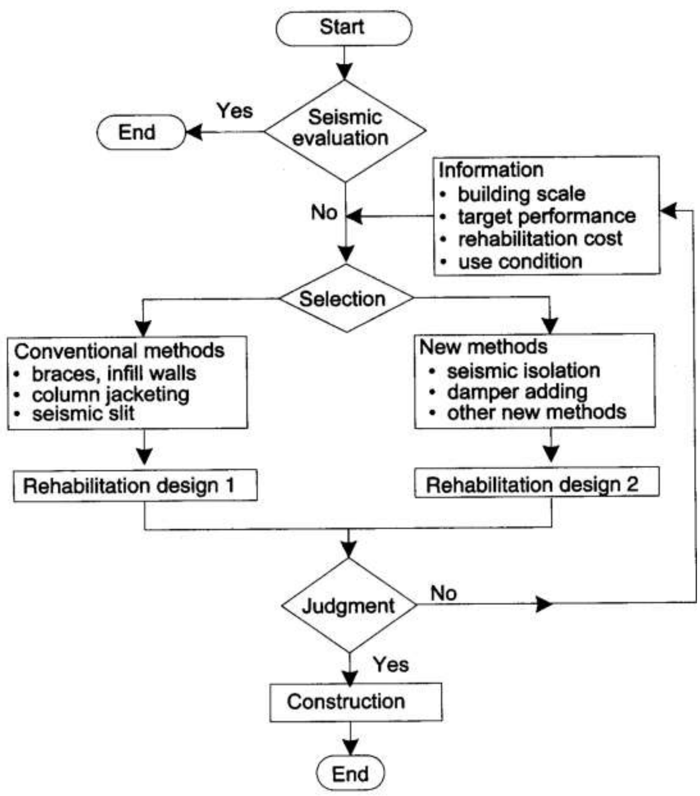
1 - existing column; 2 - existing foundation; 3 - added concrete;
4 - added reinforcement; 5 - steel profile

Strengthening or jacketing of foundation in case of un-strengthened column[26]

4.33 What is the procedure for retrofitting of a reinforced concrete building?

The complete procedure for retrofitting of an RC building illustrated in the flow diagram is shown below. The collection of information for the as-built structure is the first step of retrofitting. The configuration of the structural system, reinforcement detailing, material strengths, information relevant to the non-structural elements (e.g. infill walls), foundation system and the level of damage are recorded. Sources for the above information can be obtained from the site visits, construction drawings, engineering analyses and interviews with the original contractor. The retrofitting objective is selected from various pairs of performance targets and earthquake hazard levels. The performance target depends on acceptable damage level in a predefined seismic event on the basis of safety of occupants during and after the seismic event, the cost and feasibility of restoring the building to pre-earthquake condition, the length of time the building is removed from service to effect repairs, and the economic, architectural or historic impacts on the larger community.

In the next phase, the retrofitting method is selected starting with the selection of an analysis procedure. The development of a preliminary retrofitting scheme follows (using one or more retrofitting strategies) the analysis of the building (including retrofitting measures), and the evaluation of the analysis results. Further, the performance and verification of the retrofitting design are conducted. The retrofitting design is verified to meet the requirements through an analysis of the building, including retrofitting measures. A separate analytical evaluation is performed for each combination of building performance and seismic hazard specified in the selected retrofitting objective. If the retrofitting design fails to comply with the acceptance criteria for the selected objective, the interventions must be redesigned or an alternative strategy be considered.



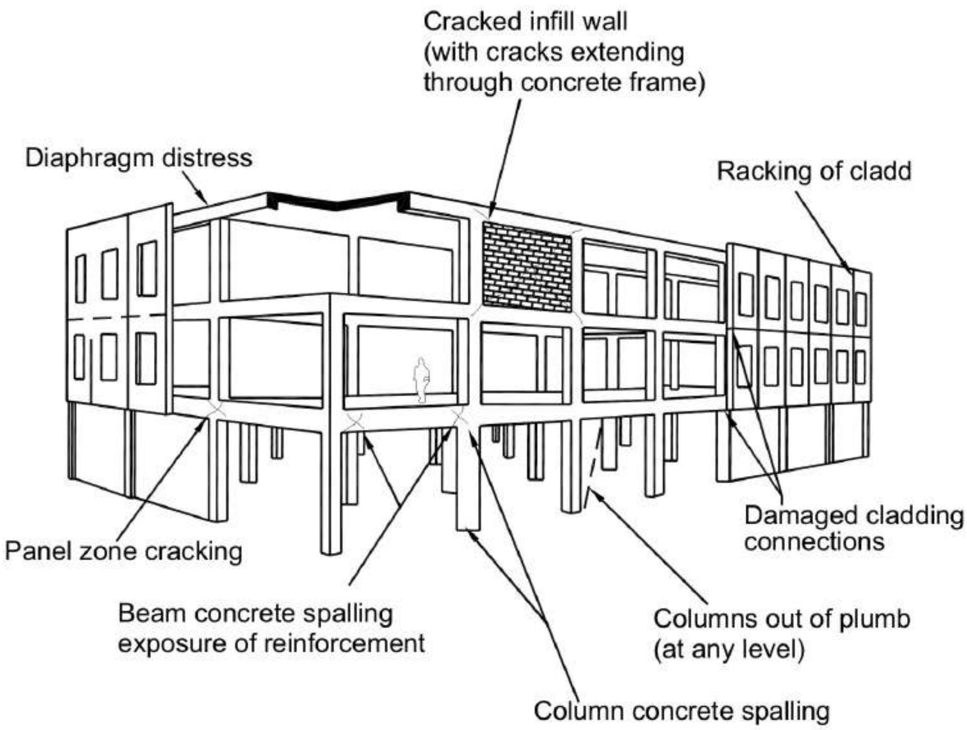
Seismic evaluation and retrofitting procedure for a RC building

Chapter 5

Post Earthquake Damage Evaluation and Retrofitting of Reinforced Concrete Buildings

5.1 Why do the repair and retrofitting of earthquake damaged buildings essential?

The occurrence of earthquake leaves behind innumerable damaged structures with varying intensities ranging from minor cracking to total collapse. At such a moment replacement is neither feasible nor practical to meet out shelter problems. Therefore, repair and retrofitting is the only solution which may convert such buildings to seismically safe structures for future earthquakes. This section of the manual intends to provide guidance for repair and retrofitting of earthquake damaged structures.



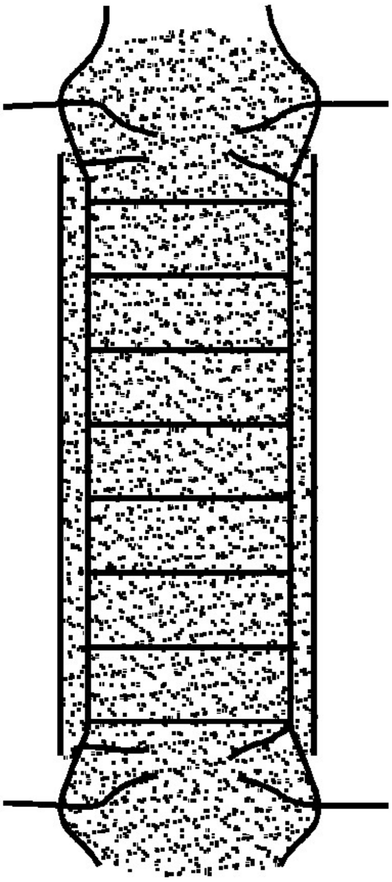
Typical damages in a RC buildings[11]

5.2 What are the causes of column failure in an RC building and in what manner?

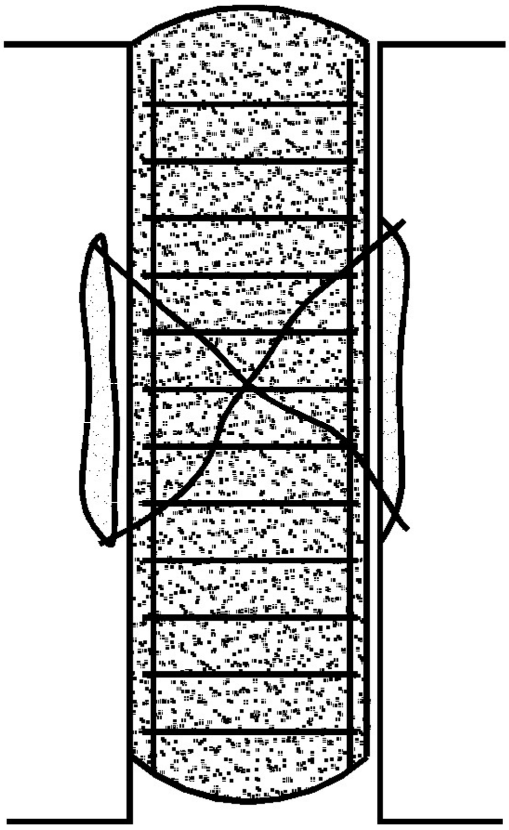
Columns are damaged mainly due to lack of confinement, large tie spacing, insufficient splices length, inadequate splicing at the same section, hook configurations, poor concrete quality, less than full height masonry infill partitions, and combinations of many of the above compounded with vertical and geometrical irregularities. Failure of columns has catastrophic consequences for a structure. Two types of failure in columns are generally observed as shown below and their consequences are also listed.

Failure sketch[28]

Type 1: Damage at the top and bottom section of column (often occurs in long columns)



Type 2: Damage in weakest part of column in the form of X-shaped cracks (often occurs in short columns)



Consequences of damage

Type 1:

- Loses its ability to carry vertical load

Type 2:

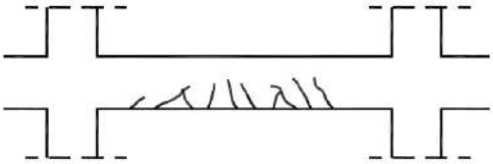
- Spectacular collapse of the building
- Generally occurs in columns of ground floor
- Loss of equilibrium

5.3 What is the cause of beam failure in a building and in what manner?

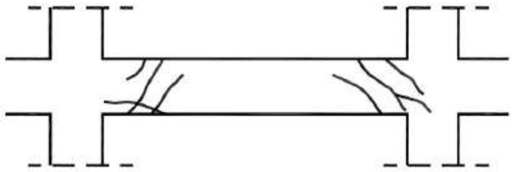
Only a few examples exist in which buildings have exhibited plastic hinging in the beam. The probable regions of hinging are at and near their intersections with supporting columns[28]. The causes of hinging are lack of confinement of concrete core and support for the longitudinal compressive reinforcement against inelastic buckling.

Failure sketch[28]

Type 1: Orthogonal to beam axis along the tension zone of the span



Type 2: Shear failure near the supports of beam



Consequences of damage

Type 1:

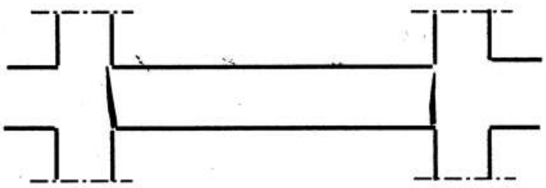
- Most common type of damage
- Existing micro cracks, due to bending of the tension zone, widened due to vertical component of earthquake.
- Does not jeopardize the safety of structure

Type 2:

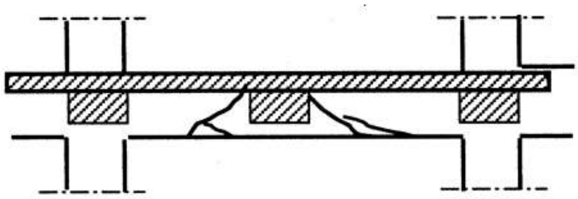
- Second most frequent type of damage
- More serious than the previous one
- Brittle in character
- Sometimes jeopardize the overall stability of the structure

Failure sketch[28]

Type 3: Flexural cracks on the upper and lower face of beam at the supports



Type 4: Shear failure at the location of indirect support/secondary beams



Consequences of damage

Type 3:

- Flexural cracks on the upper and lower face of the beam
- Cracking at lower face due to bad anchorage of the bottom reinforcement in to the support, in that case one or two cracks from close to support

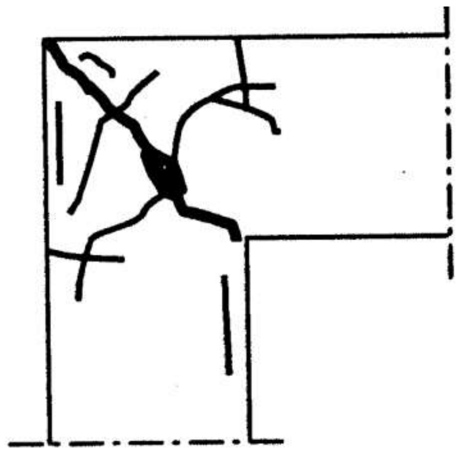
Type 4:

- Due to the vertical components of earthquake which amplified the concentrated load

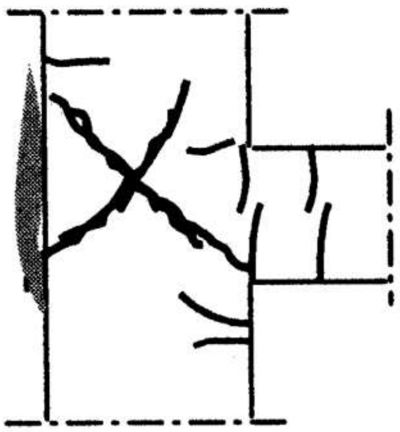
5.4 What is the cause of beam -column joint failure in a building and in what manner?

Beam-column joints are critical elements in frame structures, are subjected to high shear and bond-slip deformations under earthquake loading. Account for cross-sectional properties of the joint region, amount and distribution of column vertical steel, inadequate or absence of reinforcement in beam-column joints, absence of confinement of hoop reinforcement, inappropriate location of bar splices in columns are the common causes of failure of beam-column joints.

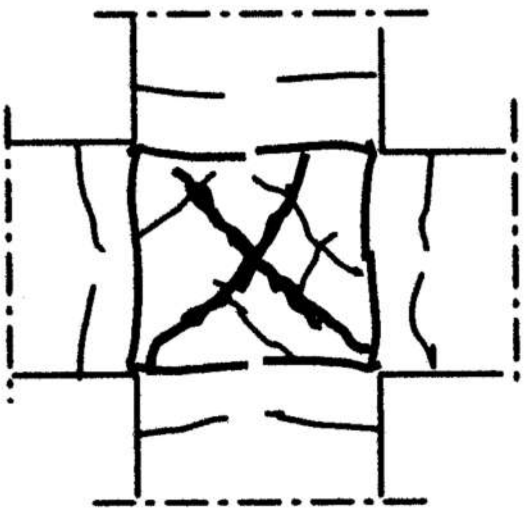
Failure sketch[28]



Type 1: Corner Joint



Type 2: Exterior Joint



Type 3: Cross shaped Joint

Consequences of damage

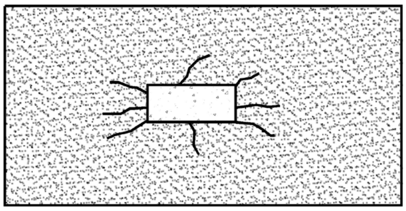
Reduce the overall stiffness of the building

5.5 What is the cause of slab failure in a building and in what manner?

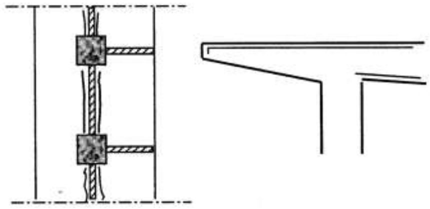
Generally slab on beams perform well during earthquakes and are not dangerous but cracks in slab create serious aesthetic and functional problems. It reduces the available strength, stiffness and energy dissipation capacity of building. In flat slab construction, punching shear is the primary cause of failure.

Failure Sketch[28]

Type 1: Cracks parallel or transverse to the reinforcement at random locations



Type 2: Cracks at critical sections of large spans or large cantilevers, transverse to the main reinforcement



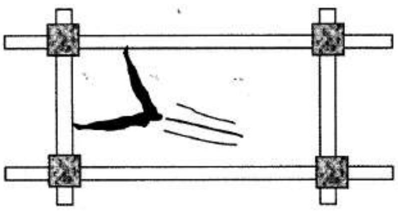
Consequences of damage

- Most of time it is due to the widening of already existing micro-cracks due to bending action/temperature changes/ shrinkage. It became visible after dynamic excitation
- Sometimes due to differential settlement of columns

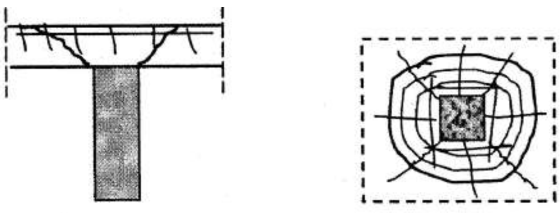
- Mostly due to the vertical component of the earthquake action

Failure Sketch[28]

Type 3: Crack at locations of floor discontinuities, such as the corner of large openings accommodating interval stairways, light shaft and so on



Type 4: Cracks in areas of concentration of large seismic load effects, particularly in the connection zones of slab to shear walls or to column in flat plate system



Consequences of damage

- Mostly due to the vertical component of the earthquake action

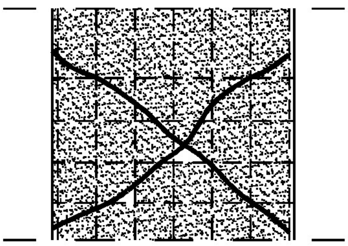
- Punching shear failure
- Aggregated by the cyclic bending moment caused by an earthquake
- Slab directly on column seismically vulnerable structures. It should be avoided

5.6 What is the cause of shear wall failure in a building and in what manner?

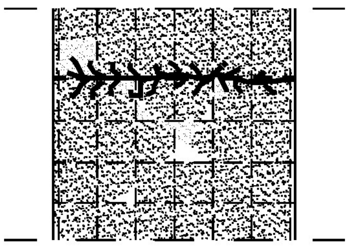
Shear wall generally performs well, but sometimes shows diagonal flexural-shear cracking causing significant damage to coupling beams and short piers between openings

Failure Sketch[28]

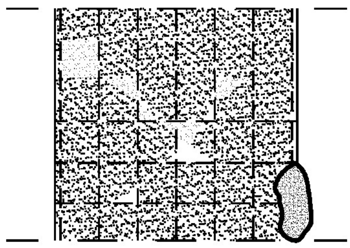
Type 1: X-shaped shear cracks - limited spelling



Type 2: Sliding at the construction joint



Type 3: Damage due to flexure and compression



Consequences of damage

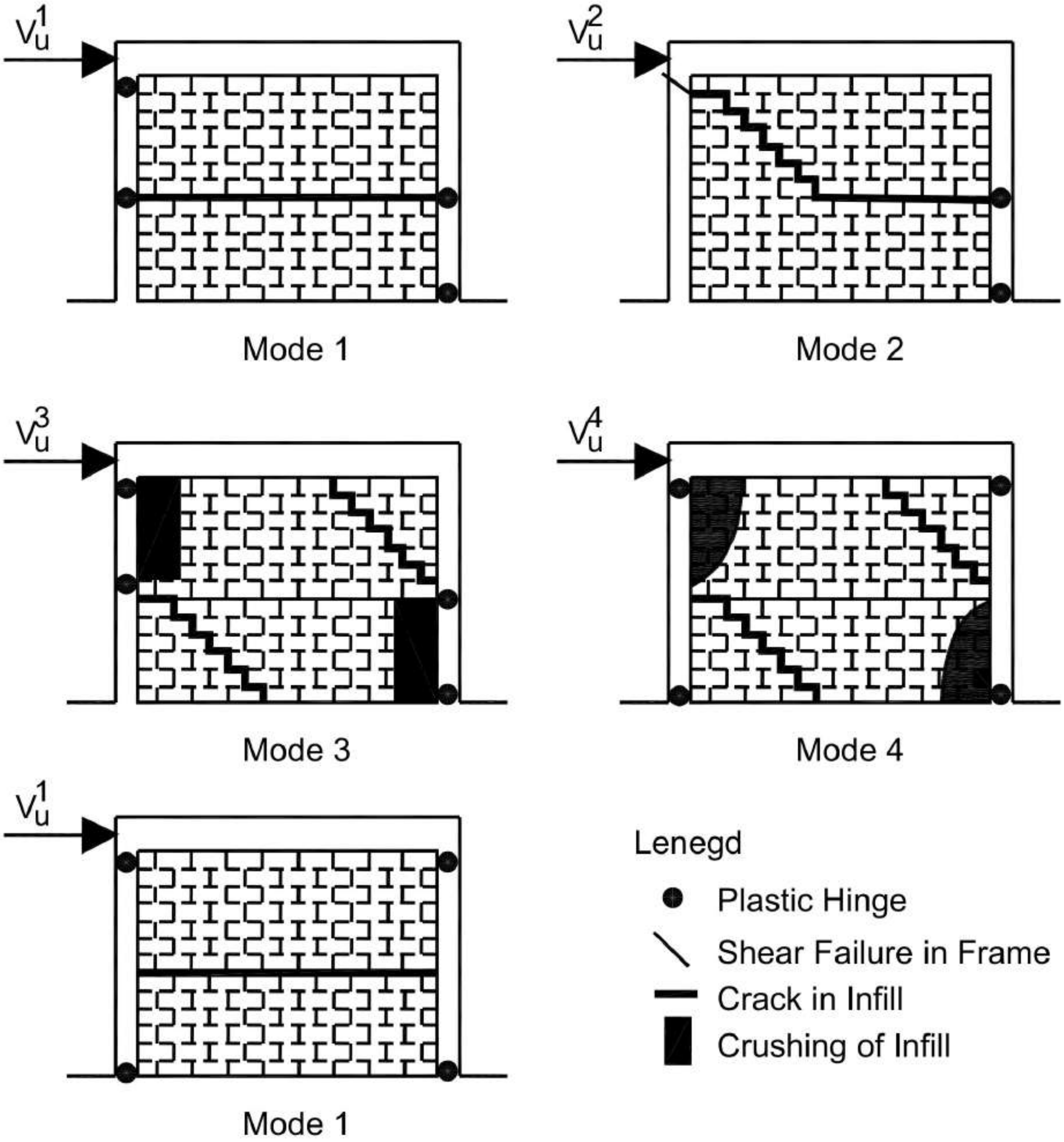
- Under the action of vertical loads, the isosceles triangles formed on the two sides tend to separate from the structure and therefore cause of collapse

- Most frequent type of failure
- Old concrete is not properly bounded with fresh concrete
- Not too serious because the structure still carries vertical load

- Very rarely occurs because of bending moment developed at the base of the wall much smaller when those calculated for the design.

5.7 What is the cause of infill failure in a building and in what manner?

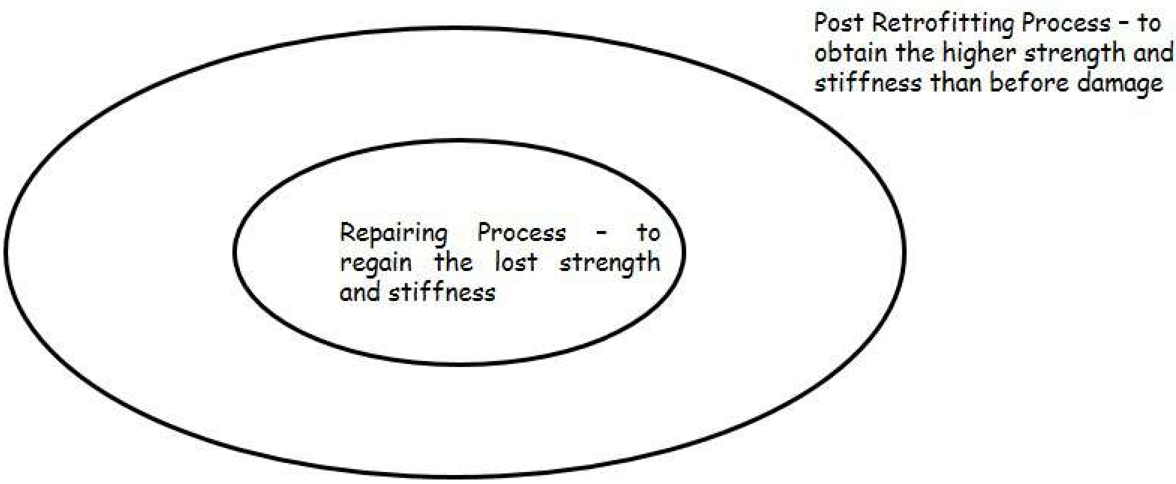
Infill wall failure occurs due to stiffening effect of infill panels because of i) unequal distribution of lateral forces in the different frames of a building - overstressing of some frames ii) vertical irregularities in strength and stiffness - soft storey iii) horizontal irregularities - torsion and iv) failure of infill itself.



Mode of failure of in-filled masonry in RC moment resisting building frame[29]

5.8 Why is repair necessary before retrofitting of earthquake damaged structure?

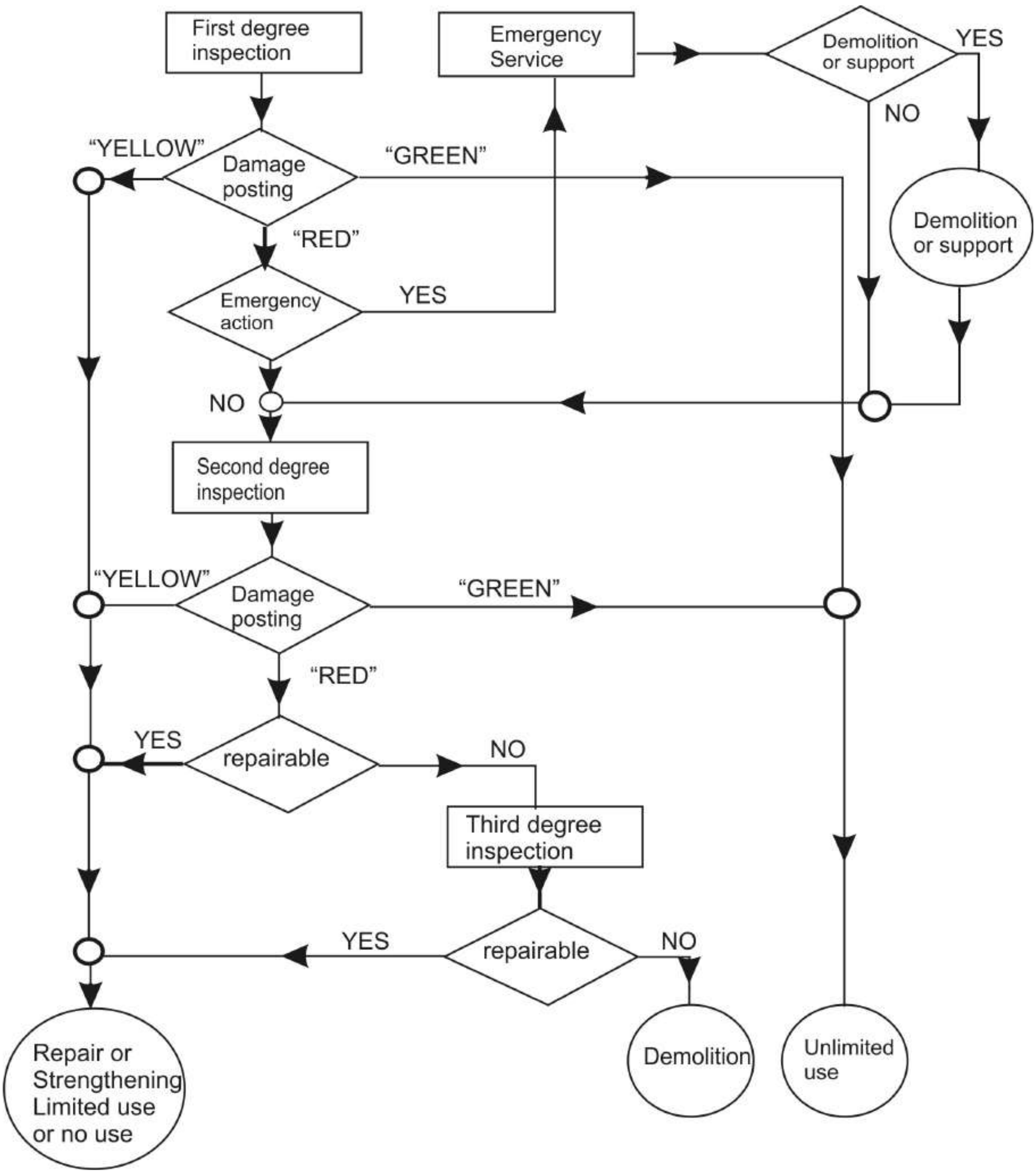
The aim of repair is to reestablish the initial strength of damaged structural members. Properly repaired structural members may posses the same strength but will have a somewhat reduced stiffness due to very fine cracks. Retrofitting is the judicious modification of the strength and/or stiffness of structural members and improves structural performance. Retrofitting will be effective when the damaged structures have sufficient strength and stiffness achieved by the repairing process. The choice of repair depends upon post earthquake evaluation besides other minor factors such as local site conditions, type and age of the structure, type and degree of damage, available time, equipment and staff for specific architectural requirements, cost, and the required level of seismic safety.



5.9 What should be the aim of post earthquake damage evaluations?

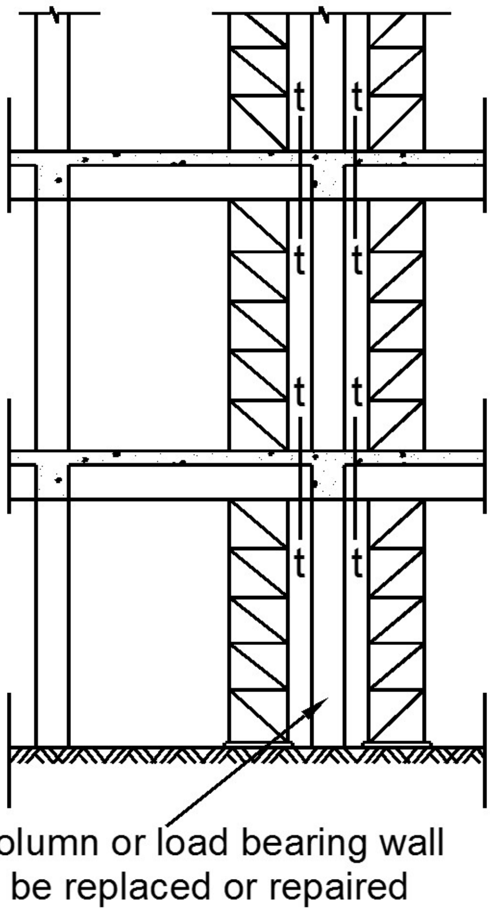
Post earthquake investigation is the statement of the structure pertaining to its nature and degree of damage, design and emergency measure for temporary support to minimize the possible material loss in case of increased damage to structure during aftershocks. The post damage evaluation is also utilized in determining repair and/or retrofitting measures.

It is an evaluation of the structure which may require removal of some non-structural components, coatings, concrete covers, details of cracks, yielded reinforcement, excessive deformations, connection failures, etc. It is also an assessment of the structural members to resist the seismic forces and the dead and live loads[26].



5.10 What are the considerations for emergency measures in temporary protection?

Temporary protection provides temporary strength or support for damaged elements and connections to achieve safety of the whole structural system. Temporary support is recommended for severely damaged buildings which exhibit in the form of a rupture of a column, serious cracking of load bearing walls, etc. Shoring of the damaged elements relieving danger of collapse during aftershocks is diminished. Support is needed at the floor of the damaged vertical element which may be extended to other floors as well. The purpose is to provide safety to the people in adjacent streets, sidewalks and yards; to workmen making repairs and retrofitting provisions. The distance between the supports and the damaged member must be minimum and providing enough unobstructed space for the eventual repair work or replacement of damaged elements. Against unstable horizontal forces, lateral counter forts or wall braces should be provided for walls which may fall laterally. Diagonal braces of structural frames can be installed. The whole procedure should be properly organized, in order to minimize the working time of people in and under the structure.



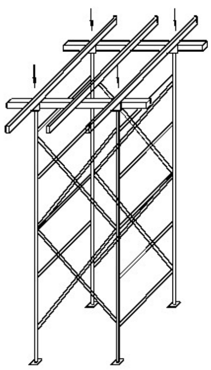
***Temporary Protection of a damaged column*[26]**

5.11 What are the methods for supporting vertical loads?

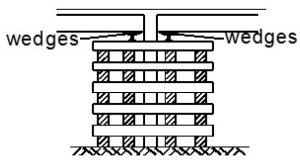
A suitable method of supporting vertical load can be support of damaged or failed members with techniques such as industrial scaffolding; tree logs, steel profiles, or grillage logging[26]. This depends on means, seriousness of damage and size of structure.

Industrial-Type Shoring and Scaffolding	For small loads, independent industrial-type metal tube shores For shoring of beams or slabs - dismountable type metal towers, wedged to the surface with the aid of special screw type bolts.
Timbers, Tree Logs and Telephone Poles	For one damaged column – Minimum one 250 mm diameter log on each side For two or more supporting elements - X-shaped braces may be preferred
Built-up steel members or Steel Profiles	In the same manner as in case of tree logs. Steel sections require bearing plates of both sections top and bottom and must be properly wedged. Steel profiles assembled around the perimeter of a damaged column similar to permanent jacketing can also be utilized for temporary support
Timber Grillages	If wooden rail sleepers or other similar timber is available, vertical support can be erected by forming a grillage. The sleepers are placed in alternating layers to the required height. On top of the grillage wide flanges steel I-beams or suitable timbers are placed

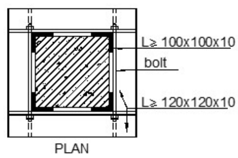
Industrial shoring[26]

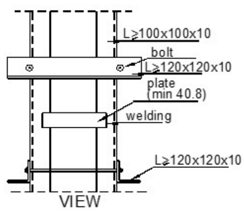


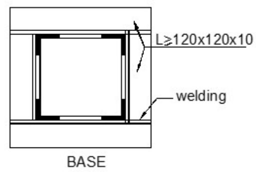
Industrial type scaffolding[26]



Shoring with timber grillage





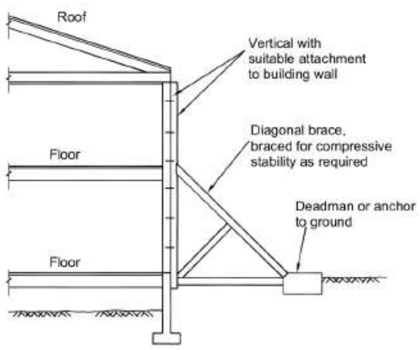


Built-up steel members or steel profiles[26]

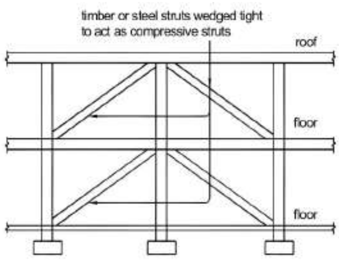
5.12 What are the methods for providing lateral support?

A suitable method of supporting lateral support of damaged walls or failed members are such as lateral wall bracing, frame bracing, wedging techniques etc[26].

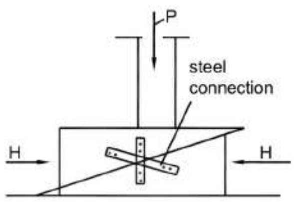
Lateral Wall Bracing	Lateral wall bracing like timbers, logs or steel profiles may be used to support the hazardous/damaged/instable exterior bearing walls of masonry, stone or concrete block construction since the walls may fall outward due to the loss of vertical support
Frame Bracing	For concrete framed structures, without in-filled walls in the lower story or with heavily damaged in-filled walls in a particular story; internal diagonal frame bracing can be used. Frame bracing consists of timbers, tree or steel profiles of sufficient strength for buckling. The bracing members are installed diagonally between columns on a frame line, consisting of several more slender elements laced together to form a built-up compression member. The location of the braces should be provided in a balanced system
Wedging Techniques	Wedging is recommended for all temporary supports to transfer the loads from the damaged member to the new support system for which several methods may be used, such as ordinary wooden wedges with suitable securing devices, mechanical jacks, hydraulic jacks or hydraulic flat jacks to ensure uniform loading and unloading



Lateral Wall Bracing[26]



Frame Bracing[26]



Wedging Techniques[26]

5.13 How to carry out retrofitting process for a damage building?

Following steps are generally followed while carrying out retrofitting of a damaged building during an earthquake;

Step	Action required
1	Fix emergency measures for temporary protection
2	Detailed documentation such as design calculations, drawings, specifications, construction details, original construction data, material strengths, foundation and soil condition data, previous repairs or alterations, codes followed etc.
3	Visually inspect each and every structural member such as, beam, column, beam-column joints, staircase, floor slabs and the connections between floors and walls and foundations and note the location and amount of damage. The reason of failure should be highlighted like shear, compression, tension, flexure, bar anchorage, etc.
4	Prepare the plan or alternative schemes to repair and /or retrofitting/ strengthening the structure with cause of damage underlined such as discontinuities in strength or stiffness, torsion, hammering with adjacent structures, improper connections or details, effects of non-structural elements.
5	Try to estimate the existing strength and stiffness of the damaged structure and with the repair and /or strengthening schemes. Precaution must be taken that the strengthening/retrofitting elements should not cause increased damage in a future earthquake. For example, If shear walls are added, new foundations will be required not only to support the weight of the wall but particularly the overturning forces otherwise it is fatal in itself
6	Finalize the schemes as per feasibility, imagination and ingenuity with professional experience best with economy
7	Finally design procedures include a completion of the detailed calculations of the strengthening/retrofitting solution and the preparation of drawings, specifications and instructions of the work

5.14 What type of materials can be used in Repair and Retrofitting Project?

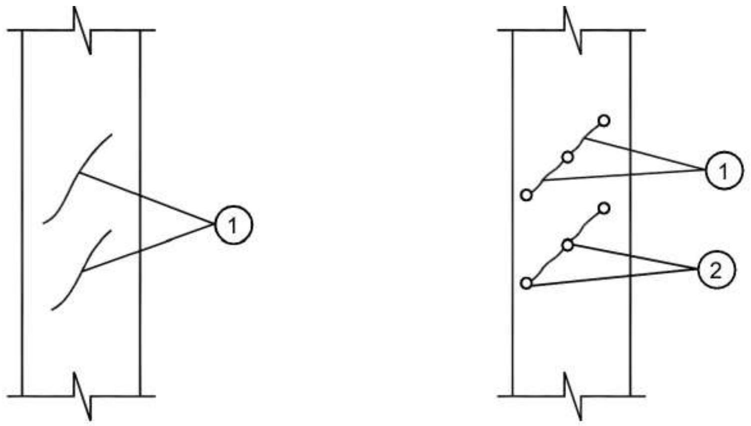
Different types of materials may be used in repairing and retrofitting structures. Before utilizing any of these materials or techniques, the designer should study technical literatures, obtain advice, and be thoroughly familiar with the process. The most common type of materials used in repair and retrofittings are;

Conventional cast-in-place concrete	Low shrink concrete with higher strength than existing is recommended ($f_{c\ rep} \geq f_{c\ exist} + 5$ MPa).
High strength concrete	Conventional concrete with super plasticizers and expansive admixtures in the appropriate proportions
Shotcrete (Gunite)	Dry mix concrete of higher strength than existing is recommended ($f_{c\ rep} \geq f_{c\ exist} + 5$ MPa).
Polymer concrete	Polymer-modified concrete has mainly two advantages (i) water-reducing plasticizers, (ii) improving the bond between old and new elements but several disadvantages like vulnerability to fire and lower alkalinity present inferiors resistance against carbonation compared to conventional concrete
Resins	Resins are used for grouting injections into cracks in order to glue together the cracked concrete or thin metal sheets on concrete surfaces. Its modulus of elasticity must be compatible to the concrete to be glued and viscosity appropriate for the crack width. Resins lose their strength in temperatures higher than 100°C and therefore such repairs are not fireproof. **Epoxy resins** are the most common type of materials in use today
Grouts	Grouts consist of cement, water, sand, plasticizers and expansive admixtures used for the filling of voids or cracks with large openings on masonry or concrete.
Gluing metal sheets on concrete	Stainless steel sheets usually 1.00-1.50 mm thick covering with an epoxy resin layer
Welding of new reinforcement	Low-alloy steel is preferred as new reinforcement because it may be welded more easily. New bars are welded on the old ones with the aid of connecting bars
Gluing Fiber-Reinforced Plastic (FRP) sheets on concrete	Similar to steel sheets fiber reinforced plastic sheets (glass, aramid and carbon fibers) are glued to structural members. Where dead weight, space or time restrictions exist, FRP are an attractive choice. But exposures can result in the weakening of the interface between FRP composites and concrete.

5.15 How to repair minor/ moderate cracking in a structural member?

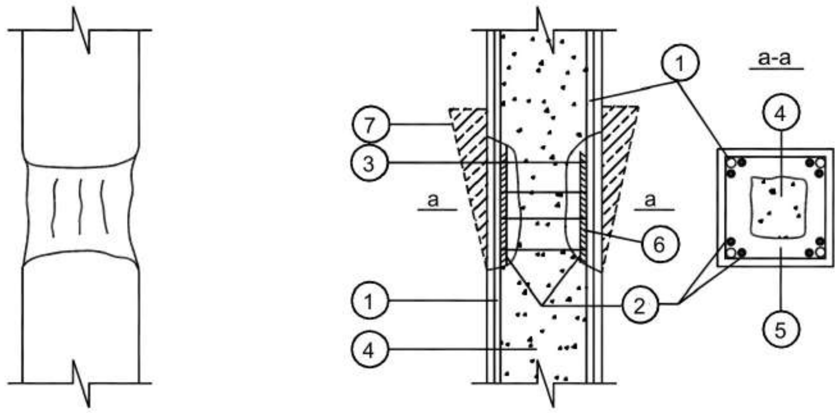
The process of repairing is similar to all structural members i.e. column, beam, beam-column joints; shear wall etc. as it depends on crack width. There are few instances and its repairing techniques are given here;

Post-earthquake condition of structural member	Repairing Technique(s)
Minor cracking	Epoxy resin injection from bottom proceeds upward through ports placed in drilled holes, spaced 20 to 100 cm
Moderate cracking	Cement grout injections from bottom proceeds upward through ports in drilled holes. Strength and compactness should be checked through appropriate testing.



1 - cracks; 2 - injection ports

***Repairing process for minor cracking*[26]**

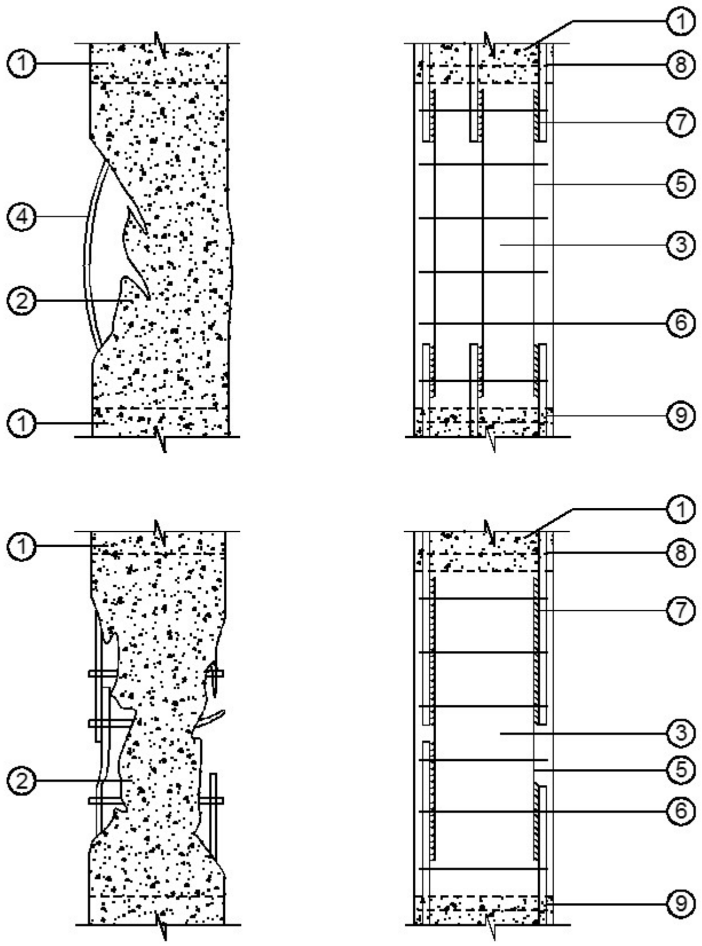


1 - existing reinforcement; 2 - added new reinforcement; 3 - added new ties;
4 - existing concrete; 5 - new concrete; 6 - welding; 7 - temporary castform

***Repairing process for moderate cracking*[26]**

5.16 How to repair severe cracking in a structural member?

Heavily damaged or crushed concrete	Replace with non-shrinkage concrete or concrete with low shrinkage properties. The temporary form and concrete should be higher than the finally required top level in order to compact the concrete sufficiently. After one day, the form can be removed and the fresh concrete that is beyond the normal configuration can be chipped away.
buckling of longitudinal reinforcement, ruptured ties and crushed concrete	Totally remove and replace the damaged parts, cut the buckled reinforcement and straighten, insert new longitudinal reinforcement and weld it to the existing reinforcement, insert new additional close ties in two piece welded to each other, place the new non-shrinkage concrete. Special attention must be paid to achieve good bond between the new and the existing concrete.

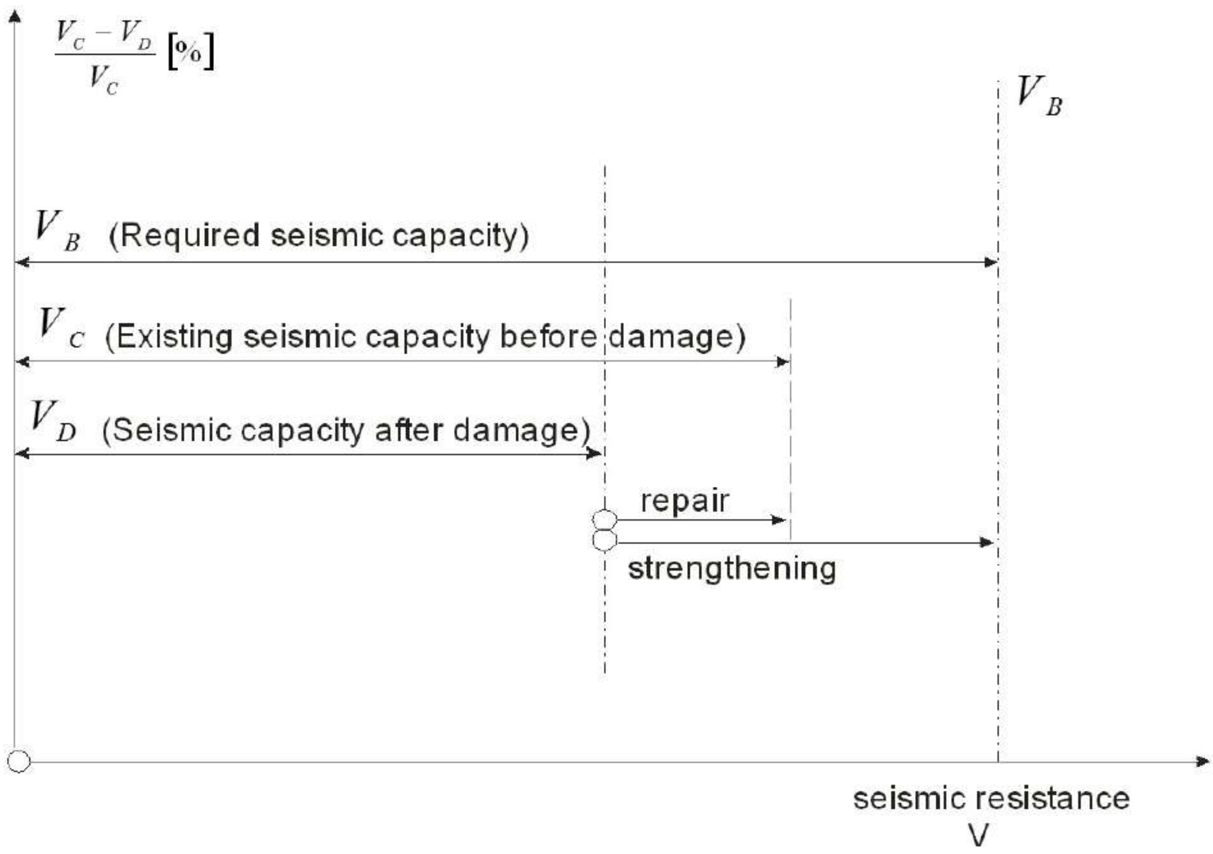


1 - existing non-damaged concrete; 2 - existing damaged concrete;
3 - new concrete; 4 - buckled reinforcement; 5 - new reinforcement;
6 - new ties; 7 - welding; 8 - existing ties; 9 - existing reinforcement

Repairing process for severe cracking[26]

5.17 Are the repairing techniques sufficient for a building in case of future earthquake?

Any repairing process is only helpful to retain the most original strength of any building. It is not helpful to increase beyond the original strength. Therefore, opt retrofitting techniques as explained in Part IV to increase its strength globally and locally wherever it is deficient.



***Required Repair and Strengthening*[28]**

Section

III

How

to

Analyze, Design, Evaluate and Retrofit Multistoried RC Framed Construction for Earthquake Forces?

Chapter 1

Estimation the Earthquake Forces by Equivalent Static Method and Modal Dynamic Analysis?

1.1 How to estimate the earthquake forces for a regular building?

Problem statement: The seismic response of a multistoried building during earthquake depends upon its configuration i.e. regular or irregular. IS: 1893 (Part 1): 2002 describes the various irregularities of a building in plan and elevation. But, the criteria to evaluate the regularity of building in plan and elevation are not explicitly mentioned. This is essential since the type of seismic analysis, the design consideration and the mathematical modeling of the building to evaluate the dynamic characteristics, depend upon it. For example; in case of irregular buildings, the structural analysis is based on a multi-modal dynamic analysis. For regular buildings a simplified procedure could be used, based on application of equivalent static lateral force. However, EC8 Part 1 Section 4.2.3 sets out quantified criteria for assessing structural regularity of the building and if a building meets these requirements, the later efforts for analyzing and designing of the building are reduced greatly.

EVALUATION OF REGULARITY

Regularity (or irregularity) is a crucial factor in the seismic performance of buildings as there is no well defined procedure to evaluate it. The approach presented in EC 8, especially to regularity in plan, is not straightforward, and application rules are not given to cover all details.

Criteria for Regularity in Plan[1,7]

A building may be called regular in plan if it satisfies the following criteria;

- ✓ The lateral load resisting element and mass should be symmetrically distributed (approximately) with respect to principal axis of the building
- ✓ The plan of the building should be in compact form i.e. perimeter line is always convex. If the building has some reentrant corners, it may still be regular if the area of each re-entrant portion is not more than 5% of total floor area i.e. it does not hamper or reduce the in-plane stiffness of the floor.
- ✓ There should be a rigid diaphragm action at each floor i.e. the in-plane stiffness of the floor/roof is much larger as compared to vertical resisting elements at that storey so that lateral forces are proportional to lateral stiffness of each vertical load resisting elements. Care must be taken in case of building having L, C, H, I and x type plan.
- ✓ The ratio of longer side to shorter sides in plan does not exceed 4.

- ✓ The structural eccentricity e_0 and torsional radius r at each level and for each direction should satisfy the following criteria as mentioned below, for example if direction of analysis in y-direction;

$$e_{ox} \leq 0.30 r_x$$
$$r_x \geq l_s$$

Similarly, if the direction of analysis in x-direction

$$e_{oy} \leq 0.30 r_y$$
$$r_y \geq l_s$$

Where,

e_{ox} or e_{oy} Is the distance between the center of stiffness and center of mass, measured along the x- direction and y-direction respectively i.e. normal to the direction of analysis considered; in case of multi-storied building, the approximate method for calculation is

$$x_{cs} \approx \sqrt{\sum \frac{(xEI_y)}{EI_y}}$$

$$y_{cs} \approx \sqrt{\sum \frac{(yEI_x)}{EI_x}}$$

r_x or r_y Torsional radius, r in x and y-direction; it is the square root of the ratio of the torsional stiffness (rotation unit moment) to the lateral stiffness (deflection per unit force)i.e. in case of multi-storied building, the approximate method for calculation is

$$r_x \approx \sqrt{\sum \frac{(x^2EI_y + y^2EI_x)}{\sum (EI_y)}}$$

$$r_y \approx \sqrt{\sum \frac{(x^2EI_y + y^2EI_x)}{\sum (EI_x)}}$$

l_s Is the radius of gyration of the floor mass in plan ; is the square root of the ratio of the polar moment of inertia to the floor mass in plan, the polar moment of inertia being calculated about the center of mass. For a rectangular building of side l and b and a uniform mass distribution, the radius of gyration is

$$l_s = \sqrt{(l^2 + b^2)/12}$$

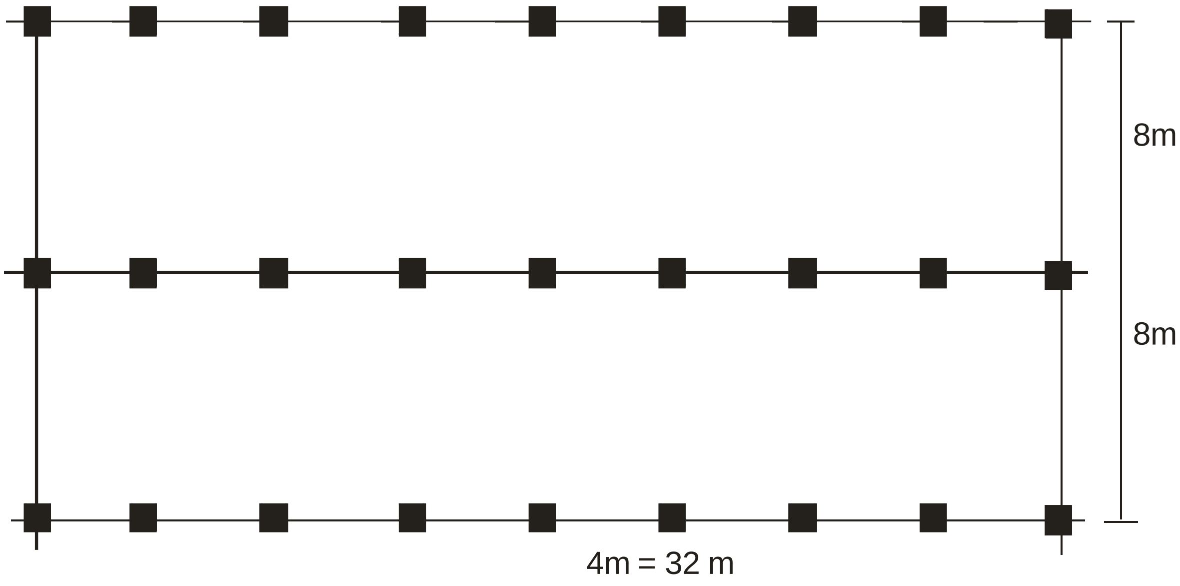
Criteria for regularity in elevation

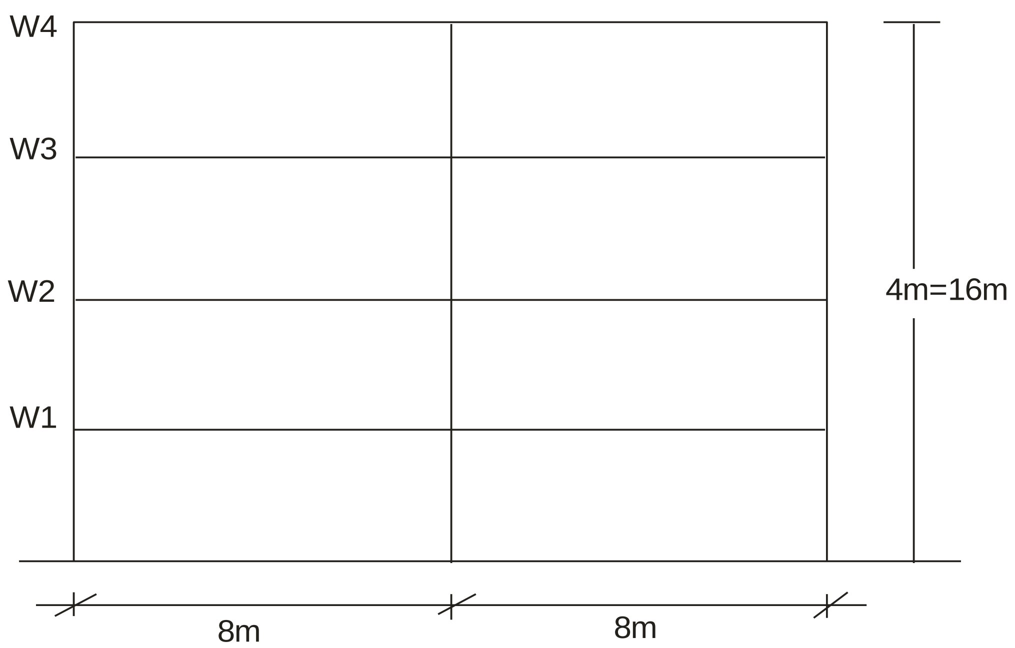
A building may be called regular in elevation if it satisfies the following criteria;

- ✓ All lateral loads resisting systems in the building should continue from foundation to roof without interruption.
- ✓ Mass and lateral stiffness of each storey should nearly be constant or reduce gradually without abrupt changes along the height of the building. Quantification of mass and stiffness irregularity as per IS 1893 (Part 1): 2002.
- ✓ In moment resisting frame buildings, the actual storey stiffness should be nearly same as analyzed and designed. Generally actual stiffness is much higher due to insertion of infill wall after the construction of frame.
- ✓ There are no setbacks in the building or their limits as per IS 1893 (Part 1): 2002. The total reduction in width from top to bottom on any face may not exceed 30%, with not more than 10% at any level compared to the level below. However, an overall reduction in width up to half is permissible within the lowest 15% of the height of building.

Example for the determination of Earthquake Forces as per Equivalent Static Method for Regular Buildings

The following example considers a four storey office/residential building founded on a hard soil and situated in Zone V in India, which represents a relatively active seismic area. The principal lateral force resisting system is concrete moment resisting frame. An equivalent static method has been used to determine the seismic forces along with the other response quantity.





(1) Determine Seismic weight at various floors

Assume

The dead load per unit area of floor is estimated to be = $6.75\,kN/m^2$

The normal line load = $3kN/m^2$

The seismic weight at various floors

$W_1 = W_2 = W_3 = 16\times 32(6.75+0.25\times 3) = 3840kN$, Assume 25% of live load

$W_4 = 16\times 32\times(6.75) = 3456kN$, Assume no live load considered at roof

Total seismic weight of the building (W) = $3840\times 3+3456 = 14976\,kN \approx 15000kN$

(2) Determination of Fundamental Period

$T = 0.075h^{0.75} = 0.075\times 16^{0.75} = 0.075\times 16^{0.75} = 0.6$ sec, where h = total height of building.

(3) Determination of Design Seismic Base Shear Force

$V_\beta = A_h W$

A_h (Design horizontal acceleration spectrum) $= \frac{Z}{2}.\frac{I}{R}.\frac{S_a}{g} = \frac{0.36}{2}.\frac{1}{5}.1.67 = 0.06$

Assume

$Z = 0.36(corresponds\ to\ Zone\ V)$

$I = 1.0\ (for\ ordinary\ structures)$

$R = 5.0(SMRF)$

$\frac{S_a}{g} = \frac{1.0}{T}\ for\ rocky\ or\ hard\ soil\ site\ 0.40 \le T \le 4.0 = 1.67$

Therefore,

$V_b = .06\times 15000 = \quad = 900kN$

(4) Determination of Lateral force at Each Floor

$$Q_i = V_B \frac{W_i h_i^2}{\sum W_i h_i^2}$$

Level	W_i (kN)	h_i (m)	$W_i h_i^2$ $(kN-m^2)$	Q_i (kN)	V_i (kN)
4roof	3456	16	884736	456.34	456.34
3rd floor	3840	12	552960	28521	741.55
2nd floor	3840	08	245760	126.76	868.31
1st floor	3840	04	61440	31.70	900.00
			$\sum$=1,744,896	$\sum$=900	

(5) Shear force at storey

$$V_i = Q_{roof} + \sum_{i=x}^{N} Q_i$$

(6) Lateral displacement δx and storey drift.

Assume building is a shear building; in this case the stiffness for a column between two consecutive floors is given by

$$K = \frac{12EI}{L^3} = \frac{12 \times 25 \times .002133}{4^3} = 0.01x10^6$$

Where,

$$h = 4m, E = 25kN/m^2$$

$$I = \frac{1}{12} \times 0.4 \times 0.4^3 = 0.002133 m^4$$

Total stiffness at each floor = $27 \times 0.01x10^6 = 0.27 \times 10^6\ kN/m^2$

Level	Storey shear (kN)	Storey Stiffness (kN/m²)	Storey drift $\Delta x(mm)$	Lateral Drift (mm)
4roof	456.34	0.27×10^6	1.69	11.00
3rd floor	741.55	0.27×10^6	2.75	9.81
2nd floor	868.31	0.27×10^6	3.22	6.55
1st floor	900.00	0.27×10^6	3.33	3.33
			$\sum = 11$	

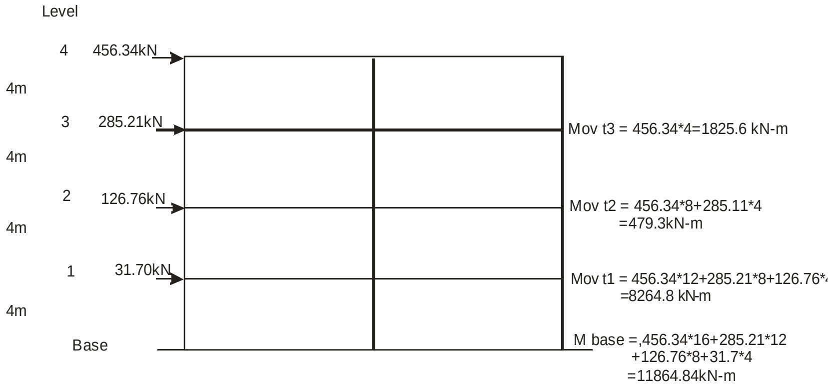
The code stipulates that storey drift

$\leq 0.004 \times Storey\ height\quad i.e\ 0.004 \times 4 = 0.016m = 16\,mm$

(7) Overturning moment and Accidental Tensional Moments

The overturning moment at each level of the building is given by

$$M_x = F_i(h_N - h_x) + \sum_{i=1}^{N} Fi(h_i - h_x)$$



(8) Accidental Torsional Moments.

Level	Storey shear (V_i), (kN)	Torsional Moments T_x (kN-m)
4^{roof}	456.34	730.144
3^{rd} floor	741.55	1186.48
2^{nd} floor	868.31	1389.29
1^{st} floor	900.00	1440.00

$$T_x = 0.05 \times D \times V_i = .05 \times 32 \times V_i = 1.6 V_i$$

Further Reading

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5. Ciongradi, I. and Budescu, M. (2002)."Evaluation of the Seismic Torsion Effects on Structures", Ovidius University Annals of Constructions, Volume 1, Number 3,4 April 2002, Ovidius University Press.
6. Cosenza, E., Manfriedi, G. and Realfonzo, R. (2000). "Torsional Effects and Regularity Conditions in RC buildings" 12th WCEE, NewZealand.
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8. Maguire, J.R. and Wyatt, T.A. (2002), "Dynamics; An Introduction for Civil and Structural Engineers" Thomas Telford, London.
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1.2 How to carry out modal dynamic analysis of a building with regular or irregular configuration?

Problem Statement: A building that lacks symmetry and has discontinuity in geometry, mass, or load resisting elements is called irregular. These irregularities may cause interruption of force flow and stress concentrations. The section 7 of IS 1893 (Part 1): 2002 enlists the irregularity in building configuration system. It is recommended by the codes in Clause 7.8 the dynamic analysis procedure shall be performed to obtain the design seismic forces, and its distribution to different levels along the height of the building and to the various lateral load resisting elements.

Criteria for irregularity in building configuration[1,4]

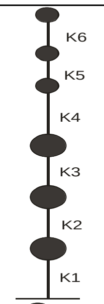
Section 7 of IS 1893 (Part 1): 2002 enlists the irregularity in building configuration system. These irregularities are categorised in two types; Vertical irregularities referring to sudden change of strength, stiffness, geometry and mass, results in irregular distribution of forces and/or deformation over the height of building and Horizontal irregularities which refer to asymmetrical plan shapes (e.g. L-, T-, U-, F-) or discontinuities in the horizontal resisting elements (diaphragms) such as cut-outs, large openings, re-entrant corners and other abrupt changes resulting in torsion, diaphragm deformations, stress concentration. These irregularities are summarized in Table 1. It is recommended that the multi-storied reinforced concrete buildings with vertical irregularities should be designed on the basis of dynamic analysis and the building configuration may be modelled as a system of mass lumped at the floor levels with each mass having one degree of freedom, that of lateral displacement in the direction under consideration (Clause 7.8.4.5 of IS: 1893 (Part 1): 2002). Here a few examples have been solved to consider the various vertical irregularities like soft stories, mass irregularities and weak storey or strength irregularities. The dynamic analyses in these examples have been carried out as per IS 1893 (Part 1): 2002 given in clauses 7.8.

Table 1: Vertical and Horizontal Irregularities in RC buildings[2,3]

Vertical Irregularities

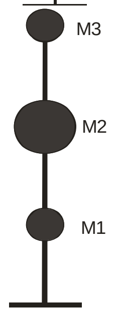
1. Stiffness irregularity – Soft story

A soft story is one in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above.
Use dynamic analysis to determine lateral-force distribution



2. Weight (mass) irregularity

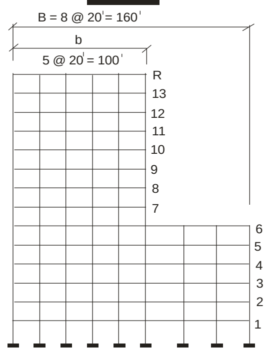
Mass irregularity shall be considered to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
Use dynamic analysis to determine lateral-force distribution



3. Vertical geometric irregularity

Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral-force- resisting system in any story is more than 130% of that in an adjacent story. One-story penthouses need not be considered.

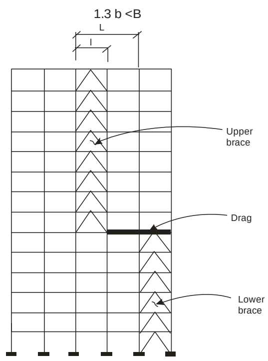
Use dynamic analysis to determine lateral-force distribution



4. In-plane discontinuity in vertical lateral-force-resisting element

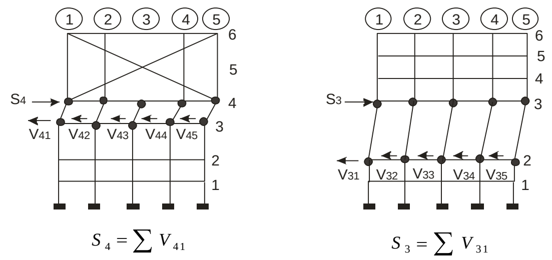
An in-plane offset of the lateral-load-resisting elements is greater than the length of those elements.

Use special seismic load combinations for members below discontinuity



5. Discontinuity in capacity – weak story

A weak story is one in which the story strength is less than 80% of that in the story above. The story strength is the total strength- resisting elements sharing the story shear for the direction under consideration.
Increase seismic loads for members below discontinuity by a factor $= \Omega_o$.

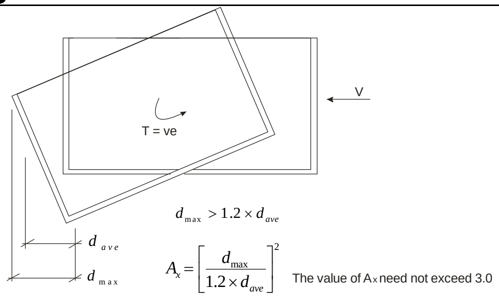


Horizontal Irregularities

1.Torsional irregularity- to be considered when diaphragms are not flexible

Torsional irregularity shall be considered to exist when the maximum story drift, computed including accidental torsion, at one end of the structure transverse to an axis is more than 1.2 times the average of the story drifts of the two ends of the structure.

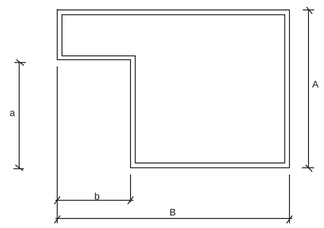
Increase torsional forces by an amplification factor A_x .



2. Re-entrant corners

Plan configurations of a structure and its lateral-force-resisting system contain reentrant corners, where both projections of the structure beyond a reentrant corner are greater than 15% of the plan dimension of the structure in the given direction.

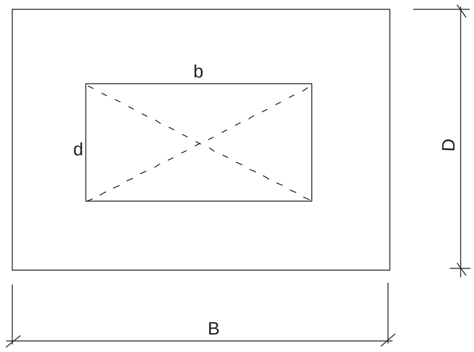
Provide structural elements in diaphragms to resist flapping actions



3. Diaphragm discontinuity.

Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 50% of the gross enclosed area of the diaphragm, or changes in effective diaphragm stiffness of more than 50% from one story to the next.

Provide structural elements to transfer forces into the diaphragm and structural system. Reinforce boundaries at openings

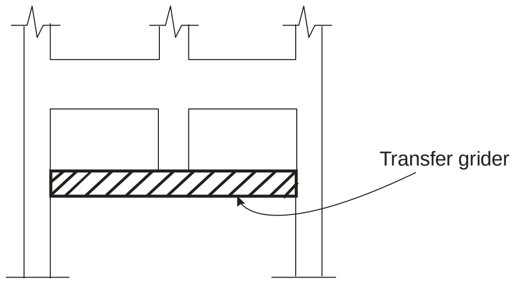


$$0.5(B \times D) < (b \times d)$$

4. Out-of-plane offsets

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

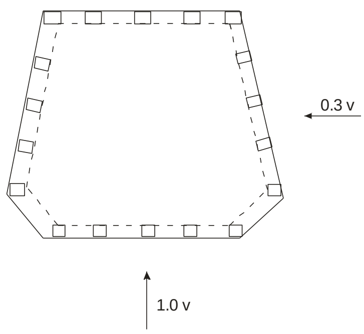
Use special seismic load combinations. One-third increase in stress not permitted



5. Nonparallel systems

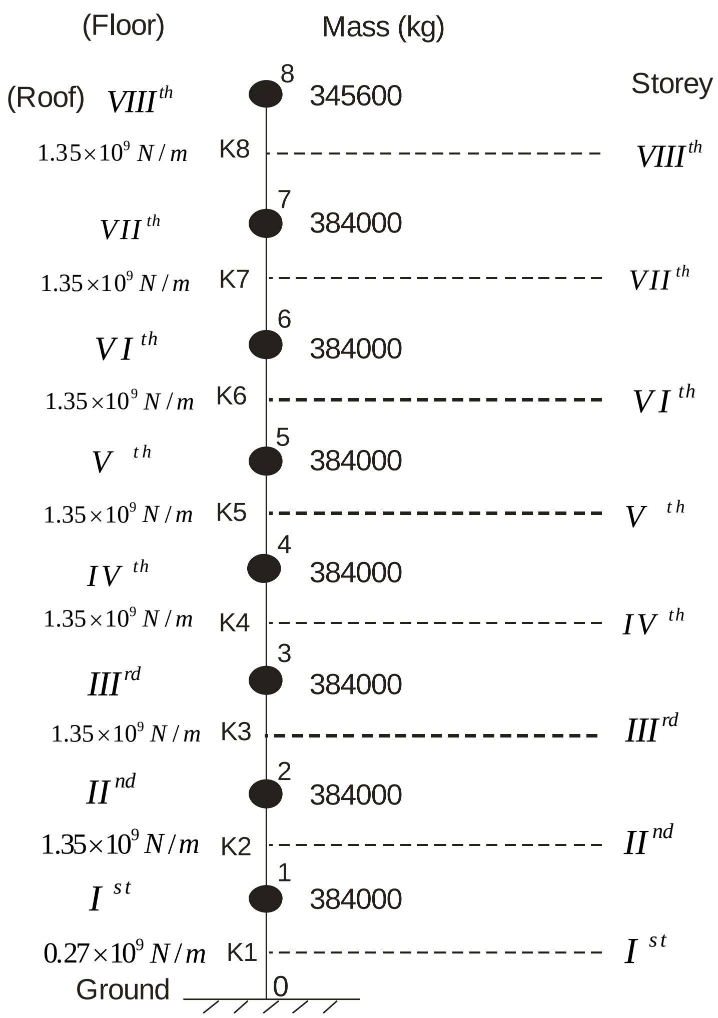
The vertical lateral-load-resisting elements are not parallel to or symmetric about the major orthogonal axes of the lateral-force- resisting system.

Design for orthogonal effects



Example of dynamic analysis of eight storied RC building with soft storey

A dynamic analysis of an eight storied RC building with soft storey whose lumped mass model as shown below has been carried out using mode superposition method. In this example, the analysis is performed to determine the base shear for each mode using building characteristics and design spectra as per IS 1893 (Part 1): 2002. The lateral forces at each floor level, storey shear, floor acceleration, displacement and lateral drift are calculated for each mode, and are combined statistically using the SRSS combinations.

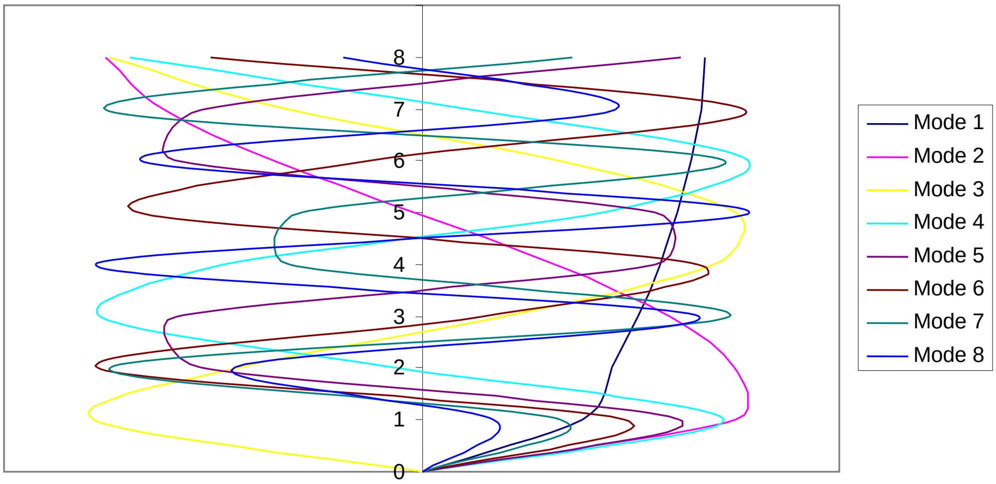


Determination of Time Period in Each Mode

Time Period	Mode 1 T_1	Mode 2 T_2	Mode 3 T_3	Mode 4 T_4	Mode 5 T_5	Mode 6 T_6	Mode 7 T_7	Mode 8 T_8
T (sec)	0.8080	0.2352	0.1320	0.0931	0.0739	0.0632	0.0571	0.0539

Determination of Mode Shape (ϕ_{ik}) : Normalized to Mass

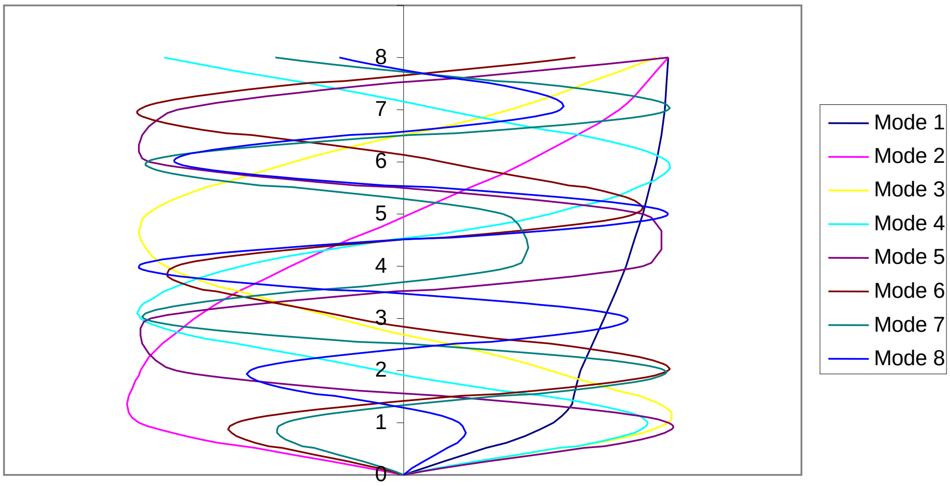
Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
ϕ_{i1}	ϕ_{i2}	ϕ_{i3}	ϕ_{i4}	ϕ_{i5}	ϕ_{i6}	ϕ_{i7}	ϕ_{i8}
0.6795	-0.7566	-0.7498	-0.7007	0.6194	-0.5062	0.3610	-0.1886
0.6689	-0.6184	-0.3150	0.1158	-0.5260	0.7723	-0.7557	0.4656
0.6469	-0.3549	0.3226	0.7824	-0.5907	-0.1165	0.7248	-0.6748
0.6138	-0.0193	0.7525	0.4358	0.5583	-0.6783	-0.2856	0.7855
0.5701	0.3200	0.6976	-0.4750	0.5602	0.6636	-0.3144	-0.7817
0.5166	0.5946	0.1932	-0.7708	-0.5889	0.1432	0.7373	0.6640
0.4542	0.7486	-0.4356	-0.0685	-0.5280	-0.7791	-0.7450	-0.4494
0.3840	0.7507	-0.7838	0.7225	0.6178	0.4850	0.3330	0.1693



MODE SHAPE NORMALIZED TO MASS

Determination of Mode Shape (ϕ_{ik}) : Normalized to Unity

Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
ϕ_{i1}	ϕ_{i2}	ϕ_{i3}	ϕ_{i4}	ϕ_{i5}	ϕ_{i6}	ϕ_{i7}	ϕ_{i8}
1.0000	1.000	0.9566	-0.8955	1.0000	0.6497	-0.4777	-0.2401
0.9845	0.8174	0.4019	0.1480	-0.8491	-0.9913	1.0000	0.5927
0.9521	0.4691	-0.4116	1.0000	-0.9535	0.1496	-0.9590	-0.8590
0.9033	0.0256	-0.9600	0.5570	0.9013	0.8706	0.3779	1.0000
0.8390	-0.4230	-0.8899	-0.6071	0.9043	-0.8517	0.4160	-0.9951
0.7602	-0.7859	-0.2465	-0.9852	-0.9507	-0.1838	-0.9756	0.8452
0.6684	-0.9894	0.5557	-0.0875	-0.8523	1.0000	0.9858	-0.5721
0.5651	-0.9922	1.0000	0.9234	0.9973	-0.6225	-0.4407	0.2155

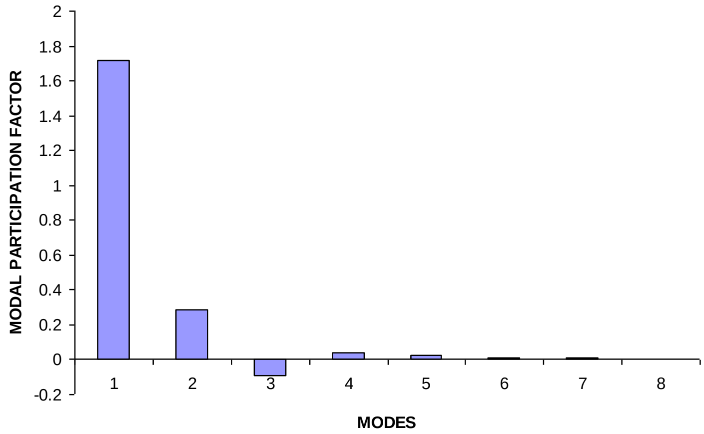


MODE SHAPE NORMALIZED TO UNITY

To obtain the time periods (T) and mode shape $\{\phi\}$, un-damped free vibration analysis of the entire building has been performed as per established methods of mechanics using the appropriate masses and elastic stiffness of the structural system (Clause 7.8.4.1 of IS: 1893 (Part 1): 2002)

Modal Participation Factor p_k in kth Mode:

Modal Participation Factor	Mode 1* P_1	Mode 2 P_2	Mode 3 P_3	Mode 4 P_4	Mode 5 P_5	Mode 6 P_6	Mode 7 P_7	Mode 8 P_8
p_k	1.715	0.284	-0.093	0.042	0.023	0.013	0.007	0.003



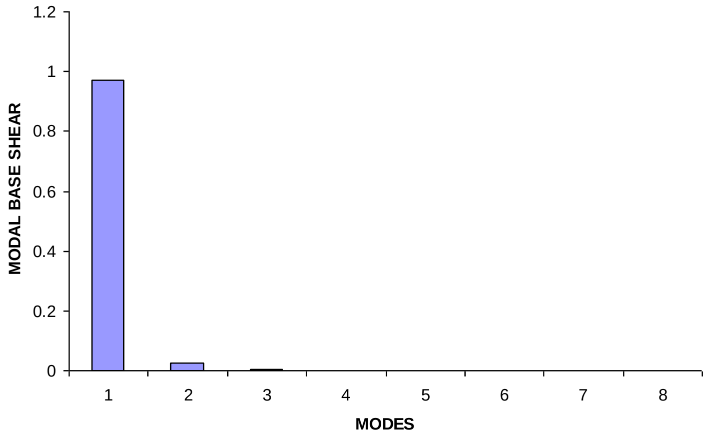
Level	m_i (kg)	ϕ_i	$m_i\phi_i$	ϕ_i^2	$m_i\phi_i^2$
8	345600	0.6795	234835.2	0.46172	159570.5
7	384000	0.6689	256857.6	0.447427	171812
6	384000	0.6469	248409.6	0.41848	160696.2
5	384000	0.6138	235699.2	0.37675	144672.2
4	384000	0.5701	218918.4	0.325014	124805.4
3	384000	0.5166	198374.4	0.266876	102480.2
2	384000	0.4542	174412.8	0.206298	79218.29
1	384000	0.384	147456	0.147456	56623.1
Sum	**3033600**		**1714963**		**999877.9**

$$p_k = \frac{\sum_{k=1}^{n} m_i \phi_{ik}}{\sum_{k=1}^{n} m_i \phi_{ik}^2} \quad , \quad P_1 = \frac{1714963}{999877.9} = 1.715$$

Similarly, can be calculated in other modes

Determination of Mode Base Shear Participation factor α_k in k[th] Mode:

(P.F)	Mode 1* α_1	Mode 2 α_2	Mode 3 α_3	Mode 4 α_4	Mode 5 α_5	Mode 6 α_6	Mode 7 α_7	Mode 8 α_8
α_k	0.9696	0.0266	0.0028	0.0006	0.0002	0.00005	0.00001	0.000004



Level	m_i (kg)	ϕ_i	$m_i\phi_i$	$\phi_i^{\,2}$	$m_i\phi_i^{\,2}$
8	345600	0.6795	234835.2	0.46172	159570.5
7	384000	0.6689	256857.6	0.447427	171812
6	384000	0.6469	248409.6	0.41848	160696.2
5	384000	0.6138	235699.2	0.37675	144672.2
4	384000	0.5701	218918.4	0.325014	124805.4
3	384000	0.5166	198374.4	0.266876	102480.2
2	384000	0.4542	174412.8	0.206298	79218.29
1	384000	0.384	147456	0.147456	56623.1
Sum	**3033600**		**1714963**		**999877.9**

$$\alpha_k = \frac{\left(\sum_{i=1}^{n} m_i\,\phi_{ik}\right)^2}{\sum_{i=1}^{n} m_i \sum_{i=1}^{n} m_i\phi_{ik}^{\,2}} \quad , \quad \alpha_1 = \frac{(1741056)^2}{(3.72000\times 1017608)} = 0.9696$$

Similarly, can be calculated in other modes

Determination of Horizontal Seismic Coefficient $(Ah)_k$ in k^th Mode

Horizontal Seismic Coefficient	Mode 1 $(Ah)_1$	Mode 2 $(Ah)_2$	Mode 3 $(Ah)_3$	Mode 4 $(Ah)_4$	Mode 5 $(Ah)_5$	Mode 6 $(Ah)_6$	Mode 7 $(Ah)_7$	Mode 8 $(Ah)_8$
$(Ah)_k$	0.0445	0.09	0.09	0.0862	0.075	0.070	0.066	0.066

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

$$(Ah)_1 = \frac{0.36}{2} \cdot \frac{1}{5} \cdot 1.237 = 0.0445$$

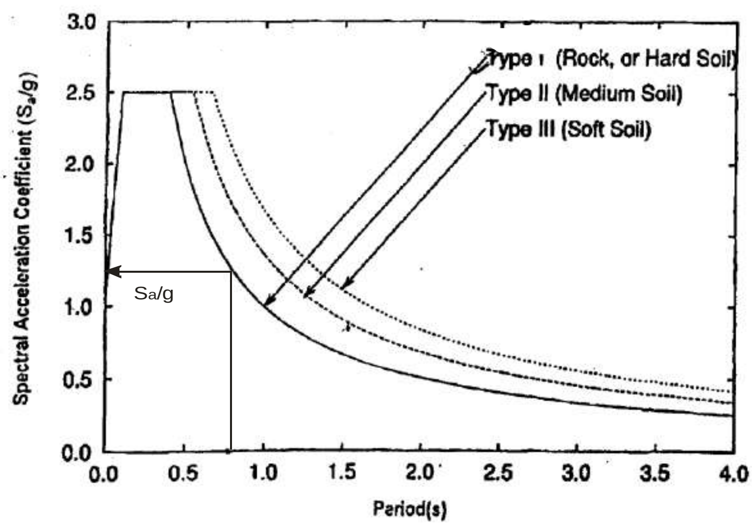
Where,

Z = Seismic zone factor = 0.36,

I = Importance factor = 1.0,

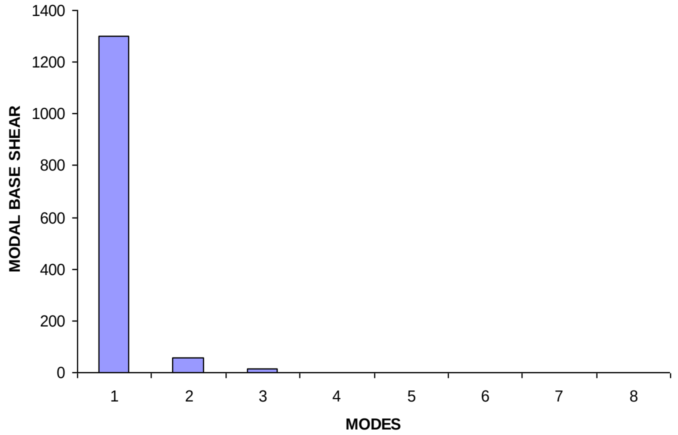
R = Response reduction factor = 5.0,

$\frac{S_a}{g}$ = Spectral horizontal seismic coefficient correspond to T_1 = 0.808= 1.237

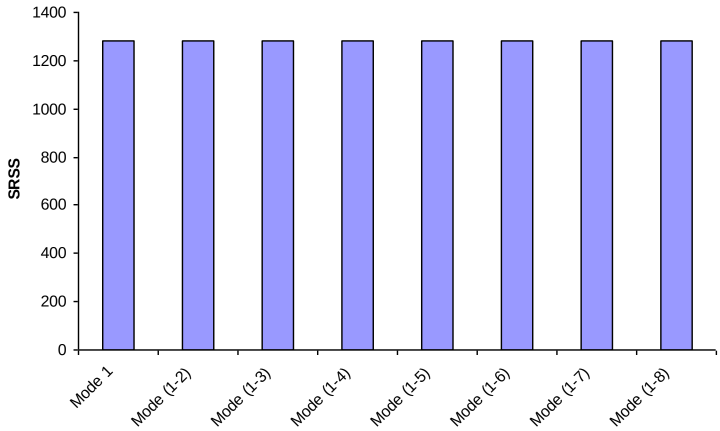


Determination of Modal Base Shear (V_k) in k[th] Mode:

Modal Base Shear	Mode 1* V_1	Mode 2 V_2	Mode 3 V_3	Mode 4 V_4	Mode 5 V_5	Mode 6 V_6	Mode 7 V_7	Mode 8 V_8	SRSS V_b
V_k (kN)	1282.77	71.296	7.702	1.551	0.392	0.121	0.036	0.007	1284.77



Base shear in each mode



Base shear

$$V_k = \alpha_k (Ah)_k W$$

$$V_1 = 0.9696 \times 0.0445 \times 3033600 \times 9.8/1000 = 1282.77 kN$$

Determination of Modal Lateral Forces (Q_{ik}) in kth Mode:

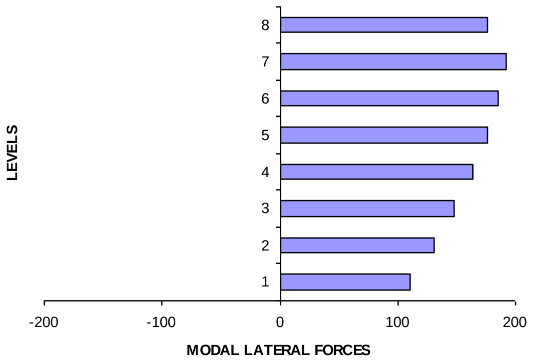
Lateral Forces in each Mode (kN)

Levels	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8
8	175.653	-65.574	21.359	-8.765	3.635	-1.595	0.598	-0.139
7	192.126	-59.552	9.970	1.609	-3.430	2.704	-1.391	0.383
6	185.807	-34.176	-10.211	10.875	-3.852	-0.408	1.334	-0.555
5	176.299	-1.858	-23.818	6.057	3.640	-2.375	-0.525	0.646
4	163.748	30.816	-22.080	-6.602	3.653	2.324	-0.578	-0.643
3	148.381	57.260	-6.115	-10.713	-3.840	0.501	1.357	0.546
2	130.458	72.090	13.787	-0.952	-3.443	-2.728	-1.371	-0.369
1	110.295	72.292	24.809	10.042	4.028	1.699	0.613	0.139

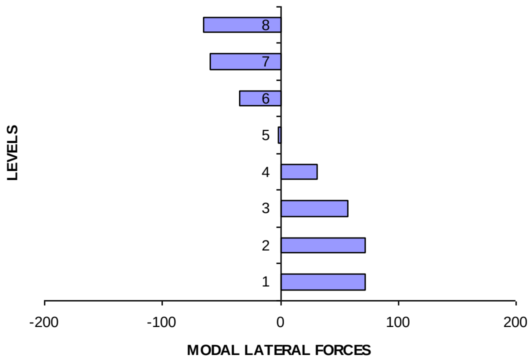
$$Q_{ik} = (Ah)_k . \phi_{ik} . p_k . W_i$$

$(Ah)_k$	ϕ_{ik}	P_k	W_i
0.0445	0.6795	1.71093	345600

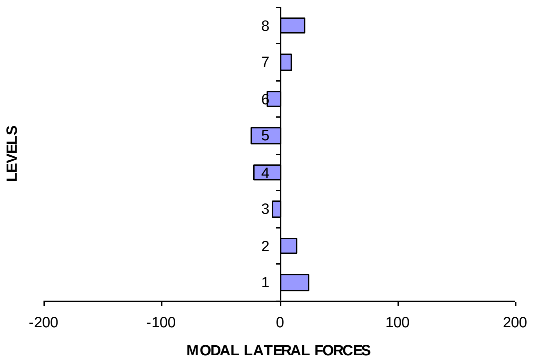
$$Q_{81} = (0.0445 \times 0.6795 \times 1.715 \times 345600 \times 9.8) / 1000 = 175.653\,kN$$



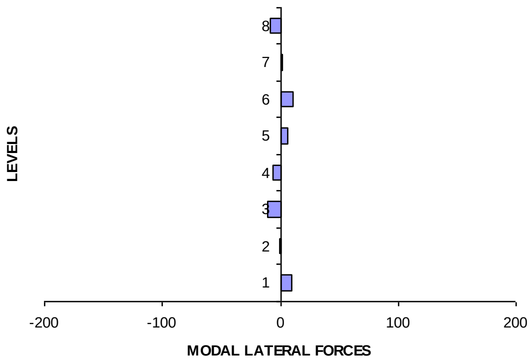
Mode 1



Mode 2



Mode 3



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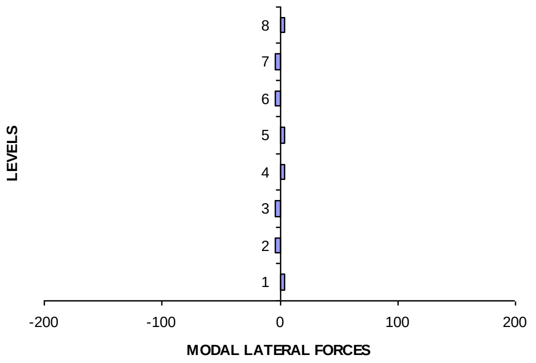

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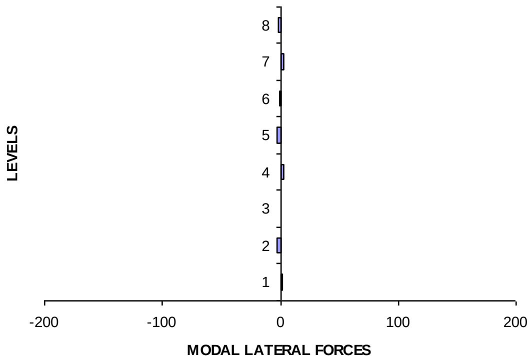

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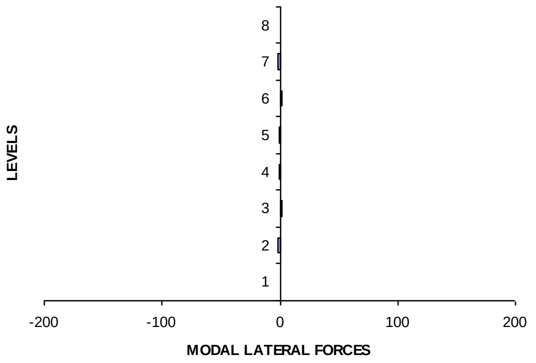

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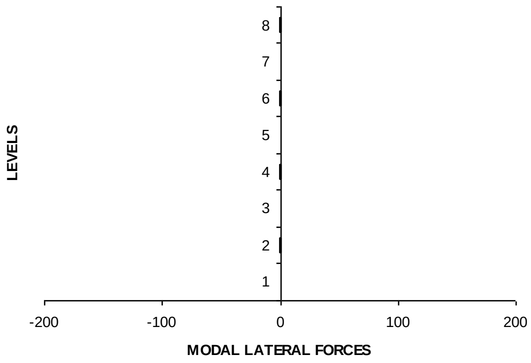
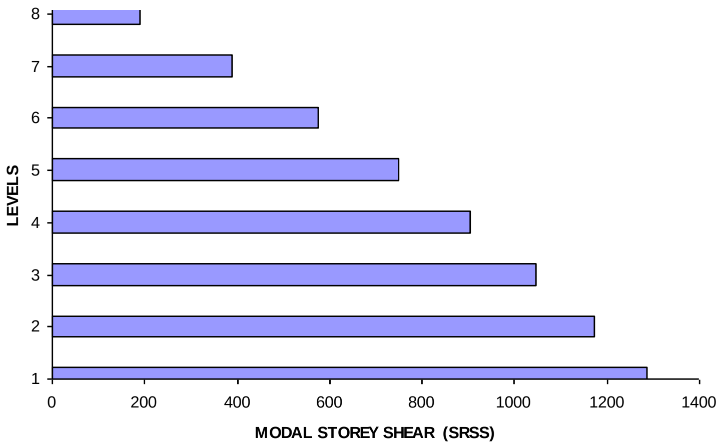


Mode 8

Determination of Modal Storey Shear at J^{th} Storey in k^{th} Mode:

Levels	Mode 1*	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	SRSS
8	175.653	-65.574	21.359	-8.765	3.635	-1.595	0.598	-0.139	188.95
7	367.779	-125.126	31.330	-7.156	0.205	1.109	-0.793	0.243	389.81
6	553.586	-159.30	21.119	3.719	-3.647	0.701	0.5412	-0.3118	576.46
5	729.886	-161.162	-2.699	9.777	-0.0061	-1.674	0.0154	0.334	747.53
4	893.634	-130.34	-24.779	3.174	3.647	0.649	-0.563	-0.30	903.44
3	1042.01	-73.086	-30.895	-7.540	-0.193	1.151	0.794	0.237	1045.06
2	1172.475	-0.996	-17.107	-8.492	-3.636	-1.577	-0.577	-0.132	1172.63
1	1282.77	71.297	7.70162	1.551	0.392	0.121	0.036	0.007	1284.77

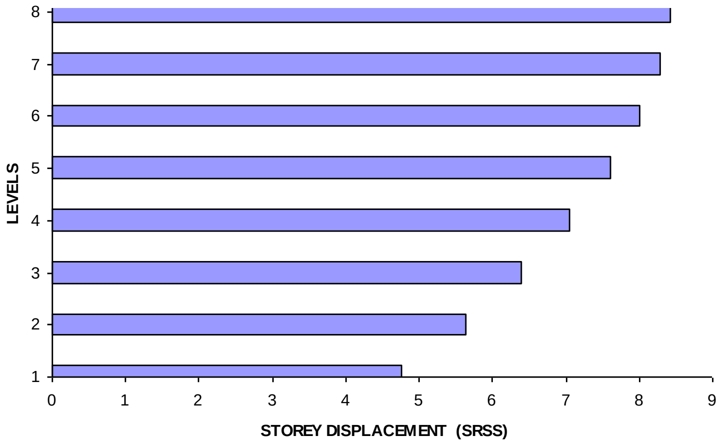


$$V_{jk} = \sum_{i=j}^{n} Q_{ik}$$

$$V_{11} = 175.653 + 192.126 + 185.807 + 176.299 + 163.748 + 148.381 + 130.458 + 110.295 = 1282.77\ kN$$

Determination of Storey Displacement at i^{th} level in k^{th} Mode in (mm):

Levels	Mode 1*	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	SRSS
8	8.411	-0.266	0.027	-0.005	0.001	-0.0004	0.0001	-0.00002	8.415
7	8.280	-0.217	0.011	0.0009	-0.001	0.0007	-0.0002	0.00007	8.282
6	8.008	-0.124	-0.011	0.006	-0.001	-0.0001	0.0002	-0.0001	8.008
5	7.598	-0.006	-0.027	0.040	0.001	-0.0006	-0.0001	0.0001	7.598
4	7.057	0.112	-0.025	-0.003	0.001	0.0006	-0.0001	-0.0001	7.057
3	6.395	0.209	-0.007	-0.006	-0.001	0.0001	0.0002	0.0001	6.398
2	5.622	0.263	0.015	-0.0005	-0.001	-0.0007	-0.0002	-0.00007	5.628
1	4.753	0.264	0.028	0.005	0.001	0.0004	0.0001	0.00002	4.760



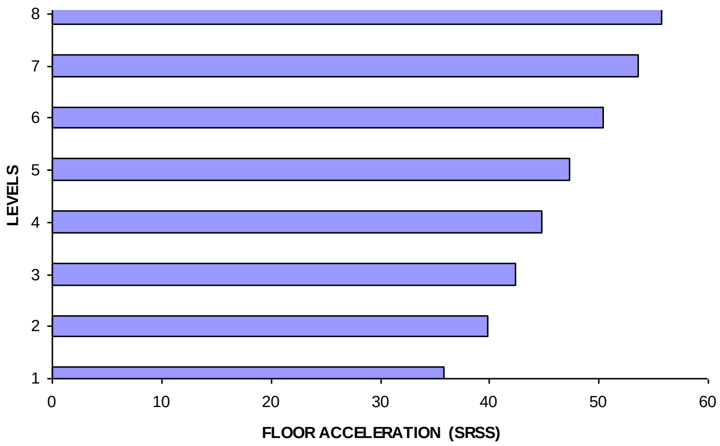
$$\delta_{ik} = p_k.\phi_k.(Ah)_k.\left(\frac{T_k}{2\pi}\right)^2 \times 9.81m$$

p_k	ϕ_{ik}	$(Ah)_k$	T_k
1.715	0.6795	0.0445	0.808

$$\delta_{81} = 1.715 \times 0.6795 \times 0.0445 \times \left(\frac{0.808}{2 \times 3.142}\right)^2 \times 9.81 \times 1000 = 8.411\ mm$$

Determination of Floor Acceleration at i^th level in k^th Mode in (mg):

Levels	Mode 1	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	SRSS
8	51.862	-19.361	6.306	-2.588	1.073	-0.471	0.176	-0.041	55.788
7	51.053	-15.824	2.649	0.427	-0.911	0.718	-0.369	0.101	53.530
6	49.374	-9.081	-2.713	2.889	-1.023	-0.108	0.354	-0.147	50.370
5	46.848	-0.493	-6.329	1.609	0.967	-0.631	-0.139	0.171	47.318
4	43.512	8.188	-5.867	-1.754	0.970	0.617	-0.153	-0.170	44.712
3	39.429	15.215	-1.625	-2.846	-1.020	0.133	0.360	0.145	42.403
2	34.666	19.156	3.663	-0.253	-0.914	-0.725	-0.364	-0.098	39.795
1	29.308	19.210	6.592	2.668	1.070	0.451	0.162	0.037	35.776



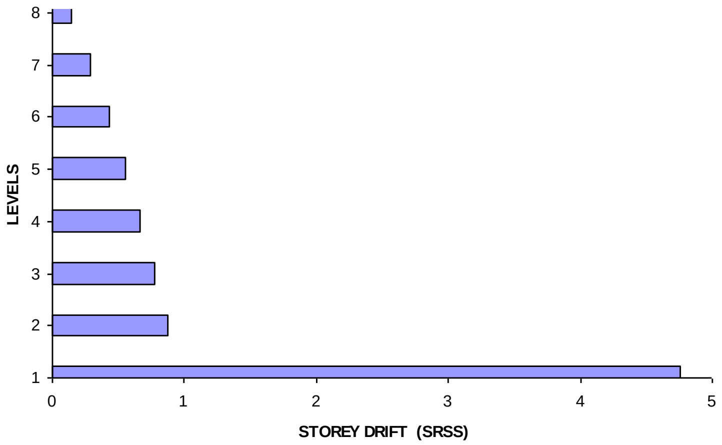
$$F_{ik} = p_k . \phi_{ik} . (Ah)_k$$

p_k	ϕ_{ik}	$(Ah)_k$
1.715	**0.6795**	**0.0445**

$$F_{81} = 1.715 \times 0.6795 \times 0.0445 \times 1000 = 51.862 \ mg$$

Determination of Storey Drift at the level of i[th] Storey (mm)

Levels	Mode 1*	Mode 2	Mode 3	Mode 4	Mode 5	Mode 6	Mode 7	Mode 8	SRSS
8	0.131	-0.048	0.015	-0.006	0.002	-0.001	0.0004	-0.0001	0.140
7	0.272	-0.092	0.023	-0.005	0.0001	0.0008	-0.0005	0.0001	0.288
6	0.409	-0.118	0.015	-0.033	-0.002	0.0005	0.0004	-0.0002	0.427
5	0.540	-0.119	-0.001	0.043	-0.000004	-0.001	0.00001	0.0002	0.554
4	0.662	-0.096	-0.018	0.002	0.002	0.0004	-0.0004	-0.0002	0.669
3	0.772	-0.054	-0.022	-0.005	-0.0001	0.0008	0.0005	0.0001	0.774
2	0.869	-0.0007	-0.012	-0.006	-0.002	-0.001	-0.0004	-0.00009	0.869
1	4.753	0.2640	0.0285	0.005	0.001	0.0004	0.0001	0.00002	4.760



$D_{81} = \delta_{81} - \delta_{71} = 0.131$ mm

Further Reading

1. BIS (2002). "Criteria for Earthquake Resistant Design of Structures" IS 1893 (Part 1): 2002, Manak Bhawan, 9 Bhhadur Shah Zafar Marg, New Delhi.
2. Taranath, B.S.(2005). "Wind and Earthquake Resistant Buildings- Structural Analysis and Design ", Marcel Dekker.
3. Taranath, B.S.(1998). "Steel, Concrete, & Composite Design of Tall Buildings", McGraw-Hill.
4. Agarwal, P. and Shrinkhande, M. (2006). "Earthquake Resistant Design of Structures", Prentice Hall of India Ltd., New Delhi.

Chapter 2

Consideration the effect of structural and non-structural components in the seismic performance of RC building

2.1 How to create a mathematical model of a multi-storied RC building to determine its seismic response?

Problem statement: The exact time period and mode shape of a multi-storied reinforced concrete building of any configuration (either regular or irregular) may be determined by creating its mathematical model in any professional software. Mathematical model of a structure is idealization of its stiffness, mass distribution and energy dissipation so that its response to earthquake could be predicted with sufficient accuracy. Now–a-days, a number of commercial softwares particularly to analyze the buildings are available in the market in which a structural engineer can create a mathematical model with sufficient accuracy. Another range of commercial softwares is for general purpose based on Finite Element Methods in which any type of structure can be idealized. For a design professional, one should be well conversant with these software's so that one can create a mathematical model with sufficient accuracy. In this example a few general tips are presented to create a mathematical model of the building so that its actual behavior during the earthquake can be predicted. The principal issues in mathematical modeling of a building system are modeling of each component of the building system along with effect of soil flexibility and the assumption involved.

IDEALIZATION OF THE BUILDING

The mathematical modeling of the building involves[6];

1. Identification of building components is thought to contribute significantly to the stiffness/resistance of the building. For example; columns, beams, floor slabs, masonry infill and other masonry walls, retaining or basement walls, staircases etc. Although not normally considered to be a building component, soil may be included in this list. This set of components may be considered to constitute real structural system as opposed to the nominal structural system that may be found in the structural drawing in the building.

2. Idealization of real structural system by an assembly of finite elements, thought to represent the behavior of the real elements, connected at selected structural nodes. Different elements are available in the element library of the software to represent most of the real system components. It is not necessary that each software has dedicated elements for each physical member of the buildings. In such cases these members have been modeled by the other appropriate element available in element library of the software based on its physical behavior. For example; masonry walls or staircase, practically reasonable elements are not available in most of the softwares in such cases to model these components, elements close to their physical behavior will be selected. Some of the details,

uncertainties and assumptions made in the idealization of individual components will be discussed later.

3. Evaluation of the distribution of mass throughout the structure and idealization of this mass distribution in terms of lumped masses is placed at the nodes and estimation of energy dissipation potential through damping.

Modeling of Beam and Columns

Beam and column components are idealized by the two or three dimensional homogeneous, isotropic, linear elastic beam elements of uniform prismatic section. Flexure, shear and torsional deformation are accounted for. Sectional properties for each component are generally based upon gross section geometry and element stiffness is formulated upon centerlines geometry assuming rigid connection of beam and column line elements at the (in infinitesimally small) joints or nodes. The material stiffness (Young's modulus) of these elements is equal to the material stiffness of concrete alone. However, many simplifications are made in this idealization, reinforced concrete beams and columns are non-homogeneous (i.e. composite), non-isotropic components whose effective section properties depends upon the loading (e.g. degree of cracking), the loading history, and the distribution of steel at each section and therefore, in general vary along the length of member in a non-uniform manner. The relatively greater rigidity of the beam column joint (the panel zone) can significantly affect the behavior of the components. Nevertheless the simplifications assumed here are often practically necessarily that result in a reasonable distribution of beam and column stiffness, in a relative sense, that provide a practically accurate evaluation of the initial stiffness and member forces of reinforced concrete frame system.

Modeling of Cracked and Un-cracked Section

The effect of cracking in RC structures depends on the type of the structural element, the reinforcement ratio and the stress level. The rigidity of RC elements is evaluated by considering an average of moments of inertia i.e. between cracked and completely un-cracked sections or as between net and gross section, unless they clearly are stressed at such low levels or that shrinkage is so limited that cracking is not likely. However, the quantifying effect of cracking of RC elements is given as follows.

The global effect of cracking on the rigidity can be described by the ratio I_e / I_g , where I_g is the moment of inertia of the uncracked element, and I_e is an equivalent moment of inertia, determined so that it will yield deflections close to the deflections of the cracked element.

- The ratio I_e / I_g varies between the values 0.3-0.5 and 0.25-0.45 for the rectangular and T & L beams respectively. For the columns subject to high compressive strength i.e. $\left(\sigma = \frac{N}{A} > 0.5 f_{ck}\right)$ the value varies from 0.7 to 0.9 and for columns subject to low compressive strength i.e. $(\sigma = 0.2 f_{ck})$ the ratio varies in 0.5 to 0.7. Where A is the cross-sectional area of the un-cracked column, and f_{ck} denotes the characteristics (cube) strength of the concrete.

  Computations based on these values show that for the whole structure, a rough preliminary value of the ratio $I_e / I_g \cong 0.5$ *can be assumed.*

- The ratio I_e / I_g for Structural RC Coupled Walls depends on the type of shear reinforcement of the coupling beams.

  If regular reinforcement is used (stirrups): $\frac{I_e}{I_g} = \frac{0.2}{1+3(h_b / l_n)^2}$ and

  If diagonal reinforcement is used: $\frac{I_e}{I_g} = \frac{0.4}{1+3(h_b / l_n)^2}$, Where l_n the clear is span and h_b is the height of the coupling beams.

  Computations based on these formulae show that rough preliminary values of the ratio $I_e / I_g \cong 0.2$ *(regular reinforcement) and* $I_e / I_g \cong 0.4$ *(diagonal reinforcement) can be assumed.*

ASCE 41-06 also gives the effective stiffness values for cracked section in the table as follows for beams and for columns based on $\frac{P}{A_g f_c}$ value:

Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity.	Axial Rigidity
Bearn's-Non-prestressed	$0.5E_cI_g$	$0.4E_cA_w$	-
Beams-Prestressed	E_cI_g	0.4EcAw	-
Columns with Compression Due to Design Gravity Loads ≥ 0.5Agf'c	$0.7E_cI_g$	0.4EcAw	E_cA_g
Columns with Compression Due to Design Gravity Loads≤0.3Agf'c or with compression	$0.5E_cI_g$	$0.4EcA_w$	E_cA_g

Mathematical Modeling of Floor Slabs

Typically slabs are considered on rigid supports; these are analyzed and designed for gravity loads separately from the frame system. The floor slabs should be adequately represented in 3D models of the structure so that their dead loads and live loads are properly accounted for. Under seismic action floor slabs play an important role of transmitting inertial loads to the frame and tying together elements of the later into a 3D entity. To perform these roles, slabs should be adequately connected with their supporting beams, walls and columns. The slabs may be modeled as:

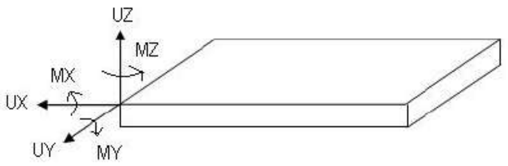
Rigid Diaphragm Constraint: Floor diaphragms in reinforced concrete (RC) buildings are typically modeled as rigid during the design phase and so the effect of in-plane diaphragm flexibility on the structure is often not considered. For the rigid diaphragm model, the diaphragm has equal in-plane displacements along its entire length under lateral load such that horizontal forces are transferred to the vertical LFRS proportional to the relative stiffness of each frame. A flexible diaphragm, however, exhibits in-plane bending due to lateral load, resulting in additional horizontal displacements along its length. This can lead to damage of the diaphragm due to high flexural stresses along its boundaries. This flexibility also increases the lateral load transfer to frames that were not designed to carry these additional lateral loads based on a rigid diaphragm model. If this effect is sizeable, it can lead to overloading of structural elements. While a rigid diaphragm model seems reasonable for structures that are nearly square additional information is needed to determine the minimum aspect ratio for which a flexible diaphragm model should be used in analysis and design of RC structures. Rigid diaphragms consist of reinforced concrete diaphragms, precast concrete diaphragms, and composite steel deck.

Shell Element: The shell object is an area element that can be used for plane stress, plane strain and axi-symmetric solid behavior. There are a few requirements to be met while using the shell elements in most of the conventional packages. Few of them are being listed below:

a) The shell could be used as a four or a three nodded element, but a four nodded element is preferred over the triangle of the reason that the triangle can undergo only translations and the stress recovery of the element is poor.
b) The angle at the corner should not be greater than 180^0, the best results could be obtained when the angle is close to 90^0, and the angle should lie between 45^0 to 135^0.
c) The aspect ratio of the elements should be close to unity with a tolerance up to four and in no case should the ration be greater than ten. The aspect ratio in case in the ratio of the longer side to the shorter side and in the quadrilateral the ratio is the longer distance between the mid points of the opposite sides to that such shorter side.

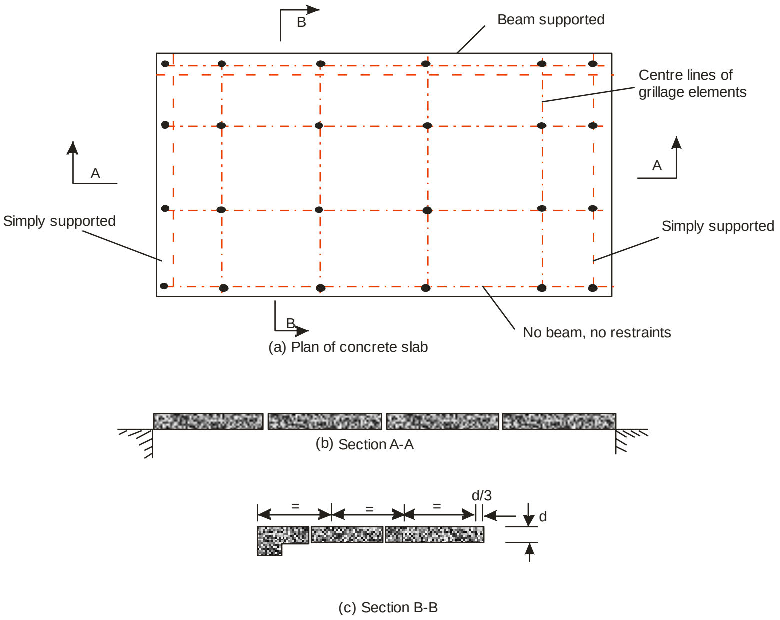
d) The joints in the shell could be non-coplanar but the accuracy of the results depend on the angle between the normal at the joints. Best results could be obtained when the larger of these angles is less than 30^0 and in no case the angle should be greater than 45^0.

There are thickness formulations in using the shell elements. There are two thickness formulations, the thick plates and the thin plates. The thick plates give consideration to the transverse shear deformation. Shearing deformation and the thin plates neglect the transverse shear deformation. Shearing deformation is important when there are higher stress concentrations due to bending, geometrical discontinuities. The thick plates are preferred over the thin plates for the reason that thick plates are accurate though they are stiffer and the accuracy of the thick plates depend on the mesh distortion and larger aspect ratio of the plates.



Shell Element

Grillage Element: The concrete slabs/diaphragms can also be idealized by grillage elements in both the directions. The grillage element is a beam element in which combined bending and torsion effects are included. Four to five grillage elements in each orthogonal direction can reasonably represent the flexibility of floor slab.

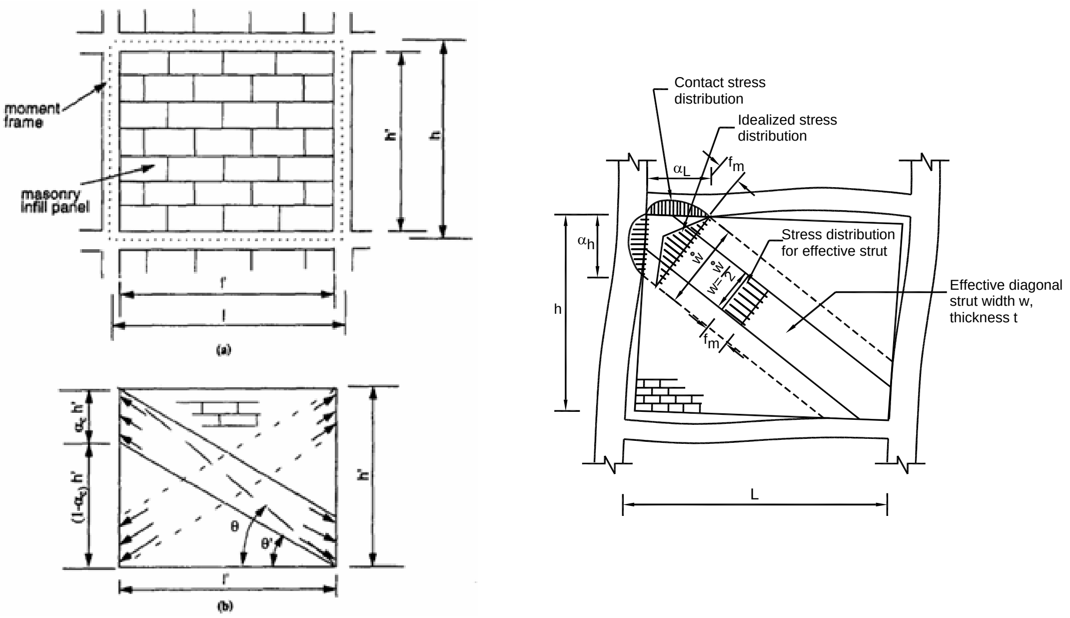


Grillage analogy for the rigid slab[2]

Modeling of Infill Walls

A large number of buildings in India are constructed with masonry infills for functional and architectural reasons. Masonry infills are normally considered as non-structural elements and their stiffness contributions are generally ignored in practice. However, infill walls tend to interact with the frame when the structure is subjected to lateral loads, and also exhibit energy-dissipation characteristics under seismic loading. Masonry walls contribute to the stiffness of the infill under the action of lateral load. The term 'infilled frame' is used to denote a composite structure formed by the combination of a moment resisting plane frame and infill walls. The infill may be integral or non-integral depending on the connectivity of the infill to the frame. In the case of buildings under consideration, integral connection is assumed. The composite behavior of an infilled frame imparts lateral stiffness and strength to the building.

In the case of an infill wall located in a lateral load resisting frame the stiffness and strength contribution of the infill are considered by modeling the infill as an equivalent diagonal strut model. In this model it is assumed that the contribution of the masonry infill panel to the response of the infilled frame can be modeled by "replacing the panel" by a system of two diagonal masonry compression struts. This type of model does not neglect the bending moment in beams and columns. Rigid joints connect the beams and columns, but pin joints at the beam-to column junctions connect the equivalent struts.



Modeling of infill wall as a equivalent diagonal strut model

This type of lateral deformation demands elongation in one diagonal length and compression in another diagonal length. If the frames are filled with infill walls which happen generally in the buildings then the infill walls try to act against these actions. Due to resistance offered by the infill walls in the diagonal lengths the brick infill within the panel can be modeled as strut elements in the two diagonal lengths.

The equivalent area of strut element is calculated as:

$$Area\ of\ \ strut\ \ element\ = W \times t$$

$$W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2}$$

Where,

$$\alpha_h = \frac{\pi}{2}\left[\frac{E_f I_c h}{2E_m t \sin 2\theta}\right]^{1/4}$$

$$\alpha_l = \pi\left[\frac{E_f I_b h}{E_m t \sin 2\theta}\right]^{1/4}$$

$$\theta = \tan^{-1}\frac{h}{l}$$

where W is width of strut, E_f elastic modulus of frame material, E_m elastic modulus of masonry wall, t thickness of infill wall, h height of infill wall, l length of infill wall, I_c moment of inertia of columns and I_b moment of inertia of beams.

Assessment of Properties of Brick Masonry:

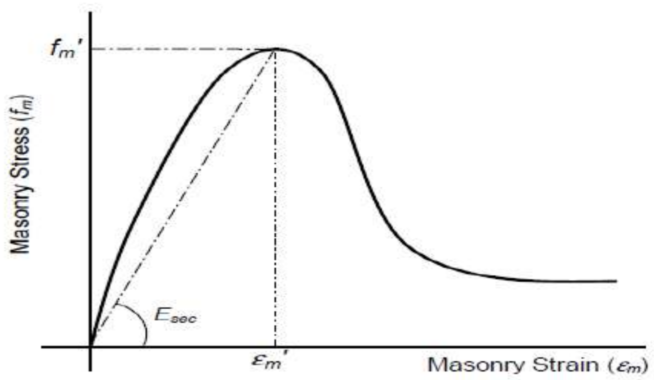
For the purpose of present studies, the properties of bricks available as per SP 20 (1991) are:
Ultimate compressive strength of common brick = 15Mpa
Compressive strength of Mortar Type M1 = 5Mpa
Masonry prism strength according to Pauley and Priestley (1992) is expressed as:

$$f_p = \frac{f_{cb}(f_{tb} + \alpha f_j)}{U_u(f_{tb} + \alpha f_{cb})}$$

Where, $\alpha = \frac{j}{4.1h_b}$ and U_u =Stress non-uniformity coefficient equal to 1.5

f_{tb}	=	tension strength of the brick	=	0.1times f_{cb}
f_{cb}	=	compression strength of the brick	=	15Mpa
f_j	=	compression strength of mortar	=	5Mpa
j	=	thickness of mortar joint	=	12mm
h_b	=	height of brick	=	75mm

The compressive strength of masonry (f_m) is considered as 75% of masonry prism strength. As per UBC (1997) formula, Elastic Modulus of masonry (E_m) =750 f_m



Stress-Strain for Masonry

Sample Calculations: Determination of width of equivalent diagonal strut (w):

Properties of Infill wall:

Thickness of infill wall with (t) =200mm

Properties of frame:

Size of beam =250x450mm

Size of column=300x300mm

Area of beam (A_b) =112500mm^2

Area of column (A_c) =90000mm^2

Elastic modulus of Frame (E_f) =$5000\sqrt{f_{ck}}$=$5000\sqrt{30}$=27386.128 MPa

Assessment of properties of Brick masonry:

f_{cb} =15 MPa

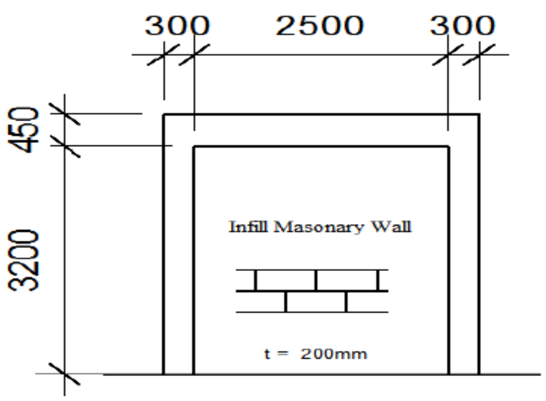
f_{tb} =1.5 Mpa

U_u =1.5

f_j =5 Mpa

j=12mm

h_b =75mm



RC Frame & Masonary Wall

$$\alpha = \frac{j}{4.1h_b} = \frac{12}{4.1 \times 75} = 0.0391$$

$$f_p = \frac{f_{cb}(f_{tb} + \alpha f_j)}{U_u(f_{tb} + \alpha f_{cb})} = \frac{15(1.5 + 0.0391 \times 5)}{1.5(1.5 + 0.0391 \times 15)} = 8.13MPa$$

$$F_m = 75\% of_p = 0.75 \times 8.13 = 6.10MPa$$

As per UBC (1997)

$$E_m = 750 \times F_m = 750 \times 6.10\ MPa = 4900MPa$$

$$\theta = \tan^{-1}\left(\frac{h}{l}\right) = \tan^{-1}\left(\frac{3.2}{2.5}\right) = 52^{\circ}$$

$$I_c = \text{Moment of inertia of column} = \frac{bd^3}{12} = \frac{300 \times 300^3}{12} = 6.75 \times 10^8 MPa$$

$$I_b = \text{Moment of inertia of beam} = \frac{bd^3}{12} = \frac{250 \times 450^3}{12} = 1.898 \times 10^9 MPa$$

$$\alpha_h = \frac{\pi}{2}\sqrt[4]{\frac{4E_fI_ch}{E_m t \sin 2\theta}} = \frac{\pi}{2}\sqrt[4]{\frac{4 \times 27386.128 \times 6.75 \times 10^8 \times 3200}{4900 \times 200 \times \sin(2 \times 52)}} = 1109.43mm$$

$$\alpha_h = \pi\sqrt[4]{\frac{4E_fI_bh}{E_m t \sin 2\theta}} = \frac{\pi}{2}\sqrt[4]{\frac{4 \times 27386.128 \times 1.898 \times 10^8 \times 3200}{4900 \times 200 \times \sin(2 \times 52)}} = 2694.88mm$$

The width of the equivalent diagonal struct (W) is given by,

$$W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2} = \frac{1}{2}\sqrt{1109.43^2 + 2694.88^2} = 1457.156mm$$

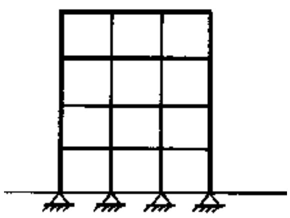
$$\text{Area of the struct} = 1475.156 \times 200 = 291431.1856mm^2$$

Modeling the Ground and Foundations

The main structural system of the building and the foundation can be modeled in four ways;

Support Fixity model

Deformations of the ground are ignored and the nodes for the structure at the contact with the ground are given fixed. Ground beams and continuous footing can be modeled as beam elements.



(a) Support fixity model

Winkler Spring Model

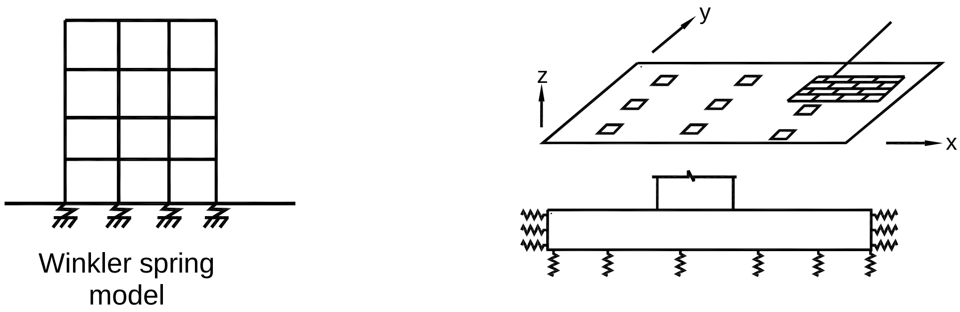
It is a very approximate and simpler method to quantifying the effect of soil deformability. Winkler's model is based on substituting a set of discrete elastic springs for the continuous, deformable soil. It is also easy to implement using a structural analysis package and may give better results than by ignoring the effect of soil stiffness in an analysis of the structure as in case of first model. It may also provide information about trends in behavior. According to this model, a single footing can be modeled as a rigid block supported on a Winkler spring with the properties

$$K_v = k_{wk} A$$
$$K_\theta = k_{wk} I_f$$

Where K_v is the vertical spring stiffness, k_{wk} is the Winkler stiffness, A is the area of the footing, K_θ is the rotational spring stiffness and I_f is the I value of the base of the footing (about the same axis as K_θ is defined). For a 3D model, rotational springs about two axes can be defined. The value of Winkler spring stiffness at a node under a raft is calculated as;

$$K_{winkler} = k_{wk} A_t$$

Where k_{wk} is the Winkler stiffness and A_t is the area of the soil surface which is tributary to the node (for example, with a mesh of nodes which has spacing a by b, the tributary area for an internal node is ab, for a side node is $ab/4$).



Winkler's soil model

Table below gives typical values for k_{wk}. It is possible to relate k_{wk} to the elastic properties of the soil using (Hemsley 1998)

$$k_{wk} = 2E_s/(\pi a(1-v^2{}_s))$$

Where E_s is Young's modulus for the soil, v_s is Poisson's ratio for the soil and a is the radius of the equivalent plate. The elastic modulus of granular soils based on effective stresses is a function of grain size, gradation, and mineral composition of the soil grains, grain shape, soil type, relative density, soil particle arrangement, and stress level and prestress. Typical values of the elastic modulus and Poisson's ratio for different types of soils are given in Table below.

Typical values of Winkler stiffness for soils[2]

Type of Soil	$K_{wk}\ : kN/m^3$
Loose sand	4800-16000
Medium dense sand	9600-80000
Dense sand	64000-128000
Clayey medium dense sand	32000-80000
Silty medium dense sand	24000-48000
Clayey soil:	
$q_u \leq 200 N/mm^2$	12000-24000
$200 < q_u \leq 400 N/mm^2$	24000-48000
$q_u > 800 N/mm^2$	>48000

Note: q_n – bearing capacity, From Bowies (2001).

Typical values for modulus of elasticity for soils[2], E_s

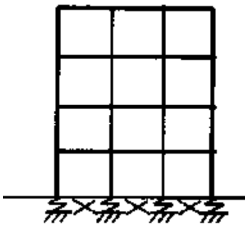
Type of soil	$E_g\ : N/mm^2$
Clay	
Very soft	2-15
Soft	5-25
Medium	15-50
Hard	50-100
Sandy	25-250
Glacial till	
Loose	10-153
Dense	144-720
Very dense	478-1440
Loess	14-57
Sand	
Silty	7-21
Loose	10-24
Dense	48-81
Sand and gravel	
Loose	48-148
Dense	96-192
Shale	144-14400

Typical Poisson's ratio for soils

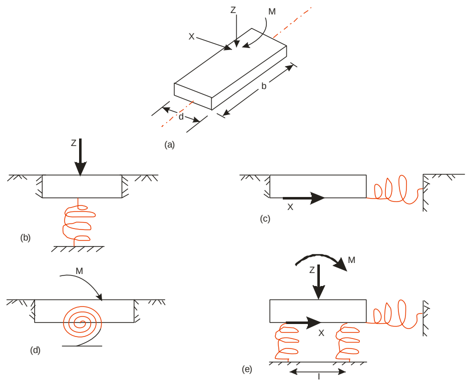
Type of soil	**Poisson's ratio (v)**
Clay, saturated	0.4-0.5
Clay, unsaturated	0.1-0.3
Sandy clay	0.2-0.3
Silt	0.3-0.35
Sand (dense)	0.2-0.4
Coarse (void ratio= 0.4-0.7)	0.15
Fine grained (void ratio = 0.4-0.7)	0.25
Rock (depends on type of rock)	0.1-0.4
Loess	0.1-0.3

Half Space Models

The concept of a half space model is that a grid of nodes is defined at the ground surface and a stiffness matrix for the soil is defined at that level. This is added to the stiffness matrix for the structure to produce a model that incorporates both the soil and the structures. This is also known as a *boundary element* model. Ground with foundation can be represented by the set of elastic springs by three 'global' springs in the centre of the foundation with the constants.



(c) Half space model



Half space soil model

For vertical displacements

$K^{v}(kN\,m^{-1})$;

For horizontal displacements

$K^{H}(kN\,m^{-1})$;

For rotations

$K^{\varphi}(kN\,m\ rad^{-1})$;

The approximate estimation of stiffnesses may be obtained by these simplified equations as represented below;

Shear modulus : $G = \dfrac{E}{2(1+v)}$

Vertical stiffness : $K_z = \dfrac{2.5GA^{0.5}}{(1-v)}$

Horizontal stiffness : $K_x = 2G(1+v)A^{0.5}$

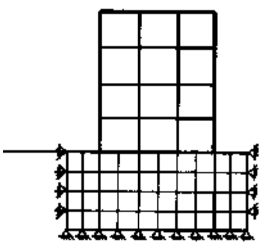
Rocking stiffness : $K_m = \dfrac{2.5GZ}{(1-v)}$

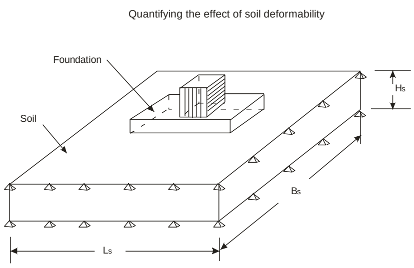
Where, G is the shear modulus of soil, E is the young's modulus of soil, V= Poisson's ratio of soil, A is a foundation area $(b \times d)$ and Z is the foundation section modulus $(bd^2/6)$. Vertical and rotational stiffnesses can be more conveniently represented by two parallel springs for which the stiffness is taken as: $K = 0.5K_z$. The parallel springs are spaced at a distance l given by $l = 2(K_m/K_z)^{0.5} = 0.82b^{0.25}d^{0.75}$. These equations are approximate; however, they provide quick means of determining the order of magnitude of foundation stiffness.

The order of magnitude of the piles rigidities is also rather close to the values found for spread footings. Therefore, these springs' stiffness are valid for pile foundations, too. Otherwise several computer programs are available which can be used to calculate the stiffness of pile groups, either using finite elements, or the theory for an elastic half space.

Finite Element Method

From a theoretical point of view, the most accurate method for determining the effect of soil deformability is the so-called **interaction analysis**, in which the structure, the foundations and the surrounding soil are dealt with as a single system. The extent of the soil (the sizes L_s, B_s, H_s) is chosen so that the stresses on its periphery are negligible; consequently, the supports along this periphery have no practical effect on the analysis.


(d) Element model of ground



Finite soil model

The following rule may be applied to limit the extent of the finite soil model;

H_s/B; 10.0 -12.0
B_s/B: 8.0-12.0
L_s/L: 8.0-12.0

Where, B and L are the width and length of foundation.

Obviously, such an analysis has to be performed by three-dimensional ('solid') finite elements only. We must choose the modulus of elasticity E_s (Young modulus) for normal stresses and G_s for shear stresses (or the Poisson's ratio) of the soil.

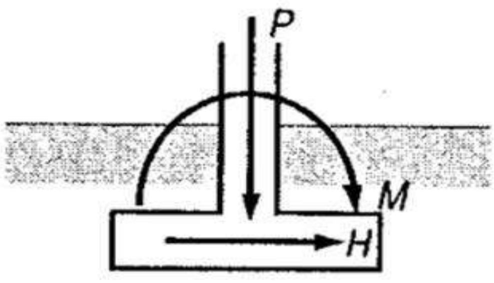
However, this method has several drawbacks also (Scarlet, 1996):

- The computation involves a very high number of unknowns and, as such, needs special computer capability.
- The validity of the usual assumption of elastic/uncracked soil elements is rather questionable; we may partly correct the effect of this assumption by a non-linear analysis that also includes gaps, but additional computational complications are involved.
- The soil is non-homogenous, and we cannot overcome this intricacy by computational means.
- The recommended moduli of elasticity of the soil are uncertain, and this strongly affects the results.
- No experimental confirmation of interaction analysis is available especially of the assumptions made in a non-linear analysis; computations have shown that the corresponding rigidity of the soil may decrease by 50% with respect to the rigidity resulting from an elastic analysis.

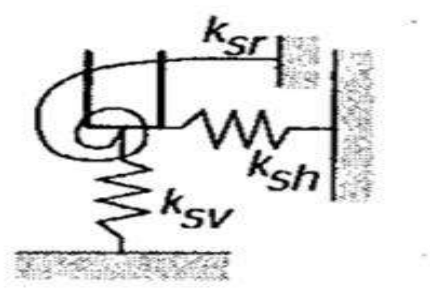
Consequently, when considering an elastic soil, it is advisable to consider two values of the modulus of elasticity of the soil-a high value and a low value- and to perform two corresponding computations; the design will be based on the highest stresses. The method is potentially useful elastic model for defining a half space model. The stress in the soil is estimated using homogeneous conditions, but the strains caused by these stresses are integrated taking into account variation of properties within the depth of the soil.

Soil Flexibility as considered in ATC40:

For shallow bearing footings that are rigid with respect to the supporting soil, an uncoupled spring model will represent the foundation stiffness. The equivalent spring constants shall be calculated as specified in the table given by Gazetas (1991)[7].



. FOUNDATION LOAD



UNCOUPLED SPRING

Spring Model

Spread and Pile Footings : Spread or pile footings are typically considered to be rigid bodies that allow support conditions to be modeled at a single point with boundary springs at the bottom of the column model at the end of the effective length extension link into the footing . For a two-dimensional column, only a vertical, a translational, and a rotational boundary spring need to be defined, whereas in a three-dimensional model, six springs, one for each possible DOF at the column base, are required. In spread footing the soil resistance is provided in the vertical direction by direct bearing pressure, in the horizontal direction by passive soil pressure in front of the footing base and side, and in the rotational direction by the soil overburden on top of the footing and gravity-load effects.

Surface Stiffness for a Rigid Plate on a Semi-infinite Homogeneous Elastic Half Space (Gazetas (1991)[7]

Stiffness Parameter	Rigid Plate Stiffness at Surface, k
Vertical Translation, k_Z	$\frac{GL}{1-v}\left[0.73+\left(\frac{B}{L}\right)^{0.75}\right]$
Horizontal Translation, k_y (toward long side)	$\frac{GL}{2-v}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]$
Horizontal Translation, k_x (toward long side)	$\frac{GL}{2-v}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right]-\frac{GL}{0.75-v}\left[0.1\left(1-\frac{B}{L}\right)\right]$
Rotation, $k_{\theta x}$ (about x-axis)	$\frac{GL}{1-v}Ix^{0.75}\left(\frac{L}{B}\right)^{0.25}\left(2.4+0.5\frac{B}{L}\right)$
Rotation, $k_{\theta y}$ (about y-axis)	$\frac{GL}{1-v}Iy^{0.75}\left[3\left(\frac{L}{B}\right)^{0.15}\right]$

Stiffness Embedment factors for a Rigid Plate on a Semi-infinite Homogeneous Elastic Half Space (Gazetas (1991)[7]

Stiffness Parameter	**Embedment Factors, e_i**
Vertical Translation, k_Z	$\left[1+0.095\frac{D}{B}\left(1+1.3\frac{B}{L}\right)\right]\left[1+0.2\left(\frac{(2L+2B)}{LB}d\right)^{0.67}\right]$
Horizontal Translation, k_y (toward long side)	$\left[1+0.15\left(\frac{2D}{B}\right)^{0.5}\right]\left\{1+0.52\left[\frac{\left(D-\frac{d}{2}\right)16(L+B)d}{BL^2}\right]^{0.4}\right\}$
Horizontal Translation, k_x (toward long side)	$\left[1+0.15\left(\frac{2D}{L}\right)^{0.5}\right]\left\{1+0.52\left[\frac{[(D-\frac{d}{2})16(L+B)d}{LB^2}\right]^{0.4}\right\}$
Rotation, $k_{\theta x}$ (about x-axis)	$1+2.52\frac{d}{B}\left(1+\frac{2d}{B}\left(\frac{d}{D}\right)^{-0.20}\left(\frac{B}{L}\right)^{0.50}\right)$
Rotation, $k_{\theta y}$ (about y-axis)	$1+0.92\left(\frac{2d}{L}\right)^{0.60}\left(1.5+\left(\frac{2d}{L}\right)^{1.9}\left(\frac{d}{D}\right)^{-0.60}\right)$

Flexibility reduces the overall stiffness of the structure and increase the natural period of the system. Considerable change in spectral acceleration with natural period is observed from the response spectrum curve. Thus the change in natural period may alter the seismic response of any structure considerably. In addition to this, soil medium imparts damping due to its inherent characteristics. The issue of increasing the natural period and involvement of high damping in soil due to soil structure interaction in building structure interaction in building structures are also discussed in some of the studies. Moreover, the relationship between the period of vibration of structure and that of supporting soil is important regarding the seismic response of the structure. The displacement increases with increase in soil-flexibility. These show that the soil-structure interaction should be accounted for in the analysis of dynamic behavior of structure interaction in building structures, in practice. Hence, soil-structure interaction under dynamic loads is an important aspect to predict the overall structural response.

UNCERTAINTY IN MATHEMATICAL MODELING OF THE BUILDING

There are a number of uncertainties associated with the stiffness, the mass, and the damping. Some important sources of uncertainty in the modeling of building are (Axley and Bertero, 1979)

- Uncertainty in stiffness
- The characteristics of the concrete, steel and especially the masonry used in the building always remain uncertain
- The uncertain modeling of beam section properties
- The uncertain consequences of the use of centerline geometry ignoring beam-column joints and slab eccentrics.
- The uncertain constraint condition between the infill and surrounding frame
- The uncertain soil stiffness and soil structure interaction
- The uncertain evaluation of tributary regions used to determine the lumped mass
- The (usual) uncertain use of viscous damping and the assignment of the magnitude of damping
- The uncertain importance of additional energy dissipation through soil structure interaction
- In addition to modeling uncertainties there are uncertainties in the loading, uncertainties that result from practical consideration of analysis (e.g. number of mode chosen to capture the behavior of the building to ground excitation).

Further Reading

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5. Scarlat, A.S., (1996). "Approximate Methods in Structural Seismic Design" E & FN Spon.
6. Axley, J.W. and Bertero, V.V (1979). "Infill panels: Their Influence on Seismic Response of Building", UCB/EERC – 79/28, University of California, California.
7. ATC 40, "Seismic Evaluation and Retrofit of Existing Concrete Buildings", Volume 1, Applied Technological Council 1996, California.

2.2 How do the various structural and non-structural components of a multi-storied building influence its modal characteristics?

Problem Statement: Modal parameters of structures are the most valuable quantity on which the dynamic response of the system depends. For example, the total base shear and its vertical distribution depend upon the fundamental time period and mode shape respectively. They are very subjective in nature since these parameters are estimated on the basis of empirical formulas as given in codes. The issue of estimation of modal parameters on the basis of mathematical modeling has always remained challenging since it involves innumerable structural and non-structural elements. It has always been an open question as to which elements have to be considered in mathematical modeling; what can be ignored and how much is its effect on basic characteristics of structures in comparison to observed behavior. In this example, modal parameters have been estimated by finite element modeling of building by considering the effects of a number of structural and non-structural elements.

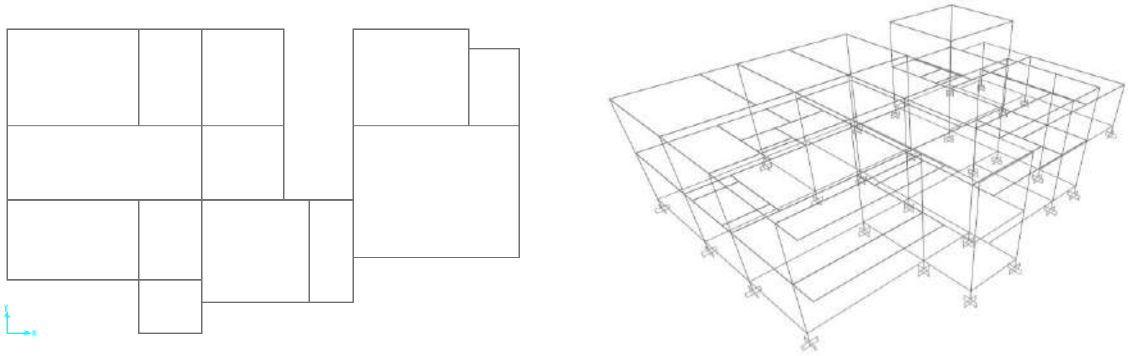
INTRODUCTION

The effects of structural and non-structural parameters on the seismic behavior of multi-storied reinforced concrete buildings (G +1 to G+12) have been carried out by creating a number of mathematical models to idealize the stiffness, mass distribution and energy dissipation so that its response to earthquake could be predicted with sufficient accuracy. The principal issues in mathematical modeling of a building system are modeling of beams, columns, floor diaphragms, infill walls, staircases, foundation etc along with effect of soil flexibility and the assumption involved. In order to study the effect of structural and non-structural members, six models from M1 to M7 have been generated, using the specific geometry, material properties and section properties using a commercial software SAP@2000.

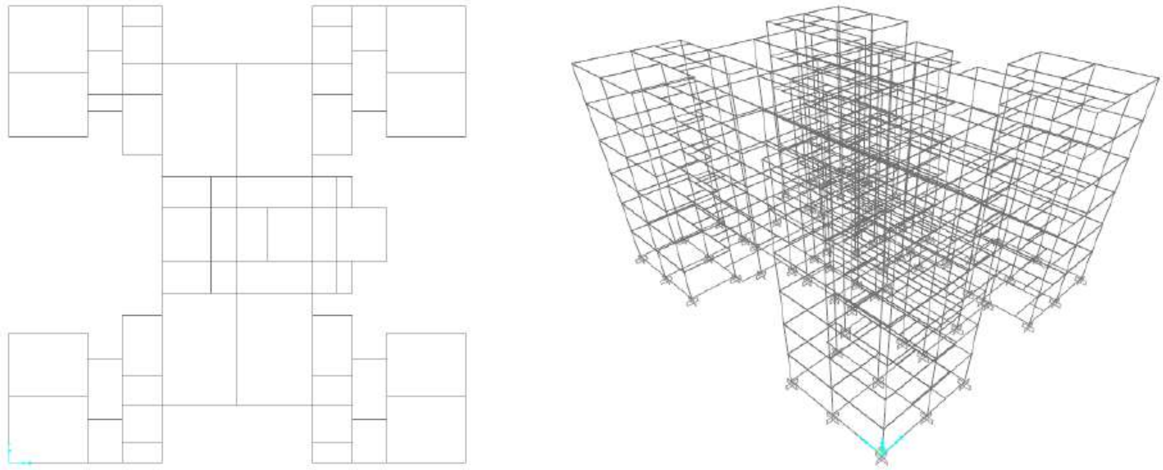
DESCRIPTION OF THE BUILDING

The selected five buildings for this study are G+1, G+5, G+6, G+9, G+12 having different plans and elevations. The common details of these buildings are;

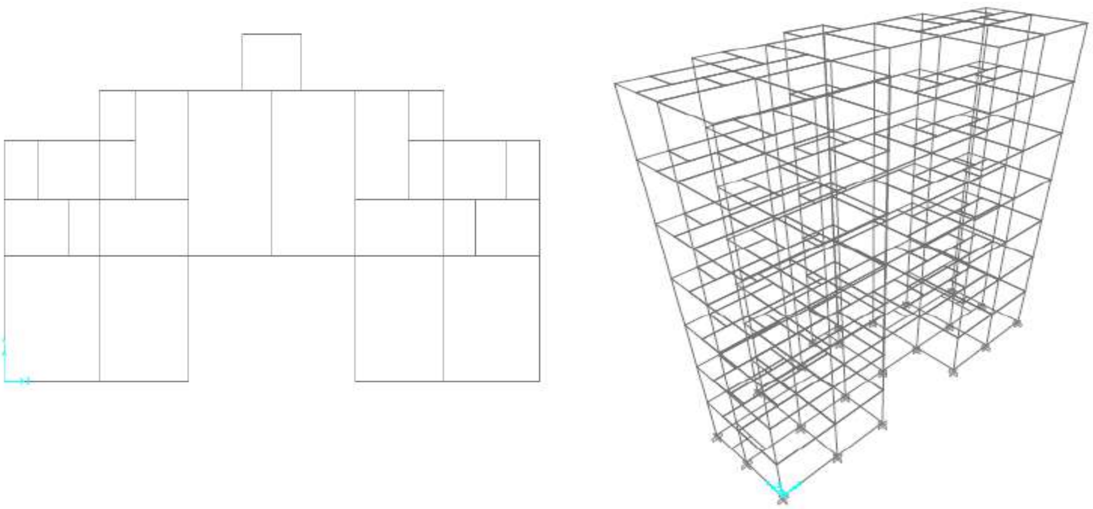
- The dead loads and live loads as per IS 875-1987, Part I and II respectively
- Seismic load as per IS 1893 (Part I):2002
- Depth of foundation below ground = 0.3m;
- Average thickness of slab = 125mm
- Thickness of pile cap = 0.75 to 1.5m
- Pile diameter varied between 0.5 to 0.7m
- Pile length = 50m for G+1 building and 60m for all the remaining buildings
- Storey height = 3.2m
- Wall thickness = 200 mm thick brick wall at exterior and 100mm thick at interior wall
- Shear wall (Lift core) for G+5 and G+12 buildings are 2.0x0.2m and 2.4x 0.2m
- The grade of concrete is M30 for all components
- The grade of steel is Fe 500 for longitudinal reinforcement and Fe 415 for confining reinforcement



Plan and isometric view of G+1 building

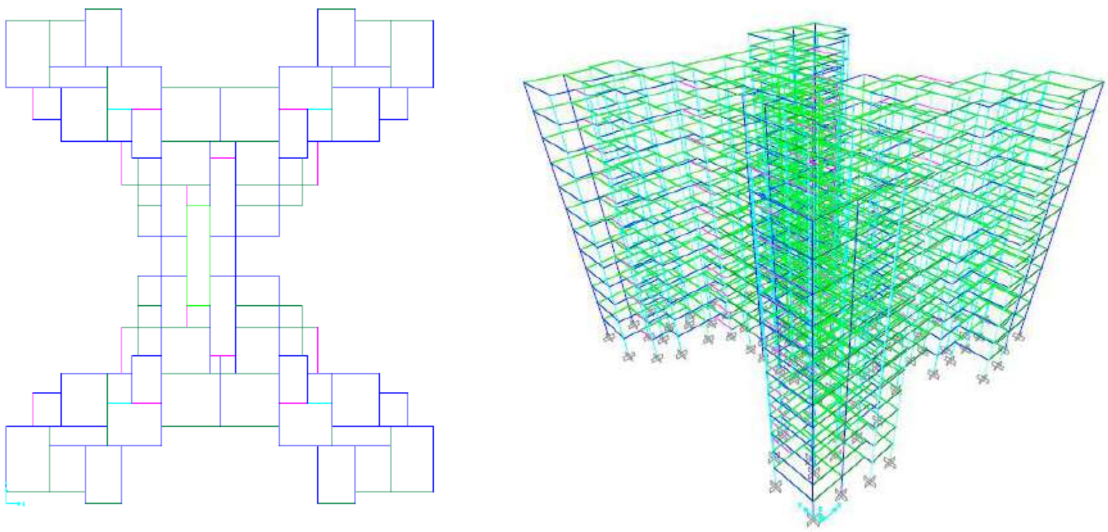


Plan and isometric view of G+5 building



Plan and isometric view of G+6 building

(Plan and isometric view of G+9 building is same as G+6 building except numbers of storey)



Plan and isometric view of G+12 building

MODELLING AND ANALYSIS

Buildings have been modeled in 3D space frame in SAP 2000 software. Seven space frame models for each building have been created with the following details;

M1: A bare frame model of the buildings is created with full E_cI_g, E_cA_w. Slab has been modeled as Rigid Floor Diaphragm i.e. all the nodes of the slab in the same plane have same amount of displacement. To model the slab, diaphragm constraint has been assigned at each floor level. The support conditions of buildings are considered as fully fixed.

M2: Buildings with cracked sections as per ATC 40, in which beams are modeled having $0.5E_cI_g$ flexural rigidity, $0.4E_cA_w$ shear rigidity and the columns are modeled with an average value of $0.5\ E_cI_g$ flexural rigidity, $0.4E_cA_w$ shear rigidity.

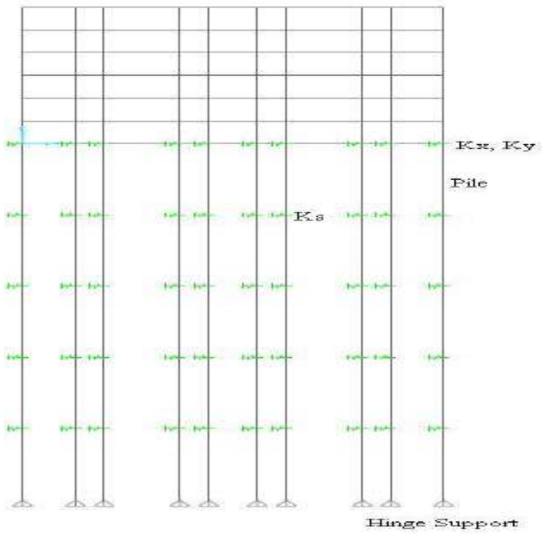
M3: Modeling is similar to M1 expect the slab has been modeled by shell element by removing the diaphragm constraint at each floor level.

M4: Modeling is similar to M1 but in addition infill walls have been modeled as equivalent compression diagonal struts. Rigid joints connect the beams and a column, but pin joints at the beam-to column junctions to connect the equivalent struts.

M5: The infill walls have been modeled as equivalent diagonal strut at the upper floors only i.e. soft storey building.

M6: Modeling is similar to M1 with uncoupled spring at the base to represent the foundation stiffness of the spread footing or pile footings which is rigid with respect to the supporting soil. In this model, the effects of pile group and soil damping have been neglected. The equivalent spring constants are calculated as per the Gazetas (1991) formulas given in ATC 40.

M7: The buildings have been modeled similar to M6 i.e. uncoupled spring at the base. The pile cap has been modeled with spring stiffness in K_x and K_y direction using Gazetas (1991) as given in ATC 40. The pile has been modeled as beam element and pile soil interaction is modeled as lateral spring stiffness K_s . At the bottom of the pile hinge support is assigned to represent the end bearing or hard strata. The details of all the modeling cases are given in Table below.



Modeling of Pile cap with Piles

Different modeling case of G+1 to G+12 buildings

Modelling Cases	**Modelling Parameters**			
	Support Condition	**Slab**	**Section**	**Frame**
Bare Frame	Fixed Support	Rigid Diaphragm	Un-cracked Section	Bare Frame
Cracked Section	Fixed Support	Rigid Diaphragm	Cracked Section	Bare Frame
Shell Element	Fixed Support	Shell Element (without Rigid Diaphragm)	Un-cracked Section	Bare Frame
Infill Frame	Fixed Support	Rigid Diaphragm	Un-cracked Section	Infill Frame
Soft Storey	Fixed Support	Rigid Diaphragm	Un-cracked Section	Ground Floor without Infill
Base(Pile cap)	Uncoupled Spring Support for pile cap	Rigid Diaphragm	Un-cracked Section	Bare Frame
Base(Pile + pile cap)	Uncoupled Spring Support for pile cap and Pile modelled as beam element with lateral spring & hinged at bottom	Rigid Diaphragm	Un-cracked Section	Bare Frame

Note: In G+5 and G+12 building's lift core has also been modeled by shell element.

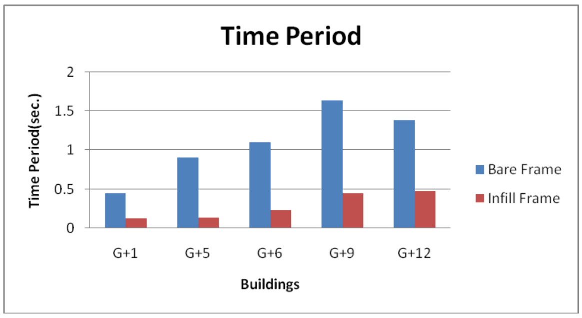
Type of Analysis

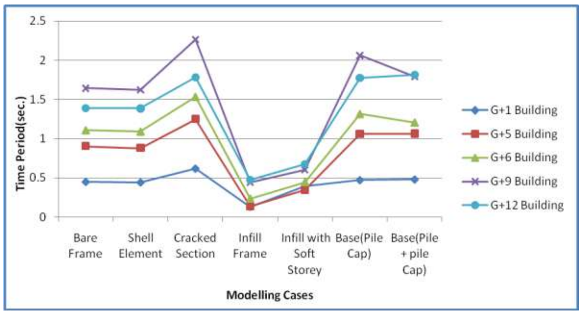
Modal analysis using response spectrum method and time history analysis has been carried out to determine the model parameters, base shear storey/weight, storey drift and amplification factors in different buildings with various modeling parameters. The studies are;

Modal analysis or Response Spectrum Analysis

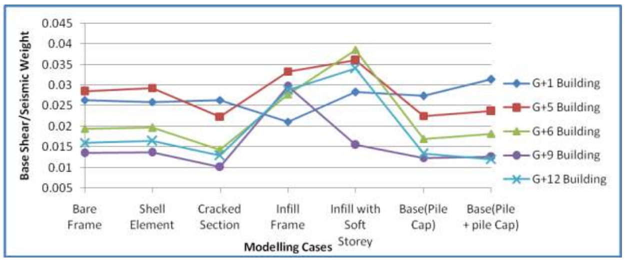
For response spectrum analysis, the following values have been considered;
Zone factor (Z) = 0.16 (Zone III)
Importance Factor (I) = 1
Response Reduction Factor(R) = 5 (SMRF)
Type of soil = soft soil (Type III)
Elastic response spectrums = as defined in IS 1893(Part 1) -2002
Numbers of modes considered = 12 to 16 (required enough for the cumulative mass participation to be more than 90%)
Modal combination method = SRSS
Damping = 5%

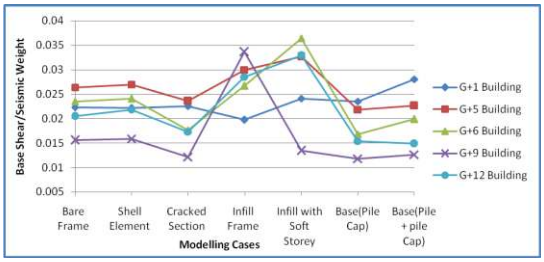
Based on the above modeling, analysis and calculations the seismic response (Time period, Base shear/Weight and Storey Displacement) for different buildings by changing the modeling parameters have been determined as below.



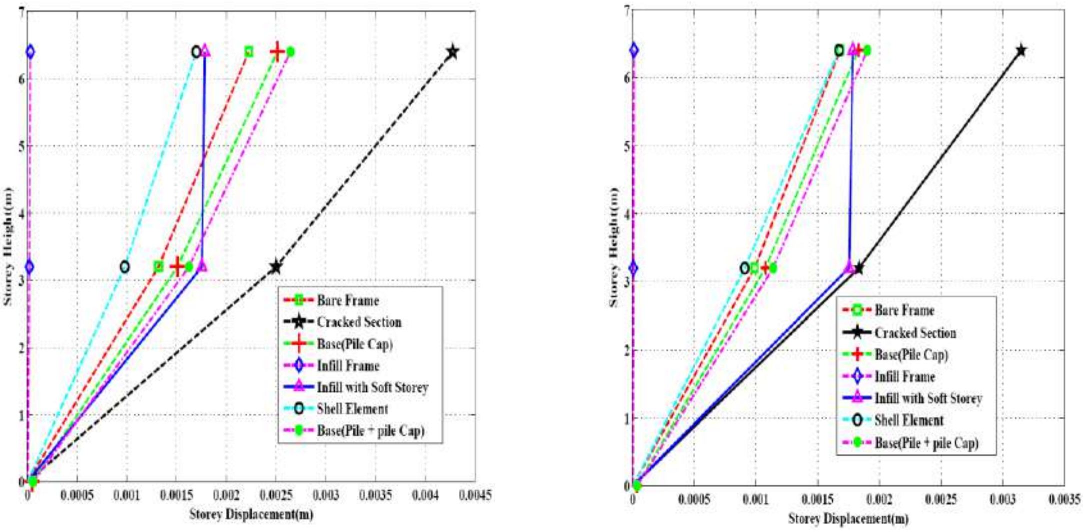


Variation of Time Period for different buildings with different modeling cases

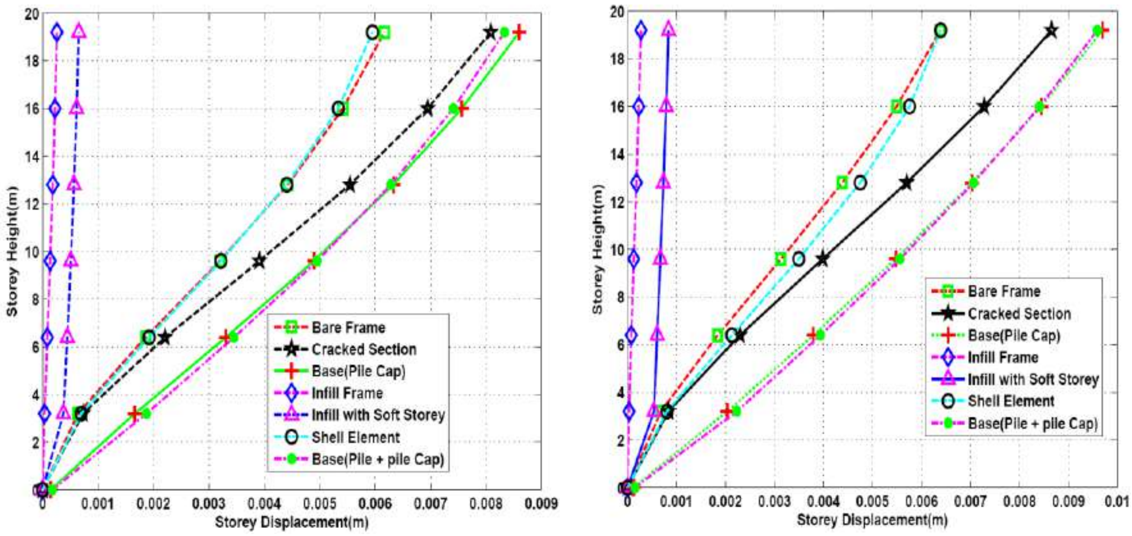




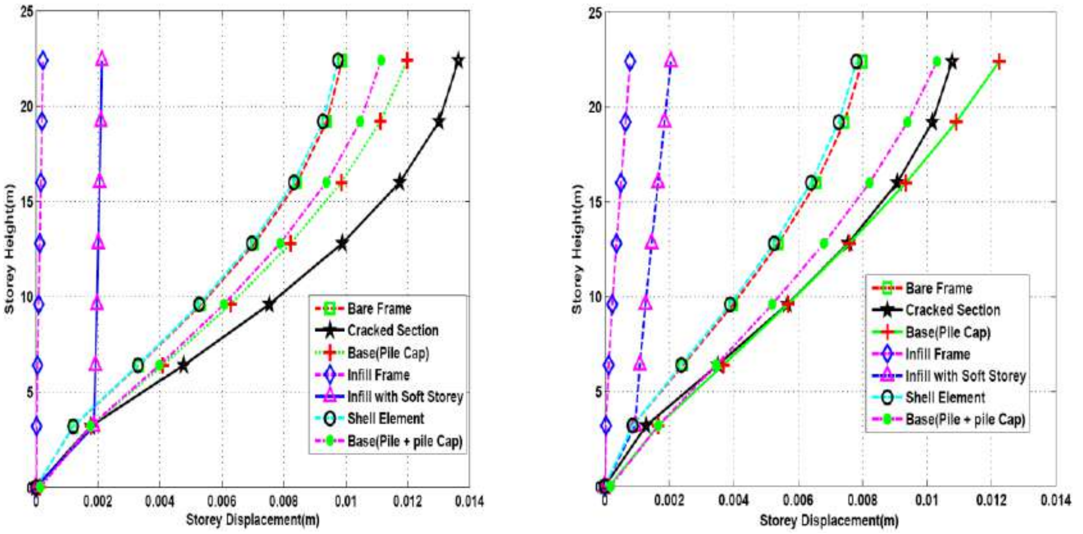
Variation of (Base Shear/Seismic Weight) ratio in EQ-X and Y- direction



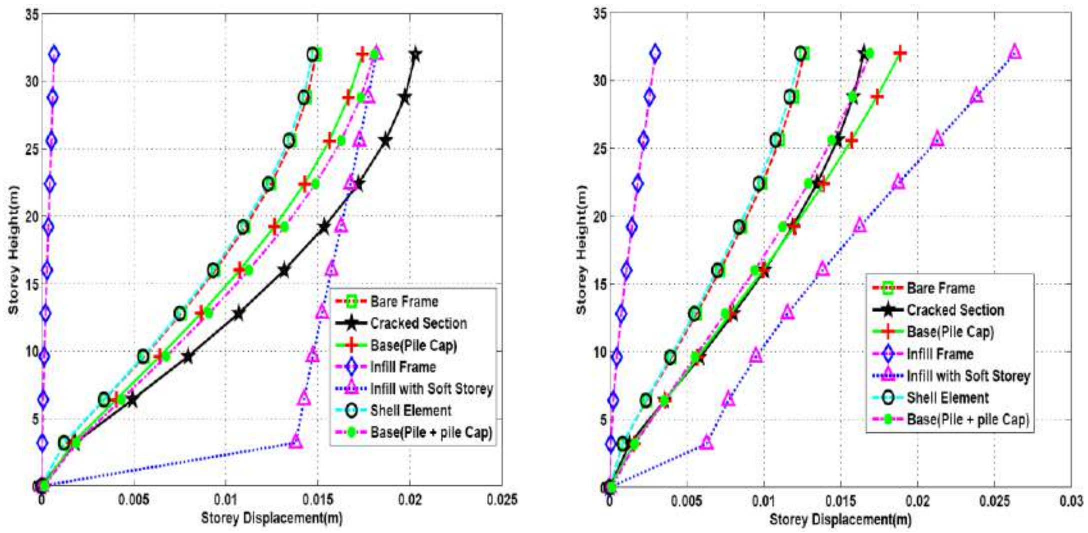
Storey Displacement for G+1 Building (a) EQ-X direction, (b) EQ-Y direction



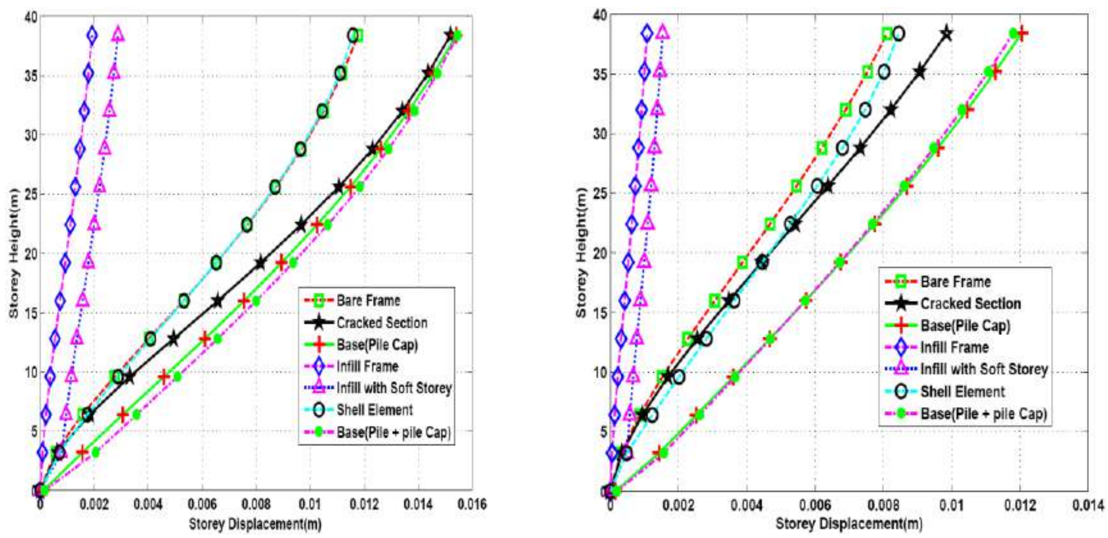
Storey Displacement for G+5 Building (a) EQ-X direction, (b) EQ-Y direction



Storey Displacement for G+6 Building (a) EQ-X direction, (b) EQ-Y direction



Storey Displacement for G+9 Building (a) EQ-X direction, (b) EQ-Y direction



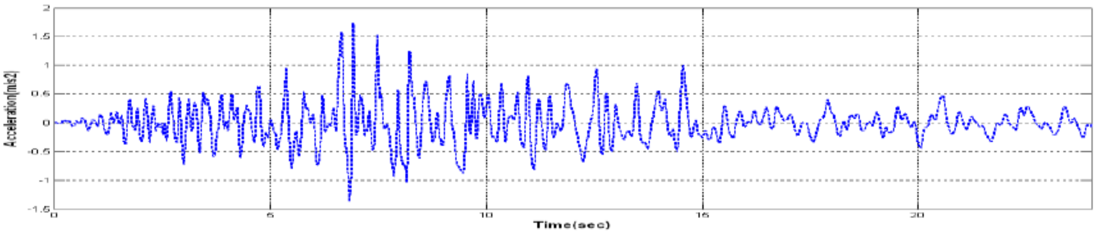
Storey Displacement for G+12 Building (a) EQ-X direction, (b) EQ-Y direction

Time History Analysis

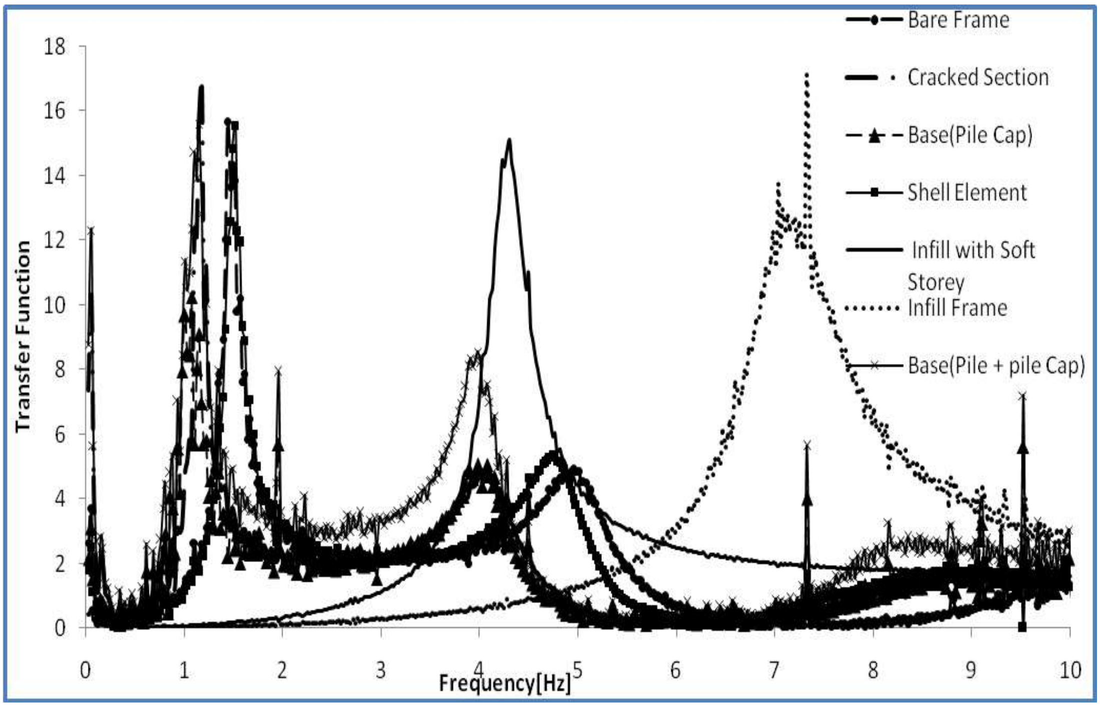
An acceleration time history corresponding to Zone III (PGA =0.16g) has been considered, Figure 12. This time history is applied to the G+5 building for obtain the response of building at roof level in each modeling case i.e. M1 to M7. The Fourier amplitude is calculated for ground motion and roof top acceleration time histories. Using Fourier amplitude of ground motion and roof level of G+5 building under different modeling case, amplification factor of peak ground acceleration has been determined and the transfer function vs. frequency was plotted in as below.

Amplification factors for G+5 building under different modeling cases

Case	Description of Modeling Cases	Amplification Factor
M1	Bare Frame	3.01
M2	Infill Frame	3.1
M3	Cracked Section	2.64
M4	Shell Element	3.42
M5	Base(Pile Cap)	2.57
M6	Infill with Soft Storey	3.3
M7	Base(Pile + Pile Cap)	3.46



Compatible Acceleration Time History for Ground Motion for Zone III with PGA =0.16g



Comparison of Transfer Functions for G+5 building Model Parameters

CONCLUSIONS

The following conclusions have been made from the study:

1. Time period is increased in cracked section model as compared to un-cracked section model of all the buildings. The base shear/weight ratio is decreased in cracked section model.
2. Time period is increased in bare frame model as compared to infill frame in all the buildings and the base shear/weight ratio is decreased in bare frame model in X and Y-directions.
3. Time period is decreased in fixed base as compared to spring model at the base of the buildings and the base shear/weight ratio is increased in fixed base in X and Y-directions.
4. There is no significant variation in the time period and base shear/weight ratio for all the buildings when the building slab is modeled with rigid diaphragm constraint and shell element separately.
5. Time period is increased in bare frame model when compared to infill with soft storey model of all the buildings except G+1 building. Base shear/weight ratio is decreased in bare frame for all the buildings in X and Y-directions.
6. In all the buildings, cracked section model and spring models are giving more storey displacements as compared to bare frame model. Storey displacements in G+1, G+6 and G+9 buildings with infill soft storey model are giving more displacements when compared to bare frame except G+5 and G+12 buildings with soft storey. The buildings with infill frame models are giving lesser storey displacements in both the directions.
7. Frequency values corresponding to the peak values from the transfer function vs. frequency plot for G+5 building using time history analysis gives the natural frequencies of the building which are approximately equal to the response spectrum analysis. Further it is possible to find the modal damping ratio of a structure using half-power band width method.

Further Studies

1. Wilson E. and Habibullah A., "Sap 2000 v 12.2.4 Integrated Finite Element Analysis and Design of Structures Basic Analysis Reference Manual, Computers and Structures, 1998, Berkeley, USA.
2. IS 875 (Part 1): 1987, "Indian Standard Code of Practice for Design Loads (other than earthquake) for Buildings and Structures" (Second Revision), Bureau of Indian Standards, New Delhi.
3. IS 875 (Part 2): 1987, "Indian Standard Code of Practice for Design Loads (other than earthquake) for Buildings and Structures" (Second Revision), Bureau of Indian Standards, New Delhi.
4. IS: 1893 - 2002, "Indian Standard Criteria for Earthquake Resistant Design of Structures", Bureau of Indian Standards, Manak Bhawan, 9 Bahadur Shah Zafar Marg, New Delhi, India.
5. ATC 40, "Seismic Evaluation and Retrofit of Existing Concrete Buildings", Volume 1, Applied Technological Council 1996, California.

Chapter 3

Earthquake resistant design of an RC frame building and shear walls

3.1 How to carry out seismic design of a multi-storied RC framed building?

Problem statement: A G+10 storey building with square plan 16 m x 16 m is considered. Floor to floor height considered is 3.1 m. The height of building measured from the ground level is 35.3 m. The depth of the foundation below the ground level is 1.2 m. The building is located in the seismic Zone IV and is resting on hard rock.

ASSUME DATA

a) All external walls are 230 mm thick and internal walls are 150 mm thick.
b) All floor diaphragms are considered to be rigid.
c) Centre to centre dimensions are followed for analysis and design and the effect of finite size joint width is not considered.
d) Seismic loads are considered to act in the horizontal direction (along either of the two principle directions) and not in the vertical direction.
e) Stiffness of infill walls is not considered in the seismic analysis of the building.
f) Deductions for opening is not done while calculating the seismic weight of building.
g) Wind load is not considered.
h) Building is considered to be fixed at the base level.

ASSUME DIMENSIONS OF BEAM AND COLUMN OF THE BUILDING

Building	Beam Size (mm)	Column Size (mm)
G+10	300 X 450	GL: 500 X 500; 1st to 3rd floor: 450 X 450; 4th to 7th floor: 400 X 400; 8th to 10th floor: 350 X 350
	40 beam per floor	25 columns per floor

PRELIMINARY DATA FOR ANALYSIS

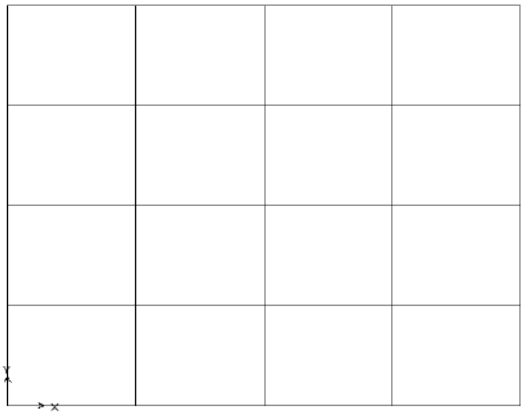
1	**Types of structure**	Multi-storey rigid space frame
2	**Seismic zone**	Zone IV (Z=0.24)
3	**Soil types**	Type I (Rock Type)
4	**Number of storeys**	11 stories (G+10)
5	**Infill wall**	External: 230 mm
		Internal: 115 mm
6	**Slab thickness**	120 mm

7	**Floor height**	3.1 m
8	**Earthquake load**	As per IS 1893-2002
9	**Wind load**	Not considered
10	**Live load on floors**	on typical floor: 3 KN/m^2
11	**Grade of concrete**	M 25
12	**Grade of steel**	Fe 415
13	**Unit weight of masonry**	20 KN/m^2
14	**Response Reduction Factor (R)**	5
15	**Type of Frame**	SMRF

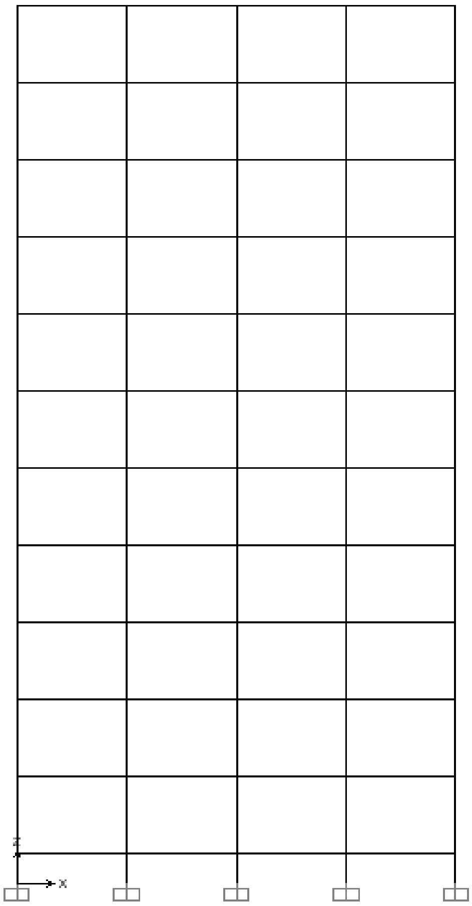
LOAD COMBINATION FOR ANALYSIS AND DESIGN

LC	Load Cases
1	1.5(DL+LL)
2	1.2(DL+LL+EQX)
3	1.2(DL+LL-EQX)
4	1.2(DL+LL+EQY)
5	1.2(DL+LL-EQY)
6	0.9DL+1.5EQX
7	0.9DL-1.5EQX
8	0.9DL+1.5EQY
9	0.9DL-1.5EQY
10	1.5(DL+EQX)
11	1.5(DL-EQX)
12	1.5(DL+EQY)
13	1.5(DL-EQY)

Where DL is Dead Load, IL is imposed load, EQX and EQY are earthquake loads in X and Y directions, respectively.



Plan of the (G+10) Storey Building



Elevation of the (G+10) Storey Building

Beam Element 535:

Results of analysis under various load combinations

	Force	LC 1	LC 2	LC3	LC 4	LC 5	LC 6	LC 7	LC 8	LC9	LC 10	LC 11	LC 12	LC 13
Left End	Shear	- 69.13	-98.74	-98.85	-49.7	-49.4	96.68	-96.8	- 36.41	- 36.41	-120.5	-121.1	-59.9	- 60.41
	Moment	-46.7	-133.7	-133.7	-33.0	-32.5	- 146.5	- 146.5	-24.6	-24.6	-161.7	-164.6	-39.4	-40.4
Right End	Shear	70.61	100.23	100.22	51.64	51.64	98.17	98.17	37.57	36.72	123.21	123.21	61.48	62.33
	Moment	-50.0	-133.2	-131.2	-36.8	-35.4	- 143.9	- 143.9	-26.9	-26.9	-163.0	-163.0	-44.3	-44.4

Column Element 506:

Results of analysis under various load combinations

	Force	LC 1	LC 2	LC3	LC 4	LC 5	LC 6	LC 7	LC 8	LC9	LC 10	LC 11	LC 12	LC 13
Left End	P	-2643	-1855	-1860	-1519	-1523	-1333	-1333	-912	-911	-2221	-2222	-2642	-2643
	S	14.06	61.96	61.96	48.85	48.86	77.44	77.44	55.92	55.92	77.45	77.45	60.29	60.29
	M	15.68	116.98	146.98	101.018	99.78	147.29	147.29	120.89	120.89	147.31	146.96	126.09	126.09
Right End	P	-2605	-1932	-1831	-1495	-1500	-1315	-1315	-894	-894	-2193	-2192	-2614	-2614
	S	14.06	61.96	61.96	48.58	48.86	77.44	77.44	55.92	55.92	77.45	77.45	60.29	60.29
	M	-27.8	74.24	74.24	14.13	14.59	92.81	92.81	27.76	27.82	91.80	92.61	19.40	19.40

DESIGN OF COLUMN (Element 506):

Designed based on the maximum interaction ratio

Design Data

1. Height of column = 3.1 m
2. Section of column 500 mm X 500 mm
3. Concrete used M 25
4. Load Combination = 1.5(DL-EQ-Y)
5. Steel used Fe 415
6. Factored Axial Load = P_u = 2643.11 kN
7. Factored Moment = M_2 = 126.09 kN-m
8. Factored Moment = M_3 = 60.44 kN-m

General Requirements:

Columns subjected to bending and axial load:

As per IS 13920:1993,

IS 13920:1993 specification will be applicable if axial stress > $0.1f_{ck}$ *(Cl. no 7.1.1)*

i.e. 2643.11×1000/ (500×500) = 10.57 N/mm2 > 0.1 X 25 = 2.5 N/mm^2

Minimum dimension of the member shall not be less than 200 mm. *(Cl. no 7.1.2)*

Here the minimum dimension of the column section is 500 mm. Hence, OK.

Ratio of shortest cross sectional dimension to the perpendicular dimension shall preferably not less than 0.4. *(Cl. no. 7.1.3)*

i.e. 500/500 = 1.0 > 0.4. Hence, OK. **Vertical (Longitudinal) Reinforcement:**

Assume 20 mm ϕ with 40 mm cover (d'=40+10=50 mm, d'/D=50/500=0.1)

From Chart 44, SP 16:1980 (d'/D=0.1, 415 N/mm^2)

$Pu / (fckbD)$ = (2643.11 ×10^3)/ (25×500×500) = 0.422

$Mu / (fckbD^2)$ = (126.09 ×10^6)/ (25×500×500^2) = 0.040

Reinforcement on four sides from **Chart 44, SP 16:1980**

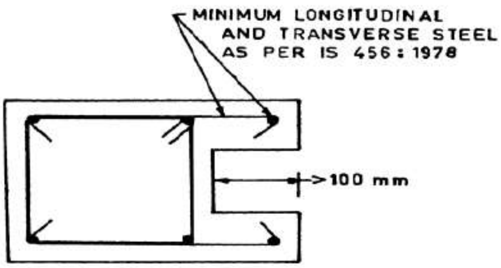
P/fck = 0.04, reinforcement in % = 0.04×25 = 1%

A_s=ρbd/100 =1.0×500×500/100 = 2500 mm^2

Provide 8@ 20mm ϕ *i.e.* 2513.27 mm^2

As per **Cl. no. 7.2.1**, Lap splices shall be provided only in central half of the member length. Hoops over the entire splice length at a spacing < 150 mm centre to centre. Not more than 50% of the bars shall be spliced at one section.

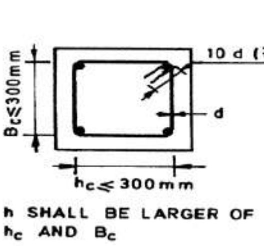
As per **Cl. no. 7.2.2**, any area of column that extends more than 100 mm should be detailed as per **Figure 6 of IS 13920: 1993.**



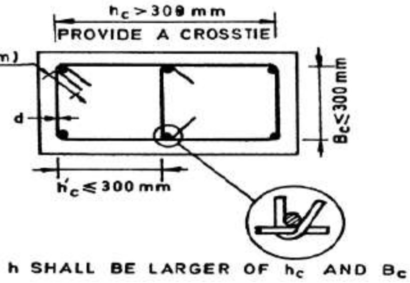
Reinforcement Requirement for Column with More Than 100 mm Projection beyond Core

Transverse Reinforcement: (As per **Cl. no. 7.3 of IS 13920: 1993)**

As per **Cl. no. 7.3.1**, Hoop reinforcement as per **Figure 7A in IS 13920: 1993.**

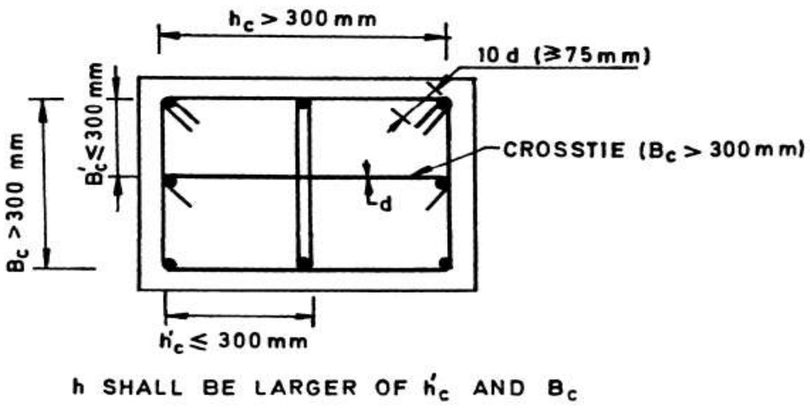


Single Hoop



Single Hoop with a Crosstie

As per **Cl. no. 7.3.2**, If the length of hoop > 300 mm a cross tie shall be provided as shown in **Figure 7B or detailed as Figure 7C in IS 13920: 1993.**



Overlapping Hoops with a Crosstie

As per **Cl. no. 7.3.3**, Hoop spacing should be greater than half the least lateral dimension of column *i.e.* 300/2 = 150 mm.

As per **Cl. no. 7.3.4**, The design shear force for the column shall be maximum of the following:

a) Calculated factored shear force as per analysis = 116.03kN

b) A factored shear force given by,

$V_u = 1.4\ [(M_u^{bL}{}_{lim} + M_u^{bR}{}_{lim})/\ (h_{st})]$ = 1.4 (236.35 +127.61)/ (3.1) = 176.56 kN.

Hence take, V_u = 176.56 kN.

Where, $M_u^{bL}{}_{lim}$ and $M_u^{bR}{}_{lim}$ are moment of resistance, of opposite sign, of beams and h_{st} is the height of the storey.

Moment of resistance of beam is,

P_t=1963.49/ (300×412.5) =1.59 % at top

P_b=981.74/(300×412.5) =0.8% at bottom

Referring **Table 51, SP 16: 1980**

$M_{u,lim}/bd^2$= 4.63 (P_t =1.59 and d'/d =0.10)

$M_{u,lim}$ (Hogging moment capacity) =4.63×300×412.5^2=236.35 kN-m

$M_{u,lim}/bd^2$ = 2.5 (P_t =0.8 and f_{ck}=25, **Table 3, SP 16: 1980**)

$M_{u,lim}$ (Sagging moment capacity) =2.5×300×412.5^2=127.61 kN-m

τ_v= V_u/ bd =(176.56 x 1000)/(500x(500-50))= 0.787 N/mm2

Referring to **Table 19 of IS 456:2000,**

τ_c for M20 and p_t=(100 A_s) /b d = (100 x 1257)/(500 x 450) = 0.56, where (A_{st} =2514/2=1257 mm^2)is

τ_c= 0.53 N/mm2

But for members subjected to axial compression P_u, the design shear strength of concrete, given in **Table 19 of IS 456:2000**, shall be multiplied by the following factors,

As per **Cl. no. 40.2.2 of IS 456:2000**, $\delta = 1+ ((3 \times P_u)/(A_g \times f_{ck}))$ but not exceeding 1.5

where, P_u=axial compression force in Newton,

A_g = Gross area of the concrete section in mm^2,

f_{ck} = characteristic compressive strength of concrete,

δ = 1+ ((3 x2643110)/ (500x450 x 25)) = 2.4 > 1.5

τ_c= 1.5 x 0.64 = 0.96 N/mm2

$V_c = \tau_c\, bd$ = 0.96 x 300 x 412.5 = 118.8 kN

As $\tau_v > \tau_c$, the shear reinforcement shall be provided in accordance with **Cl.no.40.2 of IS 456:2000.**

For vertical stirrups, V_{us}= $(0.87 \times f_y \times A_{sv} \times d)/S_v$,

$V_{us} = V_u - \tau_c\, bd$ = 176.56 - 118.8= 57.76 kN,

Use 2 legged 8 Φ stirrups, A_{sv} = 100.48 mm^2,

Then spacing of stirrups, $S_v = (0.87 \times f_y \times A_{sv} \times d)/ V_{us}$

= (0.87×415×100.48×450)/(57.76×1000)

= 282.64 mm

The spacing shall be lesser of

(a) 0.75×d=0.75×450=337.5 mm
(b) 300 mm
(c) 283mm as calculated

Provide 8mm φ two-legged stirrups about 280 mm c/c. Hoop spacing should not be greater than half the least lateral dimension *i.e.* 500/2 =250 mm. As per **Cl. no. 7.3.3 of IS 13920: 1993,** spacing of hoop reinforcement should be more than 150 mm c/c.

Therefore, finally 8 mm Φ @150 mm c/c is the permissible.

Special Confining Reinforcement: (As per **Cl. no. 7.4 of IS 13920: 1993)**

As per **Cl. no. 7.4.1**, Special confining reinforcement will provide over a length of l_o towards the mid-span of column,

The 'l_o' shall not be less than

(a) larger lateral dimension of the member = 500 mm
(b) 1/6 (clear span) = 3100/6=516.67 mm
(c) 450 mm

Adopting l_0 = 550 mm

As per **Cl. no. 7.4.6**, the spacing of hoop (S_{max}) shall not exceed the following,

(a) $\frac{1}{4}$ (minimum member dimensions) = $\frac{1}{4}$ ×500 = 125 mm
(b) Should not be less than 75 mm
(c) Should not be greater than 100 mm

Adopting S_{max} = 75mm

As per **Cl. no. 7.4.8**, minimum area of cross section of the bar forming hoop is

A_{sh} = 0.18 $Sh\ f_{ck}\,/\,f_y\,(A_g/A_k$ -1.0)

A_{sh} = 0.18×75×200×25/415 ((500×500)/(450×450)-1.0) = 38.23 mm^2

Use 8 mm φ bar (50.27 mm^2) at a spacing of 75× 50.27/38.23 =98.68 mm c/c.

i.e 90 mm c/c.

JOINT OF FRAMES

As per **Cl. no. 8.1of IS 13920: 1993**, the special confining reinforcement as required at the end of the column shall be provided through the joint as well, unless the joint is confined by **Cl. no. 8.2.**

As per **Cl. no. 8.1of IS 13920: 1993,** a joint which has beams into all vertical faces of it and beam width, is at least $\frac{3}{4}$ of the column width, shall be provided with half the special confining reinforcement required at the end of the column. The spacing of hoop shall not exceed 150 mm. Therefore, A_{sh}= 50.27/2 = 25.135 mm^2. Use 8mm dia bar (50.27 mm^2) at a spacing of 98.68×50.27/25.135 = 195 mm c/c >150mm. Provide 8mm @ 150mm c/c spacing.

DESIGN OF BEAM (Element 535)

Design Data

1. Length of beam = 4m c/c
2. Section of beam 300 mm X 450 mm
3. Load Combination = 1.5(DL-EQ-X)
4. Concrete used M 25
5. Steel used Fe 415
6. Shear force = 126.28 kN
7. Maximum hogging moment = 165.5 kN-m
8. Maximum sagging moment = 86.2 kN-m

General Requirements As Per Cl. no. 6.1 of IS 13920-1993

As per **Cl. no. 6.1.1**, Factored axial stress shall not exceed 0.1 *fck* (here, zero) (OK).

As per **Cl. no. 6.1.2,** Width = 300 mm > 200 mm (OK).

As per **Cl. no.6.1.3,** Width/depth = 300/450 = 0.67 > 0.3 (OK).

As per **Cl. no. 6.1.4,** Depth = 450 mm < 1/4 of clear span = 4000/4 = 1000mm (OK).

Design for Flexure

Longitudinal reinforcement (As Per Cl. no. 6.2 of IS 13920-1993): (At Left End)

Design hogging moment = 165.5 kN-m

Assuming 25 mm clear cover with 25 mm φ bars,
Effective depth *(d)* = 450-25-(25/2) =412.5 mm

From **Table D, SP 16: 1980**

$(M_{u,lim} / bd^2)$ = 3.45 (For M25 and Fe415)

$M_{u,lim}$= 3.45×300×412.5^2=176.11 kN-m

Actual moment 165.5 kN-m is smaller than $M_{u,lim}$, so section is singly reinforced.

Reinforcement from **Table 3, SP16:1980**

(M_u/bd^2) = (165.5 ×10^6/ (300×412.5 ^{2})) = 3.24

Referring to **Table 3, SP 16:1980** corresponding to M_u/bd^2 = 3.24 and M25

$P_{(top)}$=1.105……………………………………………………(1)

Design sagging moment = 86.2 kN-m

From **Table D, SP 16: 1980**

$M_{u,lim}$= 3.45×300×412.5^2=176.11 kN-m

Actual moment 86.2 kN-m is smaller than $M_{u,lim}$, so section is singly reinforced.

Reinforcement from **Table 3, SP 16: 1980**

(M_u/bd^2) = (86.2 ×10^6/ (300×412.5 ^{2})) = 1.69

Referring to **Table 3, SP 16:1980** corresponding to M_u/bd^2 = 1.69 and M25

$P_{(bottom)}$ =0.515…………………………………………………(2)

Required reinforcement maximum of equations (1) and (2), *i.e.*

$P_{(top)}$ = 1.105 and $P_{(bottom)}$=0.515

Reinforcement at top (A_t) =1.105×300×412.5 =1367.43 mm^2 , Provide 4 nos. of 25 mm φ

($A_{t\ (Provided)}$) = 1963.49 mm^2)

Reinforcement at top (A_b) =0.515×300×412.5= 637.31 mm^2 , Provide 2 nos. of 25 mm φ

($A_{b\ (Provided)}$) = 981.74 mm^2)

Ductile Detailing Considerations:

As per **IS 13920: 1993 Cl. no. 6.2.1**,

a) The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.
b) The tension steel ratio on any face, at any section, $\rho_{min} \leq 0.24\sqrt{fck}/fy$, *i.e.* 0.289 given 0.515 (OK).

As per **IS 13920: 1993 Cl. no. 6.2.2**, Maximum steel ratio on any face at any section, shall not exceed, ρ_{max} = 2.5% given 1.105 (OK).

Longitudinal reinforcement (As Per Cl. no. 6.2 of IS 13920-1993): (At Right End)

Design hogging moment = 163.23 kN-m

From **Table D, SP 16: 1980**

$(M_{u,lim} / bd^2)$ = 3.45 (For M25 and Fe415)

$M_{u,lim}$= 3.45×300×412.5^2=176.11 kN-m

Actual moment 163.23 kN-m is smaller than $M_{u,lim}$, so section is singly reinforced.

Reinforcement from **Table 3, SP16:1980**

(M_u/bd^2) = (163.23 ×10^6/ (300×412.5 ^{2})) = 3.19

Referring to **Table 3, SP 16:1980** corresponding to M_u/bd^2 = 3.19 and M25

Hence, $P_{(top)}$ = 1.082 ……………………………………………(3)

Design sagging moment = 76.23 kN-m

From **Table D, SP 16: 1980**

$M_{u,lim}$= 3.45×300×412.5^2=176.11 kN-m

Actual moment 76.23 kN-m is smaller than $M_{u,lim}$, so section is singly reinforced.

Reinforcement from **Table 3, SP 16: 1980**

(M_u/bd^2) = (76.23 ×10^6/ (300×412.5 ^{2})) = 1.49

Referring to **Table 3, SP 16:1980** corresponding to M_u/bd^2 = 1.49 and M25

$P_{(bottom)}$=0.449…………………………………………………(4)

Required reinforcement maximum of equations (1) and (2), *i.e.*

$P_{(top)}$ = 1.082 and $P_{(bottom)}$=0.449

Reinforcement at top (A_t) =1.082×300×412.5 =1338.97 mm^2 , Provide 4 nos. of 25 mm φ

($A_{t\ (Provided)}$) = 1963.49 mm^2)

Reinforcement at top (A_b) =0.449×300×412.5=555.63 mm^2 , Provide 2 nos. of 25 mm φ

($A_{b\ (Provided)}$) = 981.74 mm^2

Ductile Detailing Considerations:

As per **IS 13920: 1993 Cl. no. 6.2.1**,

The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.

The tension steel ratio on any face, at any section, $\rho_{min} \leq$ *0.24√fck/fy, i.e.* 0.289 given 0.472 (OK).

As per **IS 13920: 1993 Cl. no. 6.2.2**, Maximum steel ratio on any face at any section, shall not exceed, ρmax = 2.5% given 1.145 (OK).

Shear reinforcement (As Per Cl. no. 6.3 of IS 13920-1993):

As per Cl. no. 6.3.1, web reinforcement shall consist of vertical hoops.

As per **Cl. no. 6.3.2**, minimum diameter of hoop 6mm and in the case of beam with clear span greater than 5m, then the hoop diameter should be 8 mm.

As per **Cl. no. 6.3.3,**

(a) Calculated factored shear force as per the analysis *i.e.* 126.28 kN.
(b) Shear force due to formation of plastic hinges at both ends of the beam.

At Left End:

P_t=1963.49/ (300×412.5) =1.59 % at top

P_b=981.74/(300×412.5) =0.8% at bottom

Referring **Table 51, SP 1980**

$M_{u,lim}/bd^2$= 4.63 (P_t =1.59 and d'/d =0.10)

$M_{u,lim}$ (Hogging moment capacity) =4.63×300×412.5^2=236.35 kN-m

$M_{u,lim}/bd^2$ = 2.5 (P_t =0.8 and f_{ck}=25, **Table 3, SP 1980**)

$M_{u,lim}$ (Sagging moment capacity) =2.5×300×412.5^2=127.61 kN-m

At Right End:

P_t=1963.49/ (300×412.5) =1.59 % at top

P_b=981.74/(300×412.5) =0.8% at bottom

Referring **Table 51, SP 1980**

$M_{u,lim}/bd^2$= 4.63 (P_t =1.59 and d'/d =0.10)

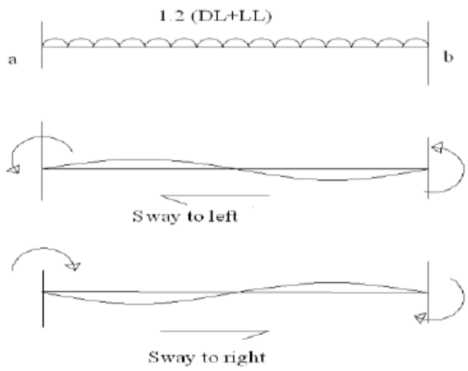
$M_{u,lim}$ (Hogging moment capacity) =4.63×300×412.5^2=236.35 kN-m

$M_{u,lim}/bd^2$ = 2.5 (P_t =0.8 and f_{ck}=25, **Table 3, SP 1980**)

$M_{u,lim}$ (Sagging moment capacity) =2.5×300×412.5^2=127.61 kN-m

Design for Shear

The deformed shapes of beam due to sway to the left and sway to the right are shown in Fig.



Deformed Shapes of Beam

Shear force due to 1.2(DL + LL) at **Left end** and **Right end** are given by,

$V_a^{D+L} = V_b^{D+L}$ = 1.2 x 32 x (clear span)/2

= 1.2 x 32 x (4/2)

= 76.8 KN

Shear Force Due to Plastic Hinge Formation at the Ends of Beam

$V_{sway\ to\ right} = \pm 1.4\ (M_u^{As} + M_u^{Bh})\ /L$

$V_{sway\ to\ left} = \pm 1.4\ (M_u^{Ah} + M_u^{Bs})\ /L$

For Sway to Right

$V_{ua} = V_a^{D+L} - 1.4\ (M_u^{As} + M_u^{Bh})\ /L$

V_{ua} = 76.8 -1.4 (127.61 + 127.61) /4

V_{ua} = -12.53 kN

$V_{ub} = V_b^{D+L} + 1.4\ (M_u^{As} + M_u^{Bh})\ /L$

V_{ub} = 76.8 + 1.4 (127.61 + 127.61) /4

V_{ub} = 166.13kN

For Sway to Left

$V_{ua} = Va^{D+L} + 1.4\ (M_u^{Ah} + M_u^{Bs})\ /L$

V_{ua} = 76.8 + 1.4 (236.35 + 236.5) /4

V_{ua} = 242.35 kN

$V_{ub} = V_b^{D+L} - 1.4\ (M_u^{Ah} + M_u^{Bs})\ /L$

V_{ub} = 76.8 - 1.4 (236.35 + 236.5) /4

V_{ub} = -88.65 kN

As per **Cl.no.6.3.3 of IS 13920**, the design shear force to be resisted shall be the maximum of:

a) Calculated factored shear force as per analysis.
b) Shear force due to formation of plastic hinges at the both the ends of the beam plus factored gravity load on the span.

But, design shear force as per analysis = 126.28 kN < Shear force due to plastic hinge formation = 242.35kN. Hence the design shear force V_u will be 242.35 kN .

Now, $\tau_v = V_u / (b \times d)$

=242.35 x 1000/(300 x 412.5)

= 1.96 N/mm^2

τ_c = 0.582 N/mm^2 for M 25 concrete and p_t = 0.793 % (Ref IS 456:2000 Table 19),

$V_{us} = (\tau_v - \tau_c) \times b \times d$

V_{us} = (1.96 – 0.582) x 300 x 412.5/1000 = 170.52 kN

Use 2 legged 8 Φ stirrups

A_{sv} = 100.48 mm^2

Spacing of stirrups required within a Distance of 2d = 2 x 412.5= 905mm

from the face of columns (S_v) (mm) (Ref IS 456:2000, Cl.26.5.1.5, and IS 13920:1993 Cl.6.3.5),

S_v = Minimum of

(a) 0.75 x d

= .75 x 412.5 = 309.375 mm

(b) 300 mm,

(c) 0.87 x f_y x A_{sv}/(0.4 x b),

= 0.87 x 415 x 100.48/(0.4 x 300) = 302.31 mm

(d) 0.87 x b x A_{sv} x d / V_{us}

= 0.87 x 300 x 100.48 x 452.5 / 109.68 x 10^3 = 108.19 mm

(e) d / 4,

= 412.5 / 4 = 103.125 mm

(f) 8 x minimum diameter of rebar used, but not > 100 mm.

= 8 x 25 = 200mm

However, it should not to be less than 100 mm. Therefore provide **100 mm** spacing.

Spacing of stirrups required beyond a distance of 2d from the face of columns (S_v) (mm)

(Ref IS 456:2000, Cl. 26.5.1.5, and IS 13920:1993 Cl. 6.3.5),

S_v = Minimum of

(a) 0.75 x d

= 0.75 x 412.5 = 309.75 mm

(b) 300 mm

(c) 0.87 x f_y x A_{sv} / (0.4 x b)

= 0.87 x 415 x 100.48/(0.4 x 300) = 302.31 mm

(d) 0.87 x f_y x A_{sv} x d / V_{us}

= 0.87 x 415 x 100.48 x 412.5/170.52 x 10^3 = 87.76 mm

(e) d / 2

= 412.5/2 = 206.25 mm

Provide **200** mm spacing.

Check:

$(0.87 \times f_y \times d \times A_{sv} / S_v + \tau_c \times b \times d) < (\tau_{c,max} \times b \times d)$

Where,

Maximum Permissible Shear Stress ($\tau_{c,max}$) (Ref IS 456:2000, Table 20, for given f_{ck} value)

= 3.1 N/mm^2

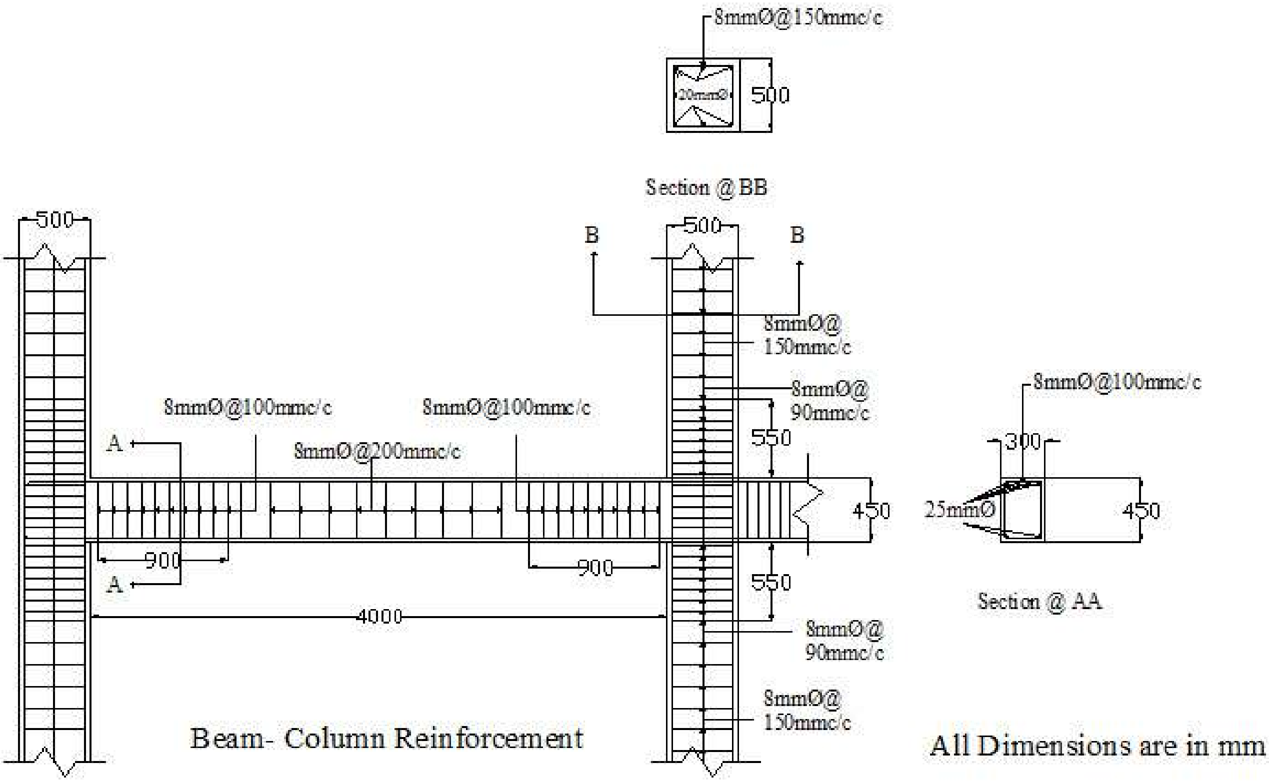
Therefore,

L.H.S. = 0.87 x 415 x 412.5 x 100.48/100 + 0.582x 300 x 412.5

= 221.71 kN

R.H.S = 3.1 x 300 x 412.5

= 383.63 kN > L.H.S., O.K.



Beam and Column Reinforcement Details As per IS 13920: 1993

Further Reading

1. Agarwal, P. and Shrinkhande, M. (2006). "Earthquake Resistant Design of Structures", Prentice Hall of India Ltd., New Delhi

3.2 How to carry out seismic design of a RC shear wall?

Problem Statement: Two major types of lateral force resisting systems have been suggested namely, moment resisting RC frame system and shear wall system. The moment resisting RC frame systems have several seismic deficiencies like certain zone of failures such as at beam-column joints, or column ends, shear failure, anchorage failure and splice failure of reinforcing bars, short columns and soft or weak stories. The shear wall system often proves to be a better choice to resist lateral loads since shear walls are the seismic collapse insurance of a building. IS 13920: 1993 gives complete details of ductile design of shear wall provide in Section 9, Annexure A of the code. To illustrate of these clause for the design of shear wall, two typical examples are considered with the assumed data. The purpose of these examples to explain the clauses of IS 13920: 1993 step-by-step[1].

TYPICAL EXAMPLE 1: STRUCTURAL DESIGN OF A DUCTILE SHEAR WALL AS PER IS-13920:1993

Typical stress resultant for the assumed shear wall

- Shear force, $V_u = 1800\, kN$
- Axial force, $P_u = 5000 kN$
- Moment, $M_u = 12500\, kN - m$
- Axial force on boundary element = 1000 KN

Material properties for concrete & steel are assumed as follows:

Grade of concrete M 20, $f_{ck} = 20\, MPa$
Reinforcing steel grade HYSD, $Fe\,415$

CL- 9.1.2: (Minimum thickness >= 150mm)
Assuming thickness of shear wall = 250mm
Length of the wall $l_w = 6000\, mm$

CL- 9.1.5: Since the thickness of shear wall is 250mm i.e >200 mm and also the factored shear stress τ_v is greater than $0.25\sqrt{f_{ck}}$ the reinforcement (vertical as well as horizontal) shell be provided in two curtains.

CL-9.1.6: Diameter of bar used in horizontal and vertical reinforcement is 8 mm, which is smaller than $\frac{1}{10} \times 250 = 25\,mm$ **OK.**

CL-9.1.7: Maximum spacing of reinforcement shall not exceed the following

(i) $\dfrac{l_w}{5} = \dfrac{6000}{5} = 1200\,mm$

(ii) $3t_w = 750\,mm$

(iii) $450\,mm$

Providing spacing in horizontal & vertical direction as 150 mm

Shear strength Requirement:

CL-9.2.1: Nominal shear stress in the wall,

$$\tau_v = \frac{V_u}{t_w d_w}$$

$$d_w = 6000 \times 0.8\ 4800$$

$$\tau_v = \frac{1800}{250 \times 4800} = 1.5\,MPa$$

CL-9.2.2: Design strength of concrete as per **T-13 of IS 456:2000**

For 0.25% reinforcement & M20 grade concrete, $\tau_c = 0.36MPa$

CL-9.2.3: The nominal shear stress τ_c, shall not exceed $\tau_{c\max}$ as per **T-14 of IS 456:2000**

$\tau_{c\max}$ for M20 = 2.8 MPa.

$\because \tau_v (1.5) < \tau_{c\max} (2.8)$ **OK.**

CL- 9.2.4: $\because \tau_v (1.5) > \tau_c\ (0.36)$ then shear reinforcement shall be provided.

CL-9.2.5:

$$V_w = (\tau_v - \tau_c) \times t_w d_w$$

$$= (1.5 - 0.36) \times 250 \times 4800 = 1368\,kN$$

Assuming 2-legged 8 mm diameter horizontally aligned closed stirrups along height of the wall

Spacing along height of the wall

Spacing of stirrups, $S_v = \dfrac{0.87\, f_y A_w d_w}{V_w}$

$$= \frac{0.87 \times 415 \times 2 \times 50.26 \times 4800}{1368000} = 128.20\, mm\ c/c$$

$$\rho = \frac{2 \times \frac{\pi}{4} \times 8^2}{128.20 \times 250} = 0.0032$$

Providing spacing of 125 mm c/c

$\rho = 0.00324$ **OK.**

Hence providing 2-Legged 8 mm diameter horizontally aligned closed stirrups @125 mm c/c along entire height of shear wall.

CL-9.2.6: Vertical reinforcement shall not be less than horizontal reinforcement.

$\therefore$ **Vertical reinforcement:** 2-L 8mmφ @125 mm c/c

Check for flexural strength: **(with reference to Annex ‘A’)**

CL- 9.3.1:

$$\left(\frac{x_u}{l_w}\right) = \left(\frac{\phi + \lambda}{2\phi + 0.36}\right) \quad ; \quad \left(\frac{x_u}{l_w}\right) = \frac{0.0035}{0.0035 + \dfrac{0.87\, f_y}{E_s}}$$

$$\phi = \frac{0.87\, f_y \rho}{f_{ck}} \quad , \quad \lambda = \frac{P_u}{f_{ck} t_w l_w}$$

$$\rho = \frac{A_{st}}{t_w l_w} \quad (Vertical\ reinforcement\ ratio)$$

$$\rho = \frac{2 \times \frac{\pi}{4} \times 8^2 \times 4800}{250 \times 125 \times 4800} = 0.0032$$

$$\phi = \frac{0.87 \times 415 \times 0.0032}{20} = 0.0577$$

$$\lambda = \frac{5000 \times 1000}{20 \times 250 \times 4800} = 0.2083$$

$$\frac{x_u}{l_w} = \frac{0.0577 + 0.2083}{2 \times 0.0577 + 0.36} = 0.5595$$

$$\frac{x_u^*}{l_w} = \frac{0.0035}{0.0035 + \dfrac{0.87 \times 415}{2 \times 10^5}} = 0.6597$$

$$\because \frac{x_u}{l_w} \le \frac{x_u^*}{l_w}$$ $\therefore$ Using equation (a) for calculating M_{uv}

$$\therefore \frac{M_{uv}}{f_{ck} t_w l_w^2} = \phi \left[\left(1 + \frac{\lambda}{\phi}\right)\left(\frac{1}{2} - 0.416 \frac{x_u}{l_w}\right) - \left(\frac{x_u}{l_w}\right)^2 \left(0.168 + \frac{\beta^3}{3}\right) \right]$$

$$\beta = \frac{0.87 f_y}{0.0035 E_s} = 0.516$$

$$\therefore \frac{M_{uv}}{f_{ck} t_w l_w^2} = 0.0577 \left[\left(1 + \frac{0.2083}{0.0577}\right)\left(\frac{1}{2} - 0.416 \times 0.5595\right) - (0.5595)^2 \left(0.168 + \frac{0.516^3}{3}\right) \right]$$

$$= 0.0665$$

$$\therefore M_{uv} = 0.0665 \times 20 \times 250 \times 6000^2 = 1197\, kN - m$$

Remaining moment; $M_u - M_{uv} = 12500 - 11970 = 530\, kN - m$ shall be resisted by reinforcement in boundary element.

Check on Boundary elements:

Cross-sectional properties:-

$$l_w = 6000\, mm, t_w = 250\, mm$$

$$A_g = 1500000\, mm^2$$

$$I_y = {t_w l_w^3}/{12} = {250 \times 6000^3}/{12} \;\; 4.5 \times 10^{12} \;\; mm^4$$

$$f_c = \frac{P_4}{A_g} \pm \frac{M_u}{I_y} = \frac{5000 \times 10^3}{1500 \times 10^3} \pm \frac{12500 \times 10^6 \times 3000}{4.5 \times 10^{12}}$$

$$= 11.67$$

$$\because 11.67\, N/mm^2 > 4.0\, N/mm^2 \;\; (0.2\, f_{ck})$$

Hence providing Boundary element

Design of Boundary element:

Providing boundary element of length 600 mm & width 250mm at each end of the shear wall
CL-9.4.2: Axial compressive load on building element due to seismic forces

$$= {(M_u - M_{uv})}/{C_w}$$

$$= {(12500 - 11970}/{5.4} \quad = 98.148\, kN$$

Required axial load capacity of boundary element $\approx 100 kN + 1000\, kN = 1100\, kN$

CL-9.4.4: Providing nominal minimum reinforcement i.e. 0.8%

$$\therefore A_{sc} \;\; provided = \frac{0.8 \times 600 \times 250}{100} = 1200\, mm^2$$

CL-9.4.2: Boundary element shall be assumed to behave as a axially loaded short column

$$\therefore P_b = 0.447\, f_{ck} A_g + (f_s - 0.477\, f_{ck}) A_{sc}$$

$$f_{sc} = 0.79\, f_y \quad for \quad f_e\, 415 \;\; steel$$

$$P_b = 0.447 \times 20 \times 600 \times 250 + (0.79 \times 415 - 0.44 \times 20) \times 1200$$

$$= 1723.692\, kN > 1100\, kN$$

OK.

Area of steel for each boundary element = 1200mm^2

Providing 6 nos. 16φ in each boundary element A_{sc} provided = $6 \times \frac{\pi}{4} \times 16^2 \;\; = 1206\, mm^2$

CL-9.4.5: Special confining reinforcement bar as per **CL 7.4.8**

Cross-sectional area of special confining reinforcement bar

$$A_{sh} = 0.18 Sh \frac{f_{ck}}{f_y}\left[\frac{A_g}{A_k} - 1.0\right]$$

$$A_g = 600 \times 250 = 1500 \times 10^3 \, mm^2$$

$$A_k = (600 - 240) \times (250 - 2 \times 40) = 520 \times 170 \, mm^2$$

Using one cross tie as dimension is larger than 300 mm

$$\therefore h = \frac{520}{2} = 260 \, mm$$

CL- 7.4.6: Spacing of confining rectangular loop,

$$S = \frac{1}{40} \times 250 = 62.5 \, mm$$

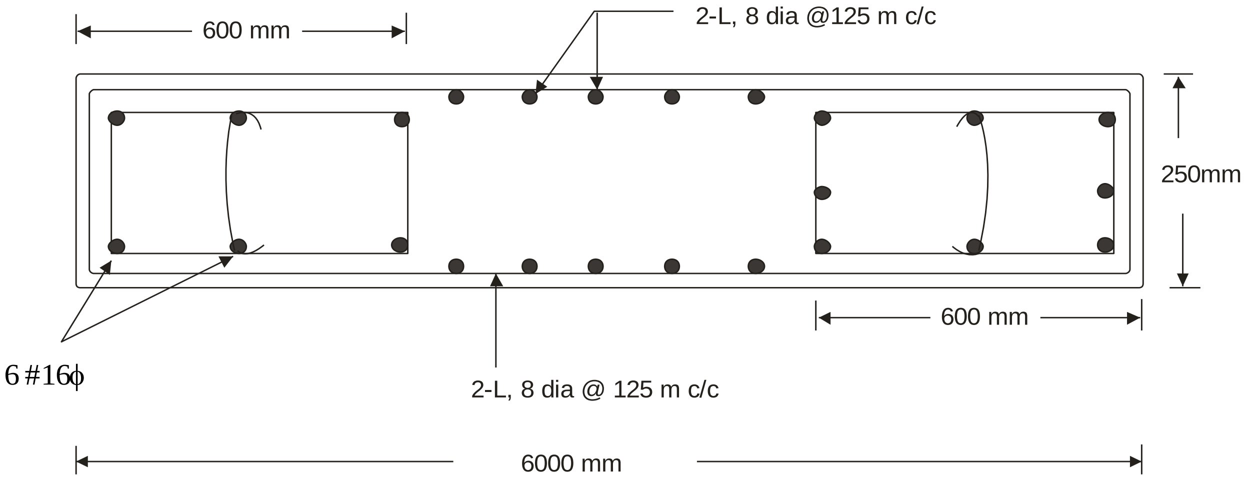
Not less than 75 mm & not greater than 100 mm

$$\therefore S = 75 \, mm$$

$$\therefore A_{sh} = 0.18 \times 75 \times 260 \times \frac{20}{415}\left[\frac{600 \times 250}{520 \times 170} - 1.0\right]$$

$$= 117.87 \, mm^2$$

Adopting 16mm diameter bar for confining loop at a spacing of 75 mm c/c.



Sectional plan of shear wall

TYPICAL EXAMPLE 2: STRUCTURAL DESIGN OF A DUCTILE SHEAR WALL AS PER IS-13920:1993

Typical stress resultants for an assumed shear wall:

- Shear force, $Vu = 600\,kN$
- Axial force, $Pu = 8120\,kN$
- Axial force on boundary element = 1550 *KN*.
- Moment, $Mu = 6105\,kN - m$

The material properties for concrete and reinforcing steel are assumed as follows:

- Concrete grade, $M30$, $f_{ck} = 30\,MPa$
- Reinforcement steel grade HYSD Fe-415

CL-9.1.2: (Minimum thickness should not be less than 150 mm)

Assuming thickness of shear wall = 250mm

Length of the wall, $L_w = 4000\,mm$

CL-9.1.5: Since the thickness of shear wall is 250 mm i.e. > 200mm and also the factored shear stress (τ_v) is greater than 0.25 $\sqrt{f_{ck}}$ the reinforcement (vertical as well as horizontal) shall be provided in two curtains.

CL-9.1.6: Diameter of bar used is horizontal and vertical reinforcement is 8 mm, which is smaller than $\frac{1}{10}(250) = 25\,mm$

CL-9.1.7: Maximum spacing of reinforcement shall not exceed the follo0wing

(i) $L_w/5 = 4000/5 = 800\,mm$

(ii) $3t_w = 3 \times 250 = 750mm$

(iii) 450 mm

Providing spacing in horizontal & vertical directions as 150 mm

Shear Strength requirement:

CL-9.2.1: Nominal stress is the wall, $\tau_v = \dfrac{V_u}{t_w dw}$

$$d_w = 0.8 \times L_w = 0.8 \times 4000 = 3200$$

$$\tau_v = \frac{600 \times 10^3}{250 \times 3200} = 0.75\, N/mm^2$$

For M-30 grade concrete $\tau_{c\max} = 3.5\, MPa$ **(T-20, IS-456:2000)**

Assuming minimum (0.25%) steel, in the wall for vertical as well as horizontal direction.

For $P_t = 0.25\%, \tau_c = 0.37\, MPa$ **(T-19, IS-456:2000)**

Since, $\tau_c < \tau_v < \tau_{c\max}$; shear reinforcement will be required.

Horizontal shear reinforcement:-

CL-9.2.5: $V_w = (\tau_v - \tau_c) \times t_w d_w = 304\, kN$

Assuming 2-legged 8 mm diameter horizontally aligned closed stirrups along the height of wall

Spacing of Stirrups = $S_v = 0.87\, f_y A_h dw / V_{us}$

$$= \frac{0.87 \times 415 \times 2 \times 50.26 \times 3200}{304 \times 1000} = 382.028\, mm \ \ c/c$$

$$= \frac{2 \times \frac{\pi}{4} \times 8^2}{382.028 \times 250} = 0.00105$$

Providing spacing of 150 mm c/c
Then, $\rho = 0.00268$ **OK.**

Hence providing 2-Legged 8 mm diameter horizontally aligned closed stirrups at 150 mm c/c along entire height of the shear wall

CL-9.2.6: Vertical reinforcement shall be uniformly distributed in the wall section and, shall not be less than the horizontal reinforcement

Vertical reinforcement: 2-legged 8 mm diameter @ 150 mm c/c

Check for flexural strength: **(with reference to Annex 'A')**

CL-9.3.1:

$$\frac{x_u}{L_w} = \left(\frac{\phi + \lambda}{2\phi + 0.36}\right) \quad , \quad \frac{x_u^{\ *}}{L_w} = \frac{0.0035}{0.0035 + \frac{0.87 f_y}{E_s}}$$

$$\varphi = \frac{0.87 f_y \times p}{f_{ck}} \quad , \quad \lambda = P_u \big/ f_{ck} t_w l_w$$

$$\rho = \frac{A_{st}}{t_w l_w} \quad (Vertical\ re\inf\ orcement\ ratio)$$

$$\rho = \frac{2 \times \frac{\pi}{4} \times 8^2 \times 3200}{250 \times 150 \times 3200} = 0.00268$$

$$\phi = \frac{0.87 \times 415 \times 0.00268}{30} = 0.03225$$

$$\lambda = \frac{8120 \times 1000}{30 \times 250 \times 3200} = 0.3383$$

$$\frac{x_u}{l_w} = \frac{0.03225 + 0.3383}{2 \times 0.003225 + 0.36} = 0.8729$$

$$\frac{x_u^{\ *}}{l_w} = \frac{0.0035}{0.0035 \times \frac{0.87 \times 415}{2 \times 10^5}} = 0.6597$$

$$\because \frac{x_u^{\ *}}{l_w} < \frac{x_u}{l_w} < 1.0;$$

Hence equation (b) of Annex ‘A’ (IS-13920:1993) will be applicable.

$$\frac{M_w}{f_{ck} t_w (l_w)^2} = \alpha_1 (x_u / l_w) - \alpha_2 (x_u / l_w)^2 - \alpha_3 - \lambda / 2$$

$$\alpha_1 = [0.36 + \phi(1 - \beta / 2 - 1 / 2\beta)]$$

$$\alpha_1 = [0.36 + 0.03225\left(1 - \frac{0.516}{2} - \frac{1}{2 \times 0.516}\right) = 0.3526$$

$$\beta = \frac{0.87 f_y}{0.0035 E_s} = \frac{0.87 \times 415}{0.0035 \times 2 \times 10^5} = 0.516$$

$$\alpha_2 = \left[0.15 + \frac{\phi}{2}\left(1 - \beta - \frac{\beta^2}{2} - \frac{1}{3\beta}\right)\right]$$

$$= \left[0.15 + \frac{0.03225}{2}\left(1 - 0.516 - \frac{0.516^2}{2} + \frac{1}{3 \times 0.516}\right)\right]$$

$$= 0.166$$

$$\alpha_3 = \frac{\phi}{6\beta}\left(\frac{1}{(x_u / l_w} - 3\right) = \frac{0.03225}{6 \times 0.516}\left(\frac{1}{0.8729} - 3\right)$$

$$= -0.0193$$

The value of $\left(\frac{x_u}{l_w}\right)$ to be used for computing α_3 should preferably be determined by solving the following quadratic equation.

$$\alpha_1\left(\frac{x_u}{l_w}\right)^2 + \alpha_4\left(\frac{x_u}{l_w}\right) - \alpha_5 = 0$$

$$\alpha_4 = \left(\frac{\phi}{\beta} - \lambda\right) = -0.2758$$

$$\alpha_5 = \left(\frac{\phi}{2\beta}\right) = 0.0313$$

$$Let, \frac{x_u}{l_w} = z$$

Then the above equation can be written as

$$0.3526z^2 + (-0.2758) - 0.0313 = 0$$

$$z_1, z_2 = \frac{0.2758 \pm \sqrt{(0.2758)^2 + 4 \times 0.3526 \times 0.0313}}{2 \times 0.3526}$$

$$= \frac{0.2758 \pm 0.3467}{0.7052} = 0.8827, -0.1005$$

Considering the positive root i.e.

$$z = \frac{x_u}{l_w} = 0.8827$$

$$\therefore \alpha_3 = \frac{0.03225}{6 \times 0.516}\left(\frac{1}{0.8827} - 3\right) = -0.0194$$

$$\therefore \frac{M_{uv}}{f_{ck} t_w (l_w)^2} = \alpha_1\left(\frac{x_u}{l_w}\right) - \alpha_2\left(\frac{x_u}{l_w}\right)^2 - \alpha_3 - \lambda / 2$$

$$= 0.3526(0.8827) - 0.166(0.8827)^2 - (-0.0194) - 0.3383/2$$

$$= 0.03215$$

$$\therefore M_{uv} = 0.03215 \times 30 \times 250 \times (3200)^2 = 2469.12\, kN - m$$

Remaining Moment

$$= 6105 - 2469.12 = 3635.88\, kN - m$$

Shall be resisted by reinforcement in boundary element

Check on Boundary Elements:

Cross-sectional properties:-

$$l_w = 4000\, mm, t_w = 250\, mm$$

$$A_g = 1000 \times 10^3 \; mm^2$$

$$I_y = {t_w l_w^3}/{12} = \frac{250 \times 4000^3}{12} = 1.333 \times 10^{12}\, mm^4$$

Combined stress at edge of wall

$$\sigma = \frac{P_u}{A_g} \pm \frac{M_u}{I_y}(y) \qquad y = \frac{l_w}{2}$$

$$= \frac{8120 \times 10^3}{1000 \times 10^3} \pm \frac{6105 \times 10^6}{1.333 \times 10^{12}} \times \left(\frac{3200}{2}\right)$$

$$= 8.120 \pm 7.328$$

Maximum stress = 15.448 N/mm^2

CL-9.4.1:

$$0.2 f_{ck} = 0.2 \times 30 = 6 < 15.448\, MPa$$

$$If\ \ 0.2\ f_{ck} < \ \ extreme\ fibre\, compressive\, stress\ \ \therefore$$ *Boundary elements are required in shear wall.*

Providing a boundary element of length 700mm and width 250mm at each end of the shear wall

Design of boundary elements:

CL-9.4.2: Axial Compressive load on the boundary element due to the seismic forces

$$= (M_u - M_{uv}) / C_w$$
$$= (6105 - 2469.12) / 3.3$$
$$= 1101.78\, kN$$

Required axial load capacity of boundary element = axial load due to gravity effects + axial load due to seismic forces = 1150+1101.78 = 2651.78 KN

CL- 9.4.4: Adopting 2% vertical reinforcement in boundary elements.

$$A_{sc}\, provided = \frac{2 \times 700 \times 250}{100} = 3500 mm^2$$

CL- 9.4.2: The boundary element shall be assumed to behave as a axially loaded short column.

$$\therefore P_b = 0.447\, f_{ck} A_g + (f_{sc} - 0.447\, f_{ck}) A_{sc}$$

$$f_{sc} = 0.79\, f_y \quad for\ \ f_e\, 415\ steel$$

$$P_b = 0.447 \times 30 \times 700 \times 250 + (0.79 \times 415 - 0.447 \times 30) \times 3500$$
$$= 3447.290\, kN > 3651.78\, kN$$

OK.

Area of steel for each boundary element= 3500 mm^2

Providing 6 # 22$^{\phi}$ + 4 # 16$^{\phi}$ in each element

A_{sc} provided = 3537.43 > 3500mm^2 **OK.**

CL-9.4.5: Special confining reinforcement in Boundary element as per **CL-7.4.8**

Area of special confining reinforcement bar

$$A_{sh} = 0.18 Sh \frac{f_{ck}}{f_y} \left[\frac{A_g}{A_k} - 1.0 \right]$$

$$A_g = 700 \times 250 = 175000\, mm^2$$

$$A_k = (700 - 2 \times 40) \times (250 - 2 \times 40) = 105400 mm^2$$

Since the dimension of core is greater than 300mm, cross tie will have to be used.

$$\therefore\ h = \frac{620}{3} = 206.67\, mm$$

CL-7.4.6: Spacing S of confining rectangular loop:-

$$\frac{1}{4} \times 250 = 62.5\, mm$$

Not less than 75 m & not more than 100mm

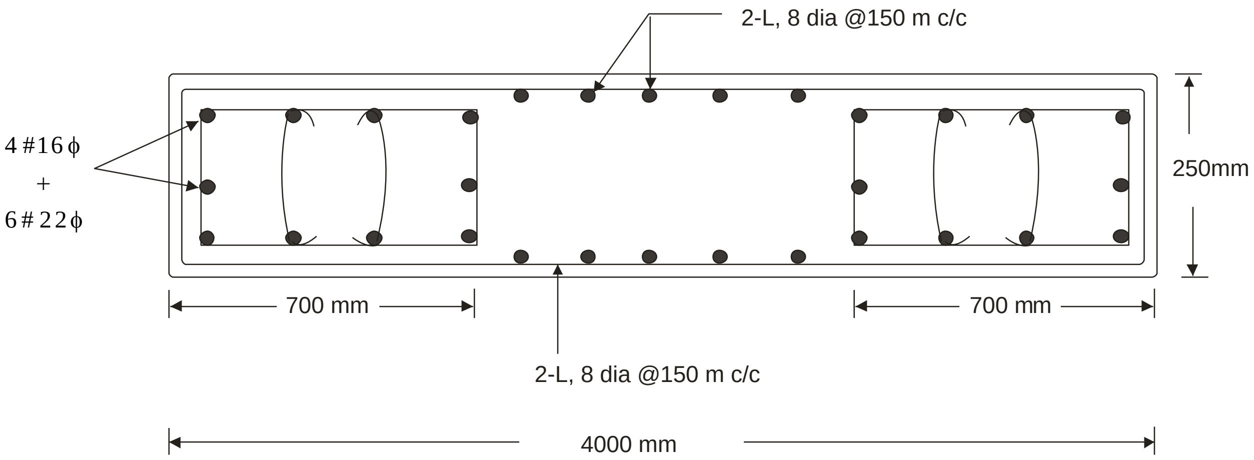
$$\therefore S = 75 mm$$

$$A_{sh} = 0.18 \times 75 \times 206.67 \times \frac{30}{415}\left[\frac{700 \times 250}{620 \times 170} - 1\right]$$

$$= 133.18\, mm^2$$

Adopting 16 mm diameter confining loop at a spacing 75 mm c/c

A_{sc} provided = 201mm^2 > 133.18mm^2, **OK.**



Sectional plan of shear wall

Further Reading

1. Agarwal, P. and Shrinkhande, M. (2006). "Earthquake Resistant Design of Structures", Prentice Hall of India Ltd., New Delhi

2. Bhupinder Singh (2011). "Structural Design of ductile shear wall" The Indian Concrete Journal, Vol. 85, No. 5. Pages 51-56.

Chapter 4

Procedures for the Seismic Evaluation of an RC building?

4.1 How to carry out seismic evaluation of existing building by Rapid Visual Screening (RVS) Procedure?

Problem Statement

The purpose of the Rapid Visual Screening (RVS) procedure is to identify those buildings that pose potentially serious loss of life and injury during a damaging earthquake. Building identified as potentially hazardous by this procedure requires further evaluation in detail by a design professional experienced in seismic design. The RVS can also be used for other purposes such as crating a ranking of the building based on their seismic hazard, retrofitting needs, programming of mitigation programme, earthquake damage and loss impact assessments, planning post earthquake building safety evaluation efforts etc. This procedure is particularly helpful after the severe earthquake in a region in which a large number of buildings have to be evaluated in limited time. In this procedure a building is to be evaluated in about 30 to 60 minutes.

Overview of RVS Procedure

The complete detail of RVS procedure is given in ATC-21 Handbook, also known as FEMA Report 154, and ATC-21-1 Supporting Documentation also known as FEMA Report 155 entitled "Rapid Visual Screening of Buildings for Potential Seismic Hazards", Figure 1. RVS procedure starts with the filling of Data Collection Forms of different seismicity (low, medium & high) as shown in Figure 2a with the help of visual inspection. A sample of data collection form is shown in Figure 2b while the details of this form are explained in Figure 2c. These forms have enough information regarding number of stories, year of construction, total floor area, a rough sketch of plan and elevation of the building alongwith photographs, type of soil, type of occupancy and occupancy load, non-structural elements. Depending upon the lateral load resisting system, Figure 3 and its material, a Basic Structural Hazard Scores (S_0) for various building types as given in Table 1 is provided on the form and the evaluator/ screener encircle the appropriate one. The evaluator then modifies this Basic Structural Hazard Score (S_0) by the Performance Modification Factors (PMF) which depend on number of factors as given in Figure 4 which relates to significant seismically related defects the evaluator/ screener may observe, in order to arrive a Final Score, S (by adjusting the Basic Structural hazard Score with the PMF). The values of PMF for the selected type of buildings in different seismicity (low, medium and high) are given in Table 2 as shown in Figure 5. The typical range of Final Score (S), Figure 6 is in between 0 to 7. Higher score corresponds to better performance and vice versa. A building having Final Structural Score (S) lower than 2 should be further investigated by the design professional experienced in seismic design. The procedure involves visual inspection of the building, normally from the outside and inspection of the building and completion of the form is assumed to require approximately 30 minutes per building.

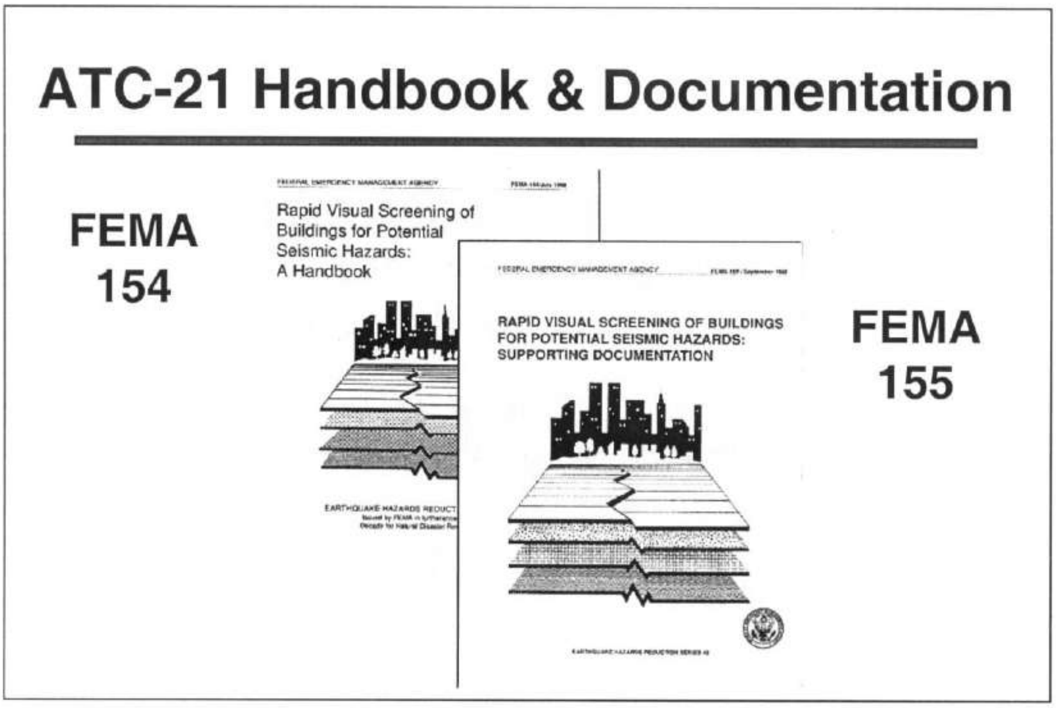


Figure 1: The Applied Technology Council, or ATC, a non-profit California corporation founded to advance the practice of structural engineering, has developed and published this information a handbook, known as ATC-21. The Federal Emergency Management Agency (FEMA) provided funding for the ATC -21, project.

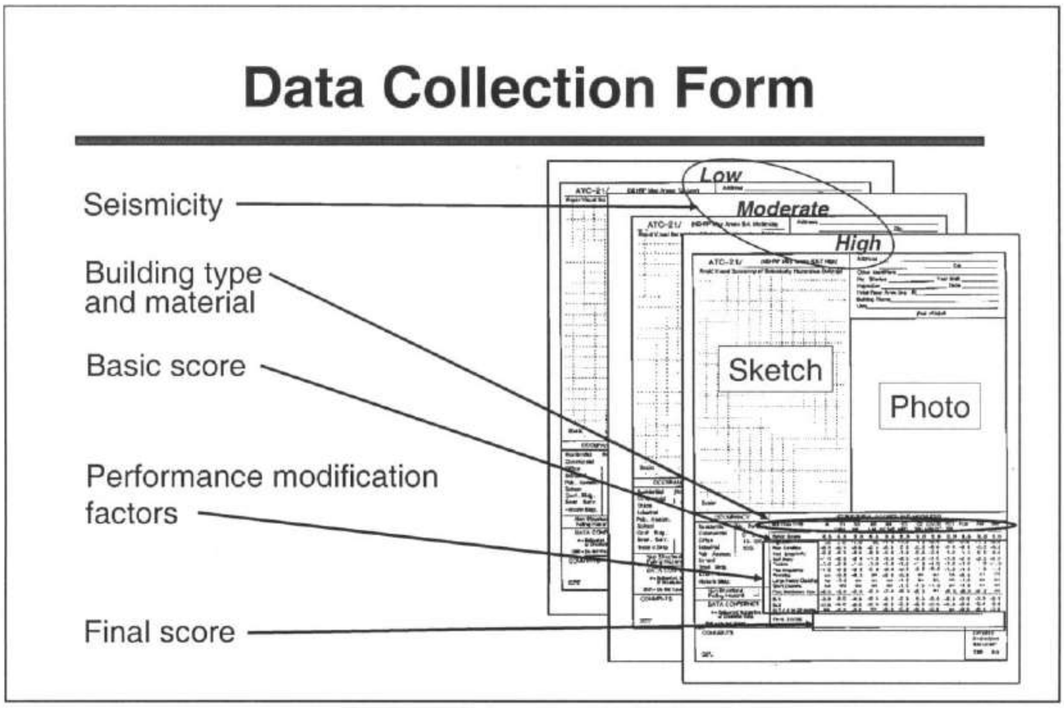


Figure 2a: Data Collection Form for different seismicity (low, moderate and high),used to record building type and material, the basic score, and relevant performance modification factors and final structural hazard score information of the building address, size, occupancy and non-structural element hazard

Table 1: Basic Structural Score (S_0) for different type of buildings in different seismicity as per FEMA 154, 2002

S. No.	Type of Buildings	Level of Seismicity		
		Low	Medium	High
1.	Light Wood -frame residential and commercial buildings smaller than or equal to 5000 square feet (W_1)	7.0	5.2	4.4
2.	Light Wood -frame buildings larger than 5000 square feet (W_2)	6.0	4.8	3.8
3.	Steel moment- resisting frame buildings (S_1)	4.6	3.6	2.8
4.	Braced steel frame buildings (S_2)	4.8	3.6	3.0
5.	Light metal building (S_3)	4.6	3.8	3.2
6.	Steel frame buildings with cast in place concrete shear walls (S_4)	4.8	3.6	2.8
7.	Steel frame buildings with un-reinforced masonry infill walls (S_5)	5.0	3.6	2.0
8.	Concrete moment resisting frame building (C_1)	4.4	3.0	2.5
9.	Concrete shear wall building (C_2)	4.8	3.6	2.8
10.	Concrete frame building with un-reinforced masonry infill walls (C_3)	4.4	3.2	1.6
11.	Tilt - up buildings (PC_1)	4.4	3.2	2.6
12.	Pre-cast concrete frame building (PC_2)	4.6	3.2	2.4
13.	Reinforced masonry buildings with flexible floor and roof diaphragm (RM_1)	4.8	3.6	2.8
14.	Reinforced masonry buildings with Rigid floor and roof diaphragm (RM_2)	4.6	3.4	2.8
15.	Un-reinforced bearing wall buildings (URM)	4.6	3.4	1.8

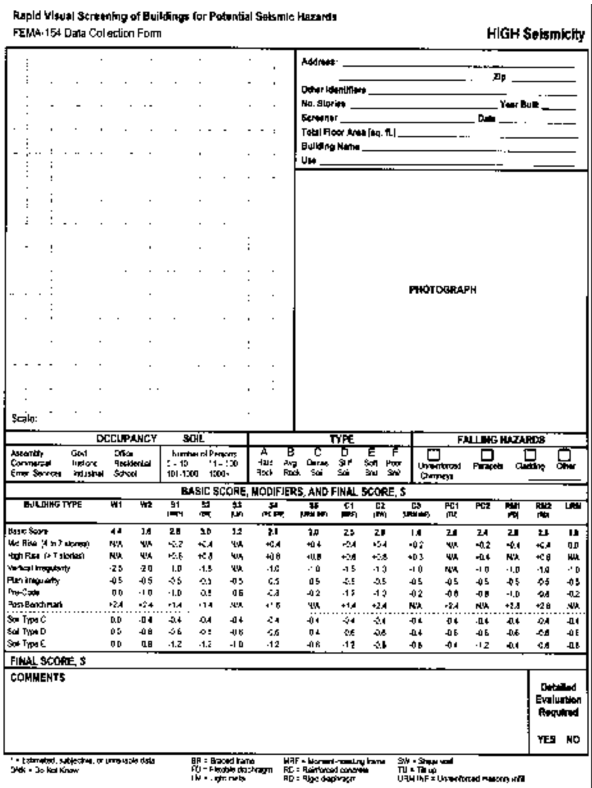


Figure 2b: Sample Data Collection Form for high seismicity for filling the information regarding the building address, size, occupancy, soil type and non-structural element hazard

Rapid Visual Screening of Buildings for Potential Seismic Hazards (FEMA 154)
Quick Reference Guide (for use with Data Collection Form)

1. Model Building Types and Critical Code Adoption and Enforcement Dates

Structural Types		Year Seismic Codes Initially Adopted and Enforced*	Benchmark Year when Codes Improved
W1	Light wood frame, residential or commercial, ≤ 5000 square feet	______	______
W2	Wood frame buildings, > 5000 square feet.	______	______
S1	Steel moment-resisting frame	______	______
S2	Steel braced frame	______	______
S3	Light metal frame	______	______
S4	Steel frame with cast-in-place concrete shear walls	______	______
S5	Steel frame with unreinforced masonry infill	______	______
C1	Concrete moment-resisting frame	______	______
C2	Concrete shear wall	______	______
C3	Concrete frame with unreinforced masonry infill	______	______
PC1	Tilt-up construction	______	______
PC2	Precast concrete frame	______	______
RM1	Reinforced masonry with flexible floor and roof diaphragms	______	______
RM2	Reinforced masonry with rigid diaphragms	______	______
URM	Unreinforced masonry bearing-wall buildings	______	______

*Not applicable in regions of low seismicity

2. Anchorage of Heavy Cladding
Year in which seismic anchorage requirements were adopted: ______

3. Occupancy Loads

Use	Square Feet, Per Person	Use	Square Feet, Per Person
Assembly	varies, 10 minimum	Industrial	200-500
Commercial	50-200	Office	100-200
Emergency Services	100	Residential	100-300
Government	100-200	School	50-100

4. Score Modifier Definitions

Mid-Rise:	4 to 7 stories
High-Rise:	8 or more stories
Vertical Irregularity:	Steps in elevation view; inclined walls; building on hill; soft story (e.g., house over garage); building with short columns; unbraced cripple walls.
Plan Irregularity	Buildings with re-entrant corners (L, T, U, E, + or other irregular building plan); buildings with good lateral resistance in one direction but not in the other direction; eccentric stiffness in plan, (e.g. corner building, or wedge-shaped building, with one or two solid walls and all other walls open).
Pre-Code:	Building designed and constructed prior to the year in which seismic codes were first adopted and enforced in the jurisdiction; use years specified above in Item 1; default is 1941, except for PC1, which is 1973.
Post-Benchmark:	Building designed and constructed after significant improvements in seismic code requirements (e.g., ductile detailing) were adopted and enforced; the benchmark year when codes improved may be different for each building type and jurisdiction; use years specified above in Item 1 (see Table 2-2 of FEMA 154 *Handbook* for additional information).
Soil Type C:	Soft rock or very dense soil; S-wave velocity: 1200 – 2500 ft/s; blow count > 50; or undrained shear strength > 2000 psf.
Soil Type D:	Stiff soil; S-wave velocity: 600 – 1200 ft/s; blow count: 15 – 50; or undrained shear strength: 1000 – 2000 psf.
Soil Type E:	Soft soil; S-wave velocity < 600 ft/s; or more than 100 ft of soil with plasticity Index > 20, water content > 40%, and undrained shear strength < 500 psf.

Figure 2c: Details of Data Collection Form (Reference Guide)

Building Types

Identifier	General Description	Abbreviation
W	Wood Building	
S1	Steel Moment-Resisting Frame	(MRF)
S2	Steel Braced Frame	(BR)
S3	Light Metal Building	(LM)
S4	Steel Frame with Concrete Shear Walls	(RC SW)
C1	Concrete Moment-Resisting Frame	(MRF)
C2	Concrete Shear Wall	(SW)
C3/S5	Concrete or Steel Frame with URM Infill Walls	(URM INF)
PC1	Concrete Tilt-up Wall	(TU)
PC2	Concrete Precast Frame	
RM	Reinforced Masonry	
URM	Unreinforced Masonry	

Figure 3: Building Types identified for RVS survey, the single most important factor in the procedure for determining earthquake resistance

Performance Modification Factors

- High Rise
- Poor Condition
- Vertical Irregularity
- Soft Story
- Torsion
- Plan Irregularity
- Pounding
- Large Heavy Cladding
- Short Columns
- Post Benchmark Year

Soil Profiles:

- SL2
- SL3
- SL3 and 8 to 20 stories

Figure 4: Building attributes which affect the structural performance of lateral load resisting system or building types as mentioned above

Structural Scores and Modifiers

Performance modification factors

Building type

STRUCTURAL SCORES AND MODIFIERS

BUILDING TYPE	W	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	C1 (MRF)	C2 (SW)	C3/S5 (URM INF)	PC1 (TU)	PC2	RM	URM
Basic Score	**8.5**	**3.5**	**2.5**	**6.5**	**4.5**	**4.0**	**4.0**	**3.0**	**3.5**	**2.5**	**4.0**	**2.5**
High Rise	N/A	0	0	N/A	-0.5	-0.5	-0.5	-0.5	N/A	-1.0	-1.5	-0.5
Poor Condition	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Vert. Irregularity	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-0.5	-1.0	-1.0	-1.0	-0.5	-1.0
Soft Story	-1.0	-2.0	-2.0	-1.0	-2.0	-2.0	-2.0	-1.0	-1.0	-1.0	-2.0	-1.0
Torsion	-1.0	-2.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-1.0	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-1.0	-1.0	-1.0	-1.0
Pounding	N/A	-0.5	-0.5	N/A	-0.5	-0.5	N/A	N/A	N/A	-0.5	N/A	N/A
Large Heavy Cladding	N/A	-2.0	N/A	N/A	N/A	-1.0	N/A	N/A	N/A	-1.0	N/A	N/A
Short Columns	N/A	N/A	N/A	N/A	N/A	-1.0	-1.0	-1.0	N/A	-1.0	N/A	N/A
Post Benchmark Year	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	+2.0	N/A	+2.0	+2.0	+2.0	N/A
SL2	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3	-0.3
SL3	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6	-0.6
SL3 & 8 to 20 stories	N/A	-0.8	-0.8	N/A	-0.8	-0.8	-0.8	-0.8	N/A	-0.8	-0.8	-0.8
FINAL SCORE												

Figure 5: PMF depend on building type for each performance attribute. A high score means the impact of that particular attribute on the performance or severity of damage of the building type is high; vice-versa

Table 2: PMF for different type of buildings in different seismicity as per FEMA 154, 2002

Low seismicity

S. No.	Building attribute	C_1	C_2	C_3	RM_1	RM_2	RM_3
1.	Mid Rise (4 to 7 stories)	+ 0.4	+ 0.2	- 0.4	- 0.4	- 0.2	- 0.6
2.	High Rise (> 7 stories)	+ 1.0	0.0	- 0.4	N/A	0.0	N/A
3.	Vertical irregularity	- 1.5	- 2.0	- 2.0	- 2.0	- 1.5	- 1.5
4.	Plan irregularity	- 0.8	- 0.8	- 0.8	- 0.8	- 0.8	- 0.8
5.	Pre-code	N/A	N/A	N/A	N/A	N/A	N/A
6.	Post-Bench Mark	+ 0.6	+ 0.4	N/A	+ 0.2	+ 0.4	+ 0.4
7.	Soil Type C	- 0.6	- 0.4	- 0.4	- 0.4	- 0.2	- 0.4
8.	Soil Type D	- 1.4	- 0.8	- 0.8	- 0.8	- 0.8	- 0.8
9.	Soil Type E	- 2.0	- 2.0	- 2.0	- 1.4	- 1.6	- 1.6

Moderate Seismicity

S. No.	Building attribute	C_1	C_2	C_3	RM_1	RM_2	URM
1.	Mid Rise (4 to 7 stories)	+ 0.2	+ 0.4	+ 0.2	+ 0.4	+ 0.4	- 0.4
2.	High Rise (> 7 stories)	+ 0.5	+ 0.8	+ 0.4	N/A	+ 0.6	N/A
3.	Vertical irregularity	- 2.0	- 2.0	- 2.0	- 2.0	- 1.5	- 1.5
4.	Plan irregularity	- 0.5	- 0.5	- 0.5	- 0.5	- 0.5	- 0.5
5.	Pre-code	- 1.0	- 0.4	- 1.0	- 0.4	- 0.4	- 0.4
6.	Post-Bench Mark	+ 1.2	+ 1.6	N/A	+ 2.0	+ 1.8	N/A
7.	Soil Type C	- 0.6	- 0.8	- 0.6	- 0.8	- 0.6	- 0.4
8.	Soil Type D	- 1.0	- 1.2	- 1.0	- 1.2	- 1.2	- 0.8
9.	Soil Type E	- 1.6	- 1.6	- 1.6	- 1.6	- 1.6	- 1.6

High Seismicity

S. No.	Building attribute	C_1	C_2	C_3	RM_1	RM_2	URM
1.	Mid Rise (4 to 7 stories)	+ 0.4	+ 0.4	+ 0.2	+ 0.4	+ 0.4	0.0
2.	High Rise (> 7 stories)	+ 0.6	+ 0.8	+ 0.3	N/A	+ 0.6	N/A
3.	Vertical irregularity	- 1.6	- 1.0	- 1.0	- 1.0	- 1.0	- 1.0
4.	Plan irregularity	- 0.5	- 0.5	- 0.5	- 0.5	- 0.5	- 0.5
5.	Pre-code	- 1.2	- 1.0	- 0.2	- 1.0	- 0.8	- 0.2
6.	Post-Bench Mark	+ 1.4	+ 2.4	N/A	+ 2.8	+ 2.6	N/A
7.	Soil Type C	- 0.4	- 0.4	- 0.4	- 0.4	- 0.4	- 0.4
8.	Soil Type D	- 0.6	- 0.6	- 0.4	- 0.6	- 0.6	- 0.6
9.	Soil Type E	- 1.2	- 0.8	- 0.8	- 0.4	- 0.6	- 0.8

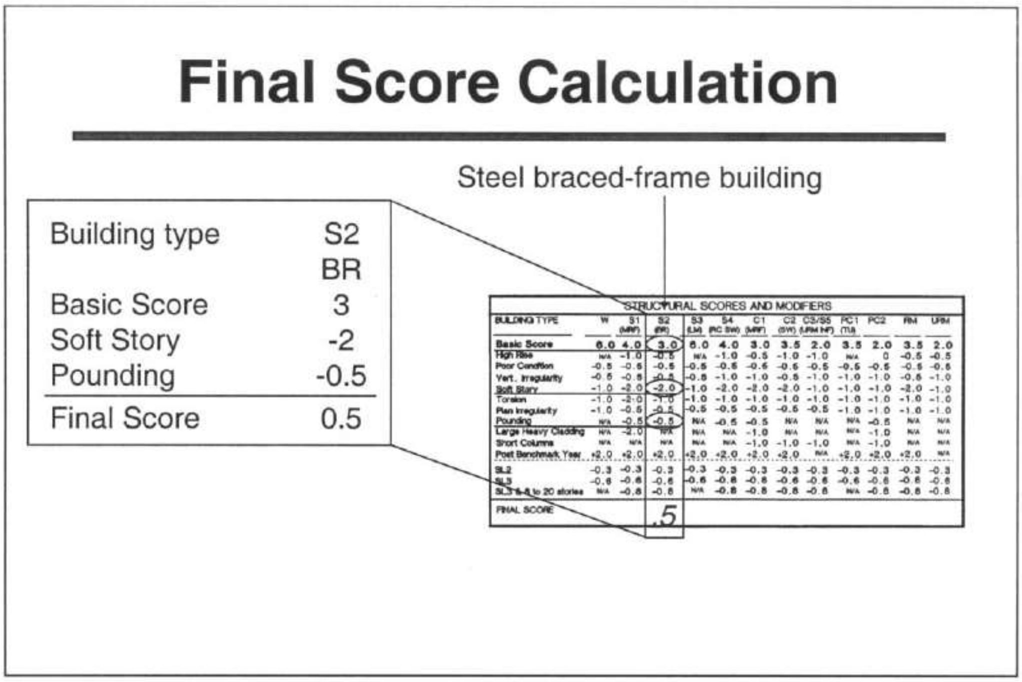


Figure 6: Estimation of Final Score which depend on Basic Score and the addition and subtraction of Performance modifiers. A high Final Score is good. A low score denote probable seismic performance, and that building should be reviewed by the professional engineer experience in seismic design

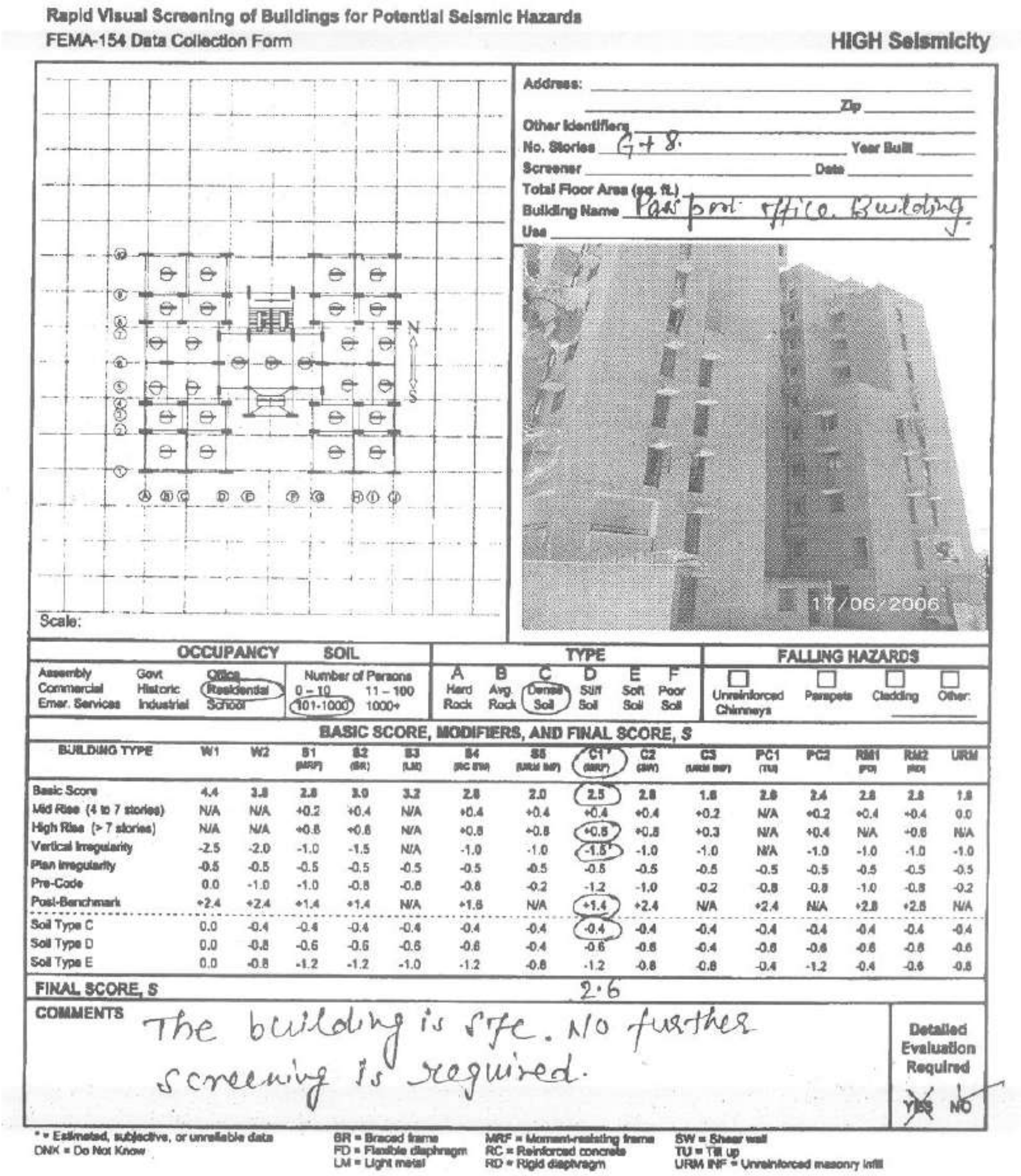
Further Reading

1. FEMA 154 (2002). "Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook", Applied Technology Council, Redwood City, California.
2. FEMA 155 (2002). "Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentations", Applied Technology Council, Redwood City, California.
3. BIS (2002). "Criteria for Earthquake Resistant Design of Structures" IS 1893 (Part 1): 2002, Manak Bhawan, 9 Bhhadur Shah Zafar Marg, New Delhi.
4. Agarwal, P. (1994). "Assessment of Seismic Vulnerabity of RC Buildings", M.Tech Thesis, University of Rorkee, Roorkee.
5. ATC – 21 - T (1996). "Rapid Visual Screening of Buildings for Potential Seismic Hazards Training Manual", Applied Technology Council, Redwood Cot, California.

Example on RVS Method

A RC frame residential building in a high seismic zone, eight storey, dense soil, soft storey with possibility of pounding (vertical irregularity), year of construction 2004.

Basic Structural Score (S_0) corresponds to type C1 (MRF) = 2.5
Prime modification factors (PMF) = +0.6 (High Rise), -0.4 (dense soil),-1 (vertical irregularities) + 1.4 (Post - Benchmark)
Final Structural Score (S) = 2.5 + 0.6 -0.4 -1.5+1.4 = 2.6 > 2.0, No need to evaluate further.


Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA-154 Data Collection Form

HIGH Seismicity

Address:
Zip
Other Identifiers
No. Stories G + 8.
Year Built
Screener
Date
Total Floor Area (sq. ft.)
Building Name Pass port office. Building
Use

Scale:

17/06/2006

OCCUPANCY			SOIL		TYPE						FALLING HAZARDS			
Assembly Commercial Emer. Services	Govt Historic Industrial	Office Residential School	Number of Persons 0 – 10 101-1000	11 – 100 1000+	A Hard Rock	B Avg. Rock	C Dense Soil	D Stiff Soil	E Soft Soil	F Poor Soil	Unreinforced Chimneys	Parapets	Cladding	Other:

BASIC SCORE, MODIFIERS, AND FINAL SCORE, S

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	4.4	3.8	2.8	3.0	3.2	2.8	2.0	2.5	2.8	1.6	2.6	2.4	2.8	2.8	1.8
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.8	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8
FINAL SCORE, S								2·6							

COMMENTS The building is safe. No further screening is required.

Detailed Evaluation Required
YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know
BR = Braced frame
FD = Flexible diaphragm
LM = Light metal
MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm
SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

4.2 How to evaluate the seismic resistance of an existing RC building by simplified ATC – 14 methodology?

Problem statement: The greatest risk to human lives and property in the event of an earthquake is due to damage and collapse of existing buildings. In spite of this, there is presently no well known procedure for the seismic evaluation of existing buildings. The ATC-14 methodology is intended to this need, which leads not only to conclusions concerning the adequacy of the structure for a given event, but also identifies the structure's weakness and, therefore, area of needed rehabilitation. The methodology assumes that the successful performance of the buildings depends on the base shear strength of the main structural elements, the connection strength and ductility of the connections, the building configuration, the material type, and the interconnection of the structural parts. This example describes the evaluation procedure for reinforced concrete buildings based on ATC-14 methodology, modified as per Indian Codal Provisions, and its application to an existing three storey reinforced concrete building.

INTRODUCTION

There are many buildings that have primary structural system not meeting the current seismic requirements. Such buildings might suffer extensive damage or even collapse if shaken by a severe ground motion causing injury to the occupants or people living in the vicinity. Therefore for earthquake hazard mitigation it is necessary to identify these buildings and to evaluate their seismic capacity to resist the future earthquakes. To this purpose ATC-14(1) developed a methodology on the basis of past performance of buildings during previous earthquakes. The methodology assumes that a "hazardous building" endangers human lives in an earthquake if one or more of the following events occur. Such as the entire building collapses, portion of the building collapses, components of the building fail and fall, exit and entry routes are blocked. Hence, a major portion of the methodology is dedicated to direct the evaluating engineer on how to determine if there are any weak links in the structure that could precipitate structural or component failure. Potential weak links have been identified from detailed review of building performance data from past earthquakes.

BASIS OF ATC-14 METHODOLOGY

In 1987, ATC-14 project developed a practical methodology to determine potential earthquake hazards and identify buildings or building components that are risky to human lives. To develop the building seismic evaluation methodology the main literature surveyed by ATC –committee are (1) earthquake damage reports (2) existing and proposed Code Provisions (3) previously developed seismic evaluation methodologies (4) reports on analytical and experimental research, including a special focus on the various materials utilized in building structural system; and (5) test methods. The following are the main features of this methodology.

Data Collection: Methodology begins with the data collection procedures which are required together with the information necessary to classify the building and perform the evaluations. It includes the following items:

(i) Contract documents, such as construction drawings, specifications, soil reports, and calculation.

(ii) Field surveys of the structure's existing condition.

Building Classification: The evaluation procedure of the methodology depends on the type of model buildings which are classified in 15 categories (ATC 1987). These categories are based on material or type of construction employed in the principal gravity and lateral force resisting elements.

Evaluation Procedure: The evaluation procedure for each model building consists of a collection of statements with related concern. Each statement relates to a vulnerable area in the structural system that requires specific consideration and its concern explains why the statements are written. The evaluation statements that have a positive or "**true**" response to a statement imply that the building is adequate in that area. If a building then passes all the related statements with true responses, it can be passed without further evaluation. For statements that are "**false**" additional evaluation is necessary. It includes the following items:

1. A description of the performance characteristics exhibited by structure of this type during past earthquakes of severe intensity.
2. Evaluation statements of buildings which are based on their performance characteristics.

A detailed description of evaluation procedure for reinforced concrete buildings, suitably modified as per Indian Code, is given below.

SEISMIC EVALUATION OF MOMENT RESISTING REINFORCED CONCRETE BUILDING

The evaluation of any building is a difficult task which requires a wide knowledge about the seismic behavior of buildings. The procedure outlined below is to be treated as a guide to that decision making process and not as the absolute method of evaluation.

Performance Characteristics

The moment resisting reinforced concrete frame buildings are typically more flexible than shear wall buildings. This low stiffness can result in large inter-storey drifts that may lead to extensive nonstructural damage. The following list shows specific performance characteristics that these buildings may exhibit during the past earthquake.

- Large tie spacing in column can lead to a lack of confinement for the concrete core and/ or shear failures.
- Insufficient column lap lengths can cause concrete to spall.
- Location of inadequate splices for all column bars at the same section can lead to column failure.
- If the column shear strength is insufficient to develop the full moment hinge capacity, the column can exhibit a brittle shear failure.
- Insufficient anchorage of shear tie reinforcing in column cores can prevent the column from developing its full shear capacity.
- Lack of continuous beam reinforcement can cause hinge formation during load reversals.
- Inadequate reinforcing of beam-column joints or location of beam bar splices at columns can lead to joint failures.
- Foundation dowels that are insufficient to develop the capacity of the column steel above can lead to local column distress.
- Use of bent-up longitudinal reinforcing in beams as shear reinforcement can result in shear failure during load reversal.
- The relatively low stiffness of the frames can lead to excessive interstorey drifts. These large drifts can cause damage to nonstructural items such as partitions, window, etc.
- Pounding between immediately adjacent structures of different heights can occur. This could lead to column distress and possibly local collapse where the floors of adjacent building are not at the same elevation.
- Building with substantial plan irregularities, such as T, L, U, or cruciform configurations, may generate large torsional effects. Depending on the rigidities of the lateral system, different wings of the structure may vibrate independently, which could lead to a concentration of damage at the junctures (i.e., reentrant corners) if separation joints or special reinforcements have not been provided.
- Buildings with abrupt change in lateral resistance have often performed poorly in past earthquakes. Significant vertical strength discontinuities tend to concentrate damage in the “soft” stories.
- Improper detailing in beams and columns reduce the toughness and ductility, therefore these are not capable to undergo extensive inelastic deformations and dissipating seismic energy in a stable manner.

Evaluation Statements

Statement 1: Rapid Evaluation of Reinforced Concrete Column
Related Concern: Reinforced concrete frame buildings have generally failed in past earthquake due to inadequate column shear capacity. So, a quick estimation of the shear stress in the concrete frame column should be evaluated in the region of high or moderate seismicity.

Statement 2: Rapid estimation of Storey Drift
Related Concern: Drift must be limited, even if the structure can tolerate more because of its effect on nonstructural components, particularly partition, exterior skin and ceiling elements, and on the comfort of occupants.

Evaluation of Material

Statement 3: For all buildings which are more than 3 storeys in height, the minimum grade of concrete shall preferably be M20.
Related Concern: The concrete strength below M20 may not have the requisite strength in bond or shear to take full advantage of the design provision.

Statement 4 Steel reinforcement upto grade Fe415 shall be used.
Related Concern For reinforcement, firstly the provisions, of adequate ductility and secondly, of an upper limit on the yield stress or characteristics strength, are essential. General practice is to limit the yield stress of reinforcement to 415 N/mm^2 i.e HYSD bars.

Evaluation of Structural Components

Statement 5: There are no in-fills of concrete or masonry placed in the concrete frames that are not isolated from the structural elements.
Related Concern: In-fill walls used for partitions or walls around the stair, that are not adequately isolated, will alter the seismic response of the structure. Random stiffening of a frame structure by masonry infill is a common instigator of damage and failure. The mechanism is that the earthquake forces are attracted to the areas of greatest stiffness, and if these are not designed to accommodate forces, they are prone to failure.

Statement 6: The lateral force resisting elements form a well-distributed and balanced system that is not subjected to significant torsion.
Related Concern: Plan irregularities may cause torsion or excessive lateral deflections that may result in permanent set or even partial collapse.

Statement 7: There are no significant strength discontinuities or soft storey in any of the vertical lateral force resisting elements.

Related Concern: The condition is most critical when it occurs at the first storey, because the loads are generally greatest at this level. The essential characteristics of a soft storey consist of a discontinuity of strength or stiffness, which occurs at the second storey connections. This discontinuity caused due to lesser strength, or increased flexibility, in the first storey structure results in extreme deflection in the first storey which, in turn, results in a concentration of forces/stresses at the second storey connections.

Statement 8: There are no significant vertical irregularities caused by either geometric or mass irregularities.
Related Concern: Torsional forces will be introduced into the structure, resulting in great complexity of analysis and behavior.

Statement 9: All of the frames continue to the building base.
Related Concern: All of the frames carry shear and overturning forces. Any frames that do not continue to the foundation must deliver their shear and overturning to the other structural elements. Unless there are supplementary elements specifically detailed to take these loads, these elements may not have sufficient capacity.

Statement 10: The moment capacity of the columns appears to be greater than that of the beams.
Related Concern: This is based on the reasoning that as beams start to fail, they will move from elastic to inelastic behaviour and start to deform permanently. This action will dissipate and absorb some of the earthquake forces. Conversely, if the column fails first and begins to deform and buckle, major vertical compressive loads may quickly lead to total collapse.

Statement 11: There is no pounding
Related Concern: The possibility of pounding is a function of the vertical deflection or drift of adjoining buildings (or parts of a building).

Evaluation of Structural Details

Flexural Members

Statement 12: Limitation on flexural reinforcement ratio.

$$\rho_{min} = \begin{bmatrix} 0.24\sqrt{f_{ck}/f_y} \\ \text{two continuous bars at both top and bottom of the member} \end{bmatrix}$$

$$\rho_{max} = 0.025$$

Related Concern: The limiting ratio of 0.025 is based mainly on consideration of steel congestion and also on limiting shear stresses in beams of typical proportions. From a practical stand point, low steel ratios should be used whenever possible. The requirement of at least two bars, top and bottom, is intended to insure integrity of the member under reversed loading.

Statement 13: Restriction on lap splices - Lap splice shall not be used (i) within joint (ii) within 2d from face of support, where d is effective depth of beam (iii) at locations of potential plastic hinging.
Related Concern: Lap splices of flexural reinforcement are not allowed in regions of potential plastic hinging, since such splices are not considered to be reliable under reversed inelastic cycle of deformation.

Columns

Statement 14: Restriction on use of Lap splices - Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice; hoops shall be provided over the entire splice length at spacing not exceeding 150 mm c/c. Not more than 50% of the reinforcement shall be spliced at one section.
Related Concern: Splices of inadequate length may lead to column distress and even failure.

Evaluation of Foundation

Statement 15: All longitudinal column steel is dowelled into the foundation.
Related Concern: The lack of sufficient dowels create a weak plane that may not have adequate shear or torsion capacity, essential for overturning forces.

AN EXAMPLE OF SEISMIC EVALUATION OF RC BUILDING

A three storey reinforced concrete building, situated in Zone V ($A_h = 0.36$) ,is examined with this methodology. The selected building consists of a moment resisting frame. The structural plan of the typical floor is shown in Figure 1. A typical frame in transverse direction is shown in Figure 2. The detailing of this frame is shown in Figure 3 and Figure 4. The evaluation statements are evaluated after a visit to the site and with the help of original structural drawings and their results are summarized in evaluation checklist (Table 1.)

Table 1: Seismic Evaluation of 3 storied Reinforced Concrete Building

No.	Statement	True/False
	Evaluation of concrete column*	True
	Rapid estimation of storey drift*	True
	The sample calculations of these two statements are given in Appendix I.	
	Evaluation of Material	
	No significance deterioration of reinforcing bar	True
	Minimum grade of concrete shall preferably M20	True
	Steel of grade Fe415 or less shall be used	True
	Evaluation of Structural Components	
	No infill	True
	No torsion	True
	No vertical strength discontinuities	True
	No vertical mass or geometric discontinuities	True
	Frame continuous to the base	True
	Strong column-weak beams	False
	No pounding of adjacent structure	True
	No eccentricity between column and beam	True
	Evaluation of Structural Details	
	Flexural Members	
	Limitation of sectional dimension	
	The member shall preferably have a width to depth ratio of more than 0.3.	True
	The width of the member shall not be less than 200 mm.	True
	The depth D of the member shall preferably be not more than $\frac{1}{4}$ of the clear span.	True
	Longitudinal Reinforcement	
	The top as well as bottom reinforcement shall consist of at least two bars throughout the member length.	True
	The tension steel ratio on any face, at any section, shall not be less than $\rho_{min} = 0.24\sqrt{f_{ck}}/f_y$; where f_{ck} and f_y are in MPA.	True
	The maximum steel ratio on any face at any section shall not exceed $p_{max} = 0.025$.	True
	Moment capacity requirement at beam ends $M^+_y > 0.50\ M^-_y$	True
	Moment capacity requirement at beam span M^+_y or $M^-_y > 0.25$	True
	Restriction of Lap Splice (i) within joint (ii) within 2d from face of support (iii) location of potential plastic hinge region	False

In an external joint, both the top and the bottom bars of the beam shall be provided with anchorage length, beyond the inner face of the column, equal to the development length in tension plus 10 times the bar diameter minus the allowance for 90 degrees bend(s). In an internal joint, both face bars of the beam shall be taken continuously through the column. The longitudinal bars shall be spliced, only if hoops are provided over the entire splice length, at spacing not exceeding 150 mm. The lap length shall not be less than the bar development length in tension. Lap splices shall not be provided (a) within a joint, (b) within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces. Not more than 50 percent of the bars shall be spliced at one section.

Web Reinforcement

Web reinforcement shall consist of vertical hoops. A vertical hoop is a closed stirrup having a 135° hook with a 10-diameter extension (but not < 75 mm) at each end that is embedded in the confined core. In compelling circumstances, it may also be made up of two pieces of reinforcement; a U stirrup with a 135° hook and a 10-diameter extension (but not < 75 mm) at each end, embedded in the confined core and cross tie. A crosstie is a bar having a 135° hook with a 10 diameter extension (but not < 75 mm) at each end. The hooks shall engage peripheral longitudinal bars. False

The minimum diameter of the bar forming a hoop shall be 6 mm. However, in beams with clear span exceeding 5 m, the minimum bar diameter shall be 8 mm. True

The shear force to be resisted by the vertical hoops shall be the maximum of: a) calculated factored shear force as per analysis, and b) shear force due to formation of plastic hinges at both ends of the beams plus the factored gravity load on the span. False

The spacing of hoops over a length of 2d at either end of a beam shall not exceed (a) d/4, and (b) 8 times the diameter of the smallest longitudinal bar; however it need not be less than 100 mm. The first hoop shall be at a distance not exceeding 50 mm from the joint face. Vertical hoops at the same spacing as above shall also be provided over a length equal to 2d on either side of a section where flexural yielding may occur under the effect of earthquake forces. Elsewhere the beam shall have vertical hoops at a spacing not exceeding d/2. False

Columns and Frame Members subjected to Bending and Axial load

Limitation on sectional dimensional

The minimum dimension of the member shall not be less than 200 mm. However, in frames, which have beams with center-to-center span exceeding 5 m or columns of unsupported length exceeding 4 m, the shortest dimension of the column shall not be less than 300 mm.
The ratio of the shortest cross sectional dimension to the perpendicular dimension shall preferably be not less than 0.4.

Longitudinal Reinforcement

Lap splices shall be provided only in the central half of the member length. It should be proportioned as a tension splice. Hoops shall be provided over the entire splice length at spacing not exceeding 150 mm center to center. Not more than 50 percent of the bars shall be spliced at one section. False

Any area of a column that extends more than 100 mm beyond the confined core due to architectural requirements shall be detailed in the following manner. In case the contribution of this area to strength has been considered, then it will have minimum longitudinal and transverse reinforcement as per this code. However if this area has been treated as non-structural, the minimum reinforcement requirements shall be governed by IS 456:1978 provisions minimum longitudinal and transverse reinforcement, as per IS 456:1978. True

Transverse Reinforcement

Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135° hook with 10-diameter extension (but not <75 mm) at each end that is embedded in the confined core. False

The parallel legs of rectangular hoop shall be spaced not more than 300 mm center to center. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided. Alternatively, a pair of overlapping hoops may be provided within the column. The hooks shall engage peripheral longitudinal bars.

The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided as 7.4. False

The design shear force for columns shall be the maximum of: a) calculated factored shear force shear forces as per analysis and b) a factored shear force given by $V_U = 1.4\left[\dfrac{M^{bL}{}_{u,\lim} + M^{bR}{}_{u,\lim}}{h_{st}}\right]$ False

Where $M^{bL}{}_{u,lim}$ and $M^{bR}{}_{u,lim}$ are moment of resistance, of opposite sign, of beams framing into the column from opposite faces and h_{st} is the storey height. The beam moment capacity is to be calculated as per IS 456:1978.

Special Confining Reinforcement

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations.

Special confining reinforcement shall be provided over a length l_o from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces. The length 'l_o' shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450 mm. False

When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat. False

When the calculated point of contra-flexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column. False

Columns supporting reactions from discontinued stiff members, such as walls, shall be provided with special confining reinforcement over their full height. This reinforcement shall also be placed above the discontinuity for at least the NA development length of the largest longitudinal bar in the column. Where the column is supported on a wall, this reinforcement shall be provided over the full height of the column; it shall also be provided below the discontinuity for the same development length.

Special confining reinforcement shall be provided over the full height of a column, which has significant variation in stiffness along its height. This variation in stiffness may result due to the presence of bracing, a mezzanine floor or a R.C.C. wall on either side of the column that extends only over a part of the column height. NA

The spacing of hoops used as special confining reinforcement shall not exceed ¼ of minimum member dimension but need not be less than 75 mm nor more than 100 mm. False

The area of cross section, A_{sh}, of the bar forming circular hoops or spiral, to be used as special confining reinforcement shall not be less than NA

$A_{sh} = 0.09\, S D_k \frac{f_{ck}}{f_y}\left\{\frac{A_g}{A_k} - 1\right\}$ Where A_{sh} = area of the bar cross section , S = pitch of spiral or spacing of hoops , D_k = diameter of core measured to the outside of the spiral or hoop , f_{ck} = characteristic compressive strength of concrete cube, f_y = yield stress of steel (of circular hoop or spiral) , A_g = gross area of the column cross section and , A_k = area of concrete core = $\pi D_k^2/4$

The area of cross section, A_{sh} of the bar forming rectangular hoop, to be used as special confining reinforcement shall not be less than False

$A_{sh} = 0.18\, S\, h \frac{f_{ck}}{f_y}\left\{\frac{A_g}{A_k} - 1\right\}$, Where, h = longer dimension of the rectangular confining hoop measured to its outer face. It shall not exceed 300 mm and A_k = area of confined concrete core in the rectangular hoop measured to its outside dimensions.

Joints of Frames

The special confining reinforcement as required at the end of column shall be provided through the joint as well, unless the joint is confined as specified by 8.2 False

A joint, which has beams framing into all vertical faces of it and where each beam width is at least ¾ of the column width, may be provided with half the special confining reinforcement required at the end of the column. The spacing of hoops shall not exceed 150 mm. False

CONCLUSIONS

The results described herein are summarized as follows (1) The building is not ductile enough to resist the earthquake force. (2) The structural deficiencies such as lack of shear reinforcement and confinement steel are observed in columns of the building. (3) The beams of the building are stronger than columns which violates the condition “strong column-weak beam” for earthquake resistance design of buildings. (4) The storey drift in the building is within permissible limit.

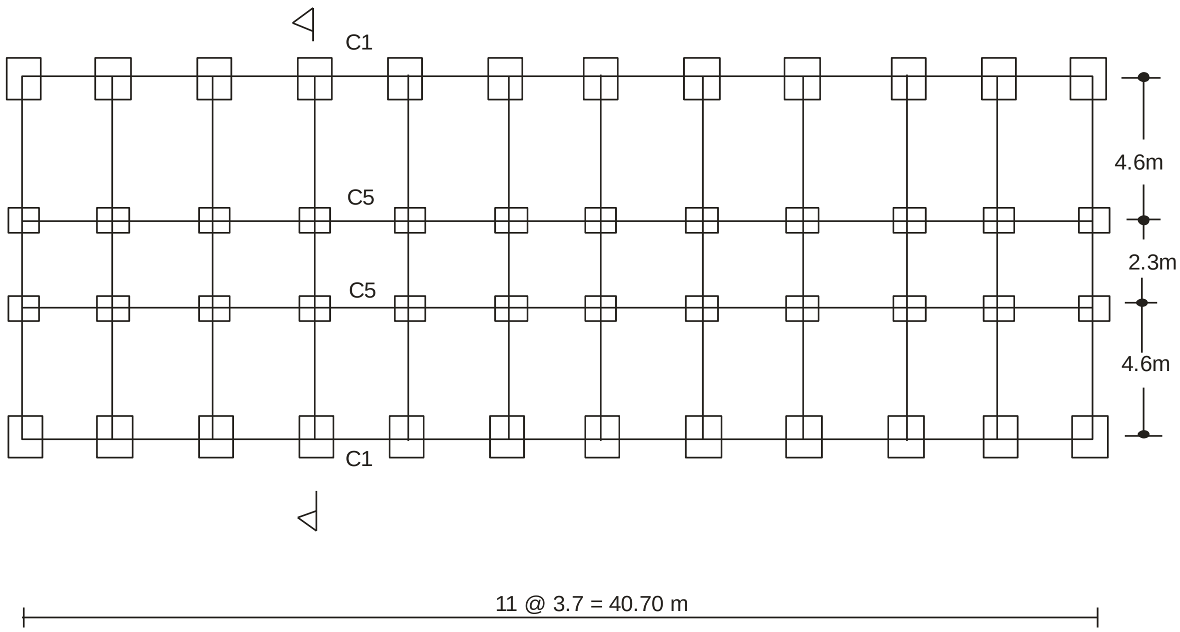


Fig.1- TYPICAL PLAN OF BUILDING

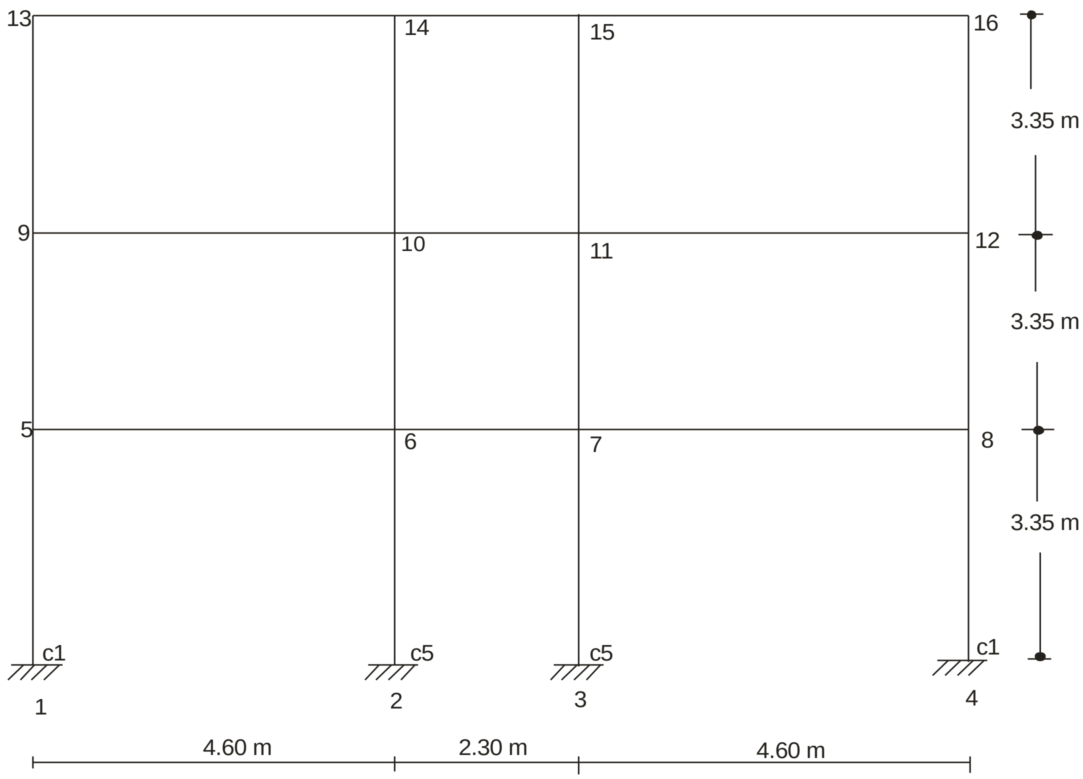


FIG.2 - FRAMING ELEVATION-GRID-4

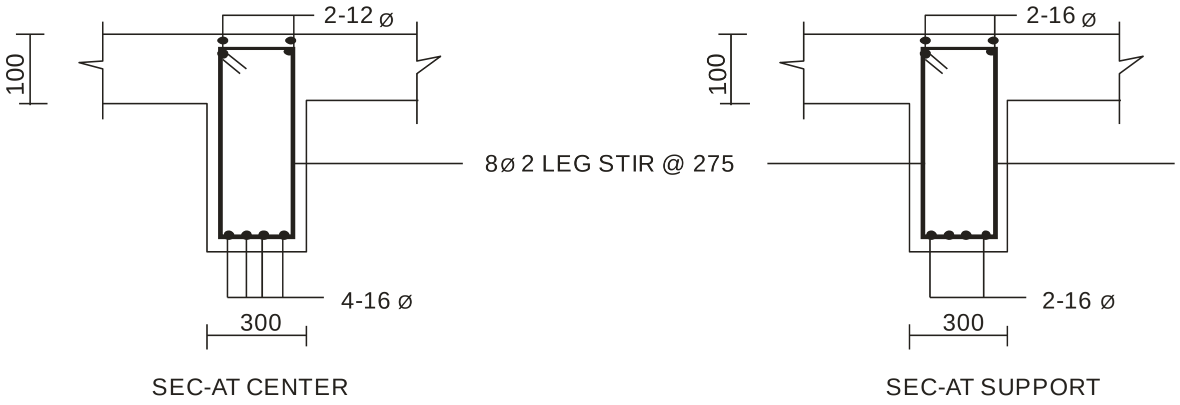


FIG.3 DETALING OF BEAM (For 1st And lind floor)

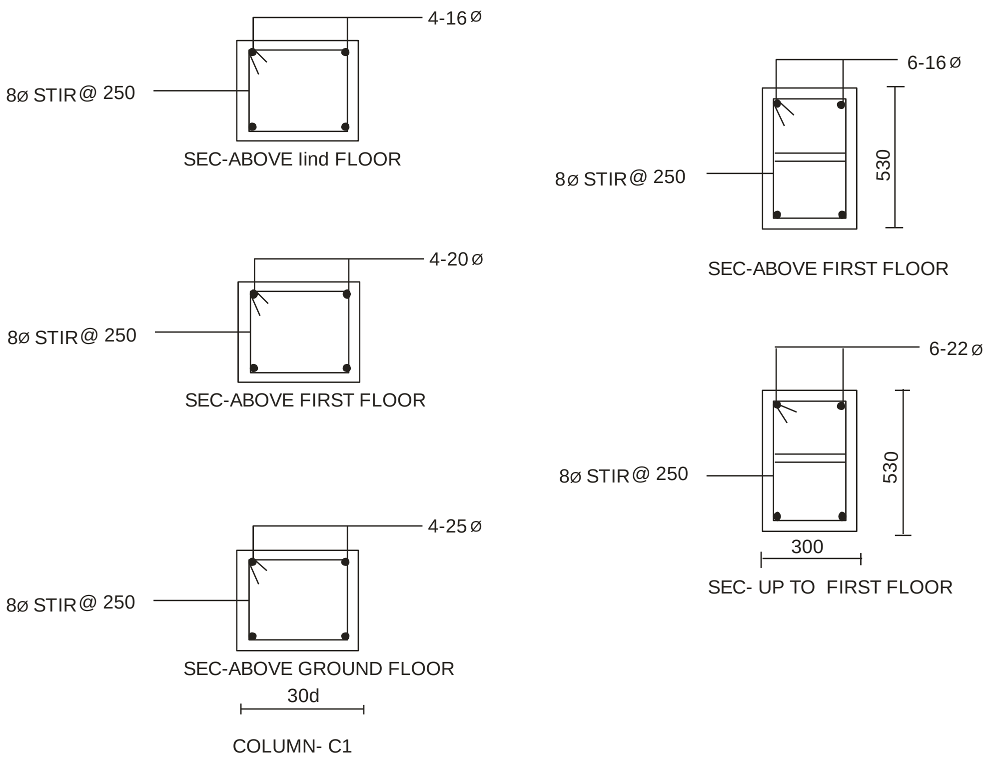


Figure 4: Detailing of column c1 and c5

Appendix 1: Rapid Evaluation of Concrete Column and Estimation of Storey drift

(1) Determine Seismic weight at various floors

Assume;

The dead load per unit area of floor is estimated to be = $10.25 kN/m^2$

The normal line load = $4 kN/m^2$

The seismic weight at various floors

$W_1 = W_2 = 11.5 \times 40.70(10.25 + 0.25 \times 4) = 5265 kN$, Assume 25% of live load

$W_3 = 11.5 \times 40.70 \times (10.25) = 4797.50 kN$, Assume no live load considered at roof

Total seismic weight of the building (W) = $5265 \times 2 + 4797.50 = 15327.50 kN$

(2) Determination of Fundamental Period

$T = 0.075h^{0.75} = 0.075 \times 11.5^{0.75} = 0.468$ sec, where h = total height of building.

(3) Determination of Design Seismic Base Shear Force.

$$V_\beta = A_h W$$

A_h = Design horizontal acceleration spectrum

$$= \frac{Z}{2} \cdot \frac{I}{R} \cdot \frac{S_a}{g} = \frac{0.36}{2} \cdot \frac{1}{5} .2.5 = 0.09$$

Assume

$Z = 0.36$ (*corresponds to Zone V*)

$I = 1.0$ (*for ordinary structures*)

$R = 5.0$ (*SMRF*)

$\frac{S_a}{g} = 2.5$ *for medium soil site* $0.10 \le T \le 0.55$

Therefore,

$V_b = .09 \times 15327.5 = \quad = 1380 kN$

(4) Determination of Lateral force at Each Floor

Level	W_i (kN)	h_i (m)	$W_i h_i^2$ $(kN-m^2)$	$W_i h_i^2 / \sum W h_i^2$	Q_i (kN)	V_i (kN)	A (m^2)	V_{avg} (N/mm^2)
3[roof]	4797.50	10.05	484,560	0.60	828	828	5.976	0.138
2[nd floor]	5265	6.70	263,346	0.32	442	1270	5.976	0.212
1[st floor]	5265	3.35	59,086	0.08	110	1380	5.976	0.232
			$\sum = 806,992.46$		$\sum = 1380$			

(5) Evaluation of storey hear in Column by ATC 14 method

Storey shear $V_j = \frac{n+j}{n+1} \cdot \frac{W_j}{W} . V_b$

Where, n= number of storey = 3, W_j = Total weight of all storey above consideration

W = Total weight of building = 15327.5 kN, V = base shear = 1380 kN

First Storey

$$V_1 = \frac{3+1}{3+1} . \frac{15327.5}{15327.5} .1380\,kN = 1380\,kN$$

Second Storey

$$V_1 = \frac{3+2}{3+1} . \frac{10062.5}{15327.5} .1380\,kN = 1132.50\,kN$$

Third Storey

$$V_1 = \frac{3+3}{3+1} . \frac{4797.5}{15327.5} .1380\,kN = 648\,kN$$

Average shear stress at First storey as per ATC -14

$$V_{avg} = \frac{n_c}{n_c - n_f} . \frac{V_j}{A_c} .$$

Where n_c = Total number of column = 48, n_f = Total number of frame in the direction of loading = 12
A_c = Sum of cross – sectional area of all column in the storey under consideration
= 24 (0.3x0.53)+ 24 (0.3x0.3) = 5.976 m^2 = 9262.82 inch^2

$$V_{avg} = \frac{48}{48-12} . \frac{1380\,x1000}{5.976\,x10^6} . = 0.307\,N/mm^2 < 0.1\sqrt{f'_{ck}} = 0.1\sqrt{20} = 0.447\ N/mm^2$$

(6) Lateral displacement δx and storey drift.

Assume building is a shear building; in this case the stiffness for a column between two consecutive floors is given by $K = \frac{12EI}{L^3}$

Where,

$$h = 3.35\,m, E = 25kN/m^2$$

$$I_{C1} = \frac{1}{12} \times 0.53 \times 0.30^3 = 1.1925\,x10^{-3}\,m^4$$

$$I_{C5} = \frac{1}{12} \times 0.30 \times 0.30^3 = 6.75\,x10^{-4}\,m^4$$

Total stiffness at each floor

$$K_c = 24\left[\frac{12\,x\,25\,x10^3\,x1.1925\,x10^{-3}}{3.35^3} + \frac{12\,x\,25\,x10^3\,x6.75\,x10^{-4}}{3.35^3}\right] = 364.80\,x10^6\ N/m$$

Level	Storey shear (kN)	Storey Stiffness (N/m)	Storey drift $\Delta x\,(mm)$	Total lateral Drift (mm)
3roof	828	364.80×10^6	2.27	9.53
2nd floor	1270	364.80×10^6	3.48	6.05
1st floor	1380	364.80×10^6	3.78	3.78
			$\sum = 9.53$	

The code stipulates that storey drift

$\leq 0.004 \times Storey\ height\quad i.e\, 0.004 \times 3.35 = 13.4\,mm$

Storey drifts by ATC -14 Method

$$\Delta = \frac{K_b + K_c}{K_b K_c} \cdot \frac{h}{12E} \cdot V_{avg}$$

Where:

Δ = Drift ratio, inter-storey displacement divided by storey height

k_b = I/L for the represented beam

k_c = I/L for the represented column

h= storey height (in.)

I = Moment of inertia (in.^{4})

L = beam length from center to center of adjacent column (in.)

E = modulus of elasticity (ksi)

V_c = Shear in the column (kips)

$$K_b = \sum K_b = 24(I/L) = 24\left[\frac{0.30\,x\,0.48^3}{12\,x\,4.6}\right] = 0.00432\ m^3$$

$$K_C = \sum K_c = 12(K_{C1} + K_{c5}) = 12\left[\frac{0.30\,x\,0.53^3}{12\,x\,3.35} + \frac{0.30\,x\,0.30^3}{12\,x\,3.35}\right] = 0.01575 m^3$$

Further Reading

1. ASCE/SEI 31-03 (2003). " Seismic Evaluation of Existing Builings", ASCE, USA.
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4. Agarwal, P., Thakkar, S.K. and Bose, P.R. (1994). "Assessment of Seismic Vulnerabity of Existing RC Buildings", 10th Symposium on Earthquake Engineering, University of Roorkee, Roorkee.
5. ATC 14 (1987). " Evaluating the Seismic Resistance of Existing Buildings", Applied Technology Council, Redwood Cot, California.

4.3 How to carryout post earthquake safety evaluation of RC buildings - Visual Inspection Method

Problem Statement: When earthquake disaster strikes a community, there is an immediate need for damage inspection. People need to be restrictive from entering or using unsafe buildings. Building requiring repairs also need to be identified and repaired for long term safety. This post earthquake evaluation of the buildings is generally carried out through visual examination or inspection. Therefore, the visual inspection is an integral part of post earthquake evaluation which provides a wealth of information that may lead to positive identification of the cause of observed distress and ensures how far the observed damage may prove to be dangerous for the structures. The detailed description of visual examination is presented in ATC-20 and important aspects of the method are summarized below.

POST-EARTHQUAKE SAFETY EVALUATION OF BUILDINGS

The post earthquake evaluation is carried out through visual examination/inspection of a damaged building, from inside and outside. It consists of a number of steps as summarized below;

STEP 1: Survey of the Building from the Outside

- ✓ Perform a walk through visual inspection to become familiar with the structure and collect background documents and information on the design, construction, maintenance, and operation of the structure.
- ✓ Try to locate the vertical and lateral load resisting system
- ✓ Identify and examine the vertical discontinuities in the building. These are regions in a structure where there is a sudden change in stiffness, such as a setback or soft storey. Past experience has shown that damage often concentrates at these places.
- ✓ Identify and examine the structure for irregular configuration in plan. As with vertical discontinuities, damage often tends to concentrate at irregularities in plan. Also look for torsional distress in corner buildings.
- ✓ Look for racking of exterior walls, glass frames, etc., which are symptoms of excessive drift.
- ✓ Examine nonstructural elements, such as curtain walls, parapets, signs, and ornamentation, for damage before entering the building.
- ✓ Look for new fractures in the foundation or exposed lower walls of the building.

STEP 2: Examine the Site for Geotechnical Hazards

- ✓ Examine the site for fissures, bulged ground, and vertical movements.
- ✓ In hillside areas, examine the area for landslide displacement or debris encroaching onto the site.
- ✓ When geotechnical hazards are suspected, the detailed evaluation must be made by a team including a geotechnical engineer or geologist.
- ✓ Since geotechnical hazards can extend in area to include several or more buildings, undamaged buildings in an unstable area may be posted Limited Entry or Unsafe. For example, if an embankment on which an undamaged structure is located has the potential to move further under static loading, the structure should be considered unsafe.

STEP 3: Inspect the Structural System/from Inside the Building

- ✓ In carrying out the survey of the building from inside it is advisable first to look for falling hazards and probability of collapse of structure. Do not enter obviously unsafe buildings. In case of safe building, the surveyor must be equipped with the following equipments; ***Optical magnification*** - allows a more detailed view of local areas of distress; ***Stereomicroscope*** - that allow a three dimensional view of the surface; ***Fiberscope and bore scopes*** - allow inspection of regions that are inaccessible to the naked eye; ***Tape*** - to measure the dimension of structure, length of cracks; ***Flashlight*** - to aid in lighting the area to be inspected, particularly in power failure; ***Crack comparator*** - to measure the width of cracks ; ***Pencil*** - to draw the sketch of cracks; ***Sketchpad*** - to prepare a representation of wall elevation, indicating the location of cracks, spelling, or other damage, records of significant features such as non-structural elements; Camera - for photographs or video tape of the observed cracking
- ✓ Ordinarily, the structural system is concealed by walls, ceilings, and other architectural elements. The damage inspector may remove ceiling panels for better view of the structural system, but destructive exploration of walls by the damage inspector is ordinarily not done. If the owner is willing, indicate where gypsum or plaster walls and other architectural elements should be removed to facilitate examination of the structural system. Any destructive exploration must be done only by the owner.
- ✓ Look in stairwells, basements, mechanical rooms, and other exposed areas to view the structural system if it is covered elsewhere.
- ✓ Examine the vertical and lateral load-carrying system. Examine for situations of structural components of the resisting system. There are some common types of failure of various components given in Appendix-A and it is also mentioned which type of failure is unsafe/safe. At the same time it is also to be identified that damage/cracking is due to earthquake or other than earthquake. Appendix - B describes some of the causes of cracking other than earthquake.

- ✓ Inspect exposed components of the foundation system and the basement or lowest level floors for fractured components and uneven settlement. Also inspect basement floors and exterior walls for cracks and bulges.
- ✓ Examine every floor, including basement, roof, and penthouse.
- ✓ Remember that severe wall damage and broken glass are evidence of large storey drifts.

STEP 4: Inspect/or Nonstructural Hazards

- ✓ Cladding connections (if visible) masonry partitions, particularly in older buildings where these may be unreinforced
- ✓ Demountable partitions
- ✓ Ceilings and light fixtures, Rooftop water tanks, Other appendages
- ✓ If cladding damage is suspected, inspect representative connections.
- ✓ Severe damage to a nonstructural element does not in itself necessitate posting the entire structure unsafe. Only the unsafe area (e.g., rooms with damaged partitions) should be restricted if the building is otherwise safe.

STEP 5: Inspect/or Other Hazards

- ✓ Elevators should not be restarted without inspection.
- ✓ Look for spills or leaks in areas of stored chemicals or other hazardous materials.
- ✓ If damage to fire protection and detection equipment is observed, it may be necessary to restrict building use.
- ✓ Inspect stairs for structural safety and exits for jammed doors and obstructions.

STEP 6: NDT Tests for Condition Assessment of Structures

- ✓ Visual inspection has the obvious limitation that only visible surface can be inspected. Internal defects go unnoticed and no quantitative information is obtained about the properties of the concrete. For these reasons, a visual inspection is usually supplemented by NDT methods. Other detailed testing is then conducted to determine the extent of deterioration and to establish causes. Appendix – C lists the most common NDT tests being carried out and its limitation.

STEP 7: Complete Checklist and Post Building

- ✓ Evaluate the structure and complete the detailed evaluation
- ✓ Indicate if shoring or bracing or other action is needed.
- ✓ Post the structure according to the results of the evaluation. Use one of the three placards (Inspected, Limited Entry, or Unsafe). Post every entrance to a building classified as Limited Entry or Unsafe.

✓ Explain the significance of Unsafe and Limited Entry postings to building occupants, and advise them to leave immediately. Areas designated unsafe must also be evacuated.

Further Reading

1. FEMA-306 (1999). "Evaluation of Earthquake Damaged Concrete and Masonry wall Buildings" Applied Technology Council (ATC-43 Project), 555 Twin Dolphin Drive, Suite 550, Redwood City, California 94065.
2. Nawy. G. Edward (1997). "Concrete Construction Engineering Handbook", CRC Press, New York
3. Poston, W. R. (1997) "Structural Concrete Repair: General Principles and a Case Study" Concrete Construction Engineering Handbook (editor-in-chief: Edward G. Nawy), Chapter 19, CRC Press, New York.
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6. ATC – 20-3 (1996). "Case Studies in Rapid Postearthquake Safety Evaluation of buildings", Applied Technology Council, Redwood Cot, California..
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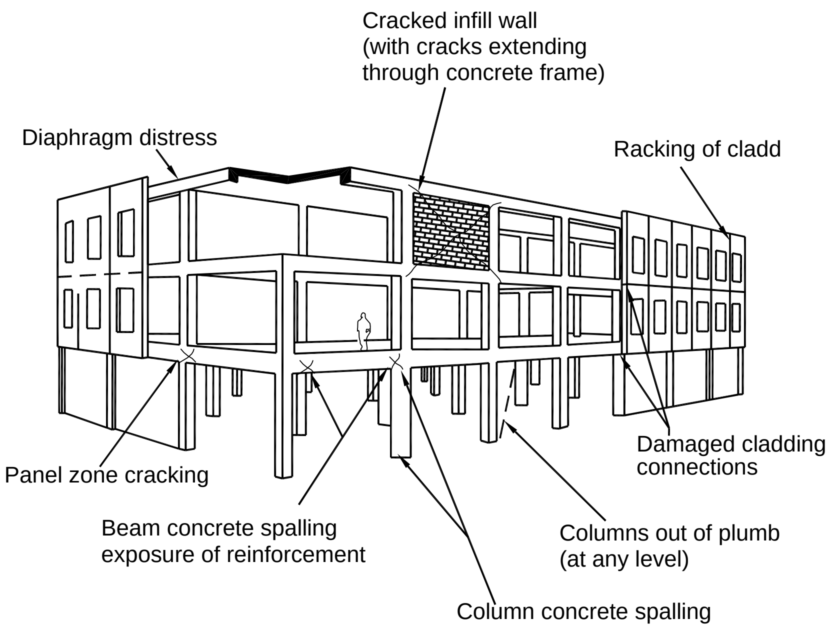
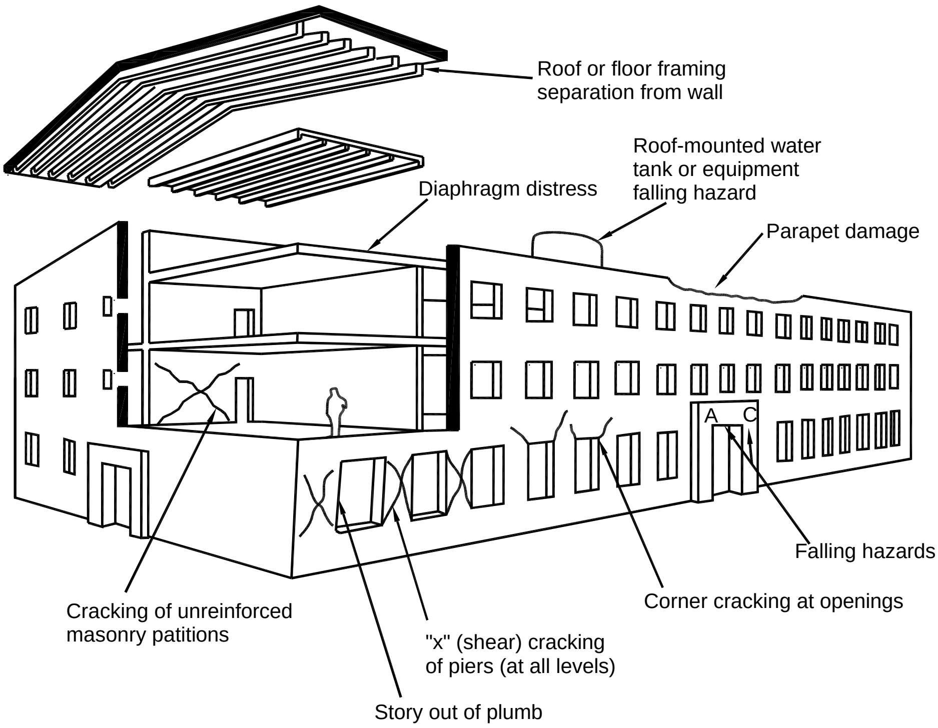
8. ATC – 20 (1989). "Procedures for Postearthquake Safety Evaluation of Buildings", Applied Technology Council, Redwood Cot, California.

Appendix-A: Identification of Damages in Building Components

Possible damages in building component, which are frequently observed after the earthquakes are as follows;





Appendix-B: Concrete Distress and Deterioration other than Earthquake

Table 1 (Poston, 1997) summarizes the list of other causes or reasons of distress and deterioration observed in buildings other than earthquake.

Table 1: Forms of concrete distress and deterioration other than earthquake

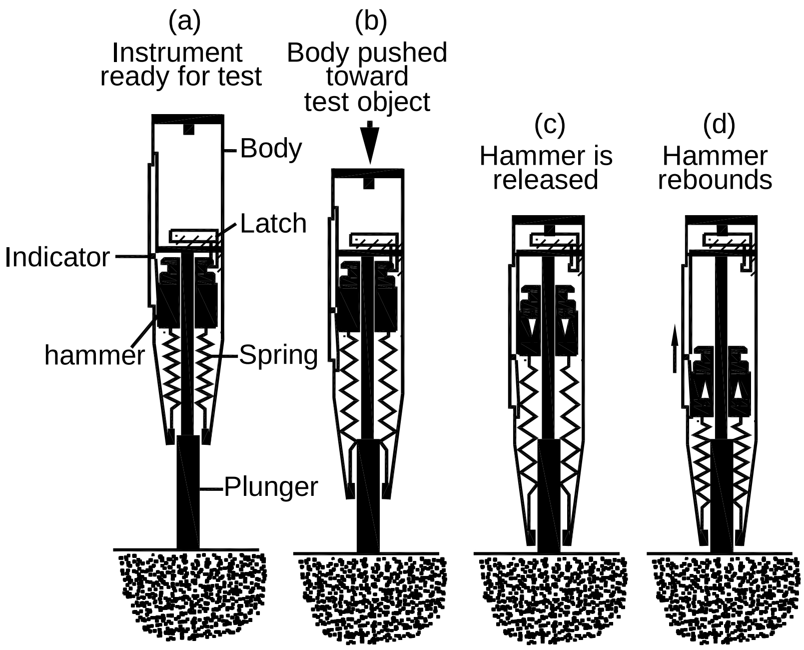
Description	Typical Causes
Cracking	Plastic shrinkage, Drying shrinkage, Restraint, Sub-grade support deficiencies, Vapour barrier, Expansion, Corrosion of reinforcing steel, Thermal loading, Overloading, Aggregate reaction,
Scaling	Inadequate air content, Finishing problems, Freeze-thaw cycling, Chemical de-icers
Spalling	Aggregate reaction, Corrosion, Freeze-thaw cycling, Construction problems: poor preparation of construction joints, early age loading
Disintegration	Frozen concrete, Freeze-thaw cycling, Low strength, Chemical attack, Sulfate attack
Discoloration and straining	Different cement production, Different water-cement ratios, Corrosion, Aggregates, Use of calcium chloride, Curing, Finishing, Non-uniform absorption of forms
Honeycombing and surface voids	Poor placement, Poor consolidation, Congested reinforcement

Appendix- C: NDT Tests for Condition Assessment of Structures

Some methods of field and laboratory testing (FEMA, 1999 and Nawy, 1997) that may assess the minimum concrete strength and condition and location of the reinforcement in order to characterize the strength, safety, and integrity are described here.

Rebound hammer/ Swiss hammer

The rebound hammer is the most widely used non-destructive device for quick surveys to assess the quality of concrete. In 1948, Ernest Schmidt, a Swiss engineer, developed a device for testing concrete based upon the rebound principal strength of in-place concrete; comparison of concrete strength in different locations and provides relative difference in strength only

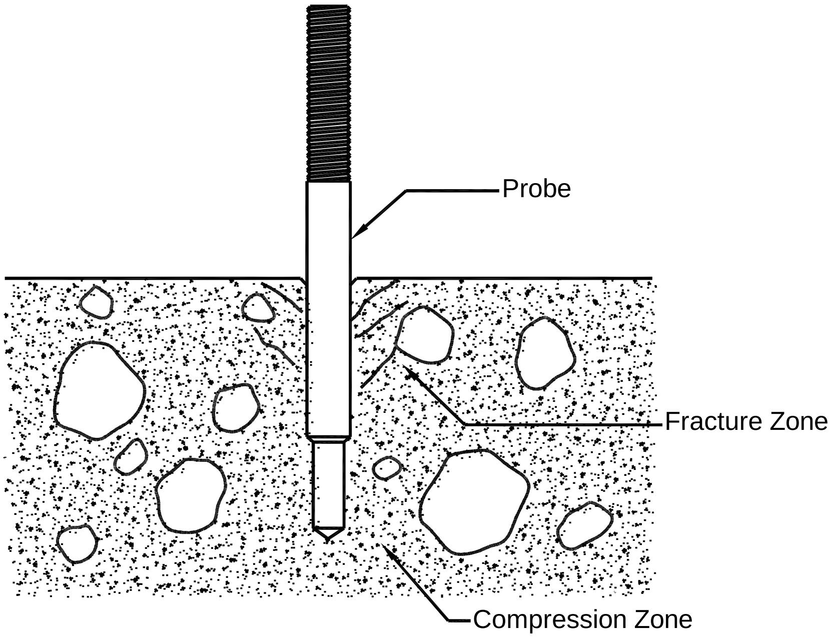


Limitations

- Does not give a precise value of compressive strength, provides estimate strength for comparison
- Sensitive to the quality of concrete on the outer surfaces; carbonation increases the rebound number
- More reproducible results from formed surface rather than finished surface; smooth hard-towelled surface giving higher values than a rough-textured surface.
- Surface moisture and roughness also affect the reading; a dry surface results in a higher rebound number
- Does not take more than one reading at the same spot

Penetration Resistance Method – Windsor probe test

Penetration resistance methods are used to determine the quality and compressive strength of in-situ concrete. It is based on the determination of the depth of penetration of probes (steel rods or pins) into concrete by means of powder- actuated driver. This provides a measure of the hardness or penetration resistance of the material that can be related to its strength.

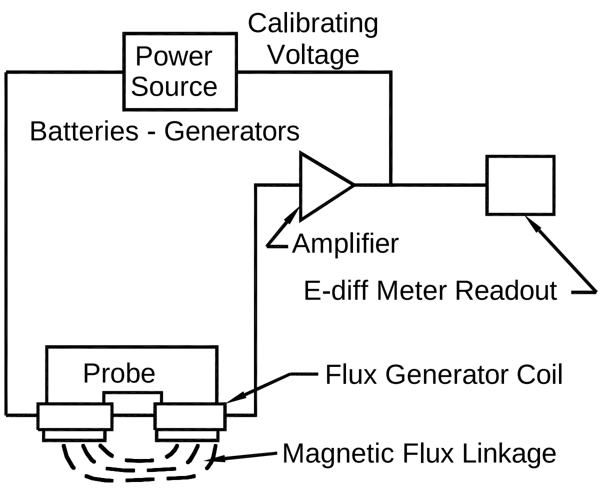


Limitations

- Both probe penetration and rebound hammer test provide means of estimating the relative quality of concrete rather than absolute value of strength of concrete
- Probe penetration results are more meaningful than the results of rebound hammer
- Because of greater penetration in concrete, the probe test results are influenced to a lesser degree by surface moisture, texture, and carbonation effect
- Probe test may cause of minor cracking in concrete

Rebar locator/ convert meter

It is used to determine quantity, location, size and condition of reinforcing steel in concrete. It is also used for verifying the drawing, and preparing as-built data if no previous information is available. These devices are based on interaction between the reinforcing bars and low frequency electromagnetic fields. Commercial covermeter can be divided into two classes: those based on the principle of magnetic reluctance and those based on eddy currents.

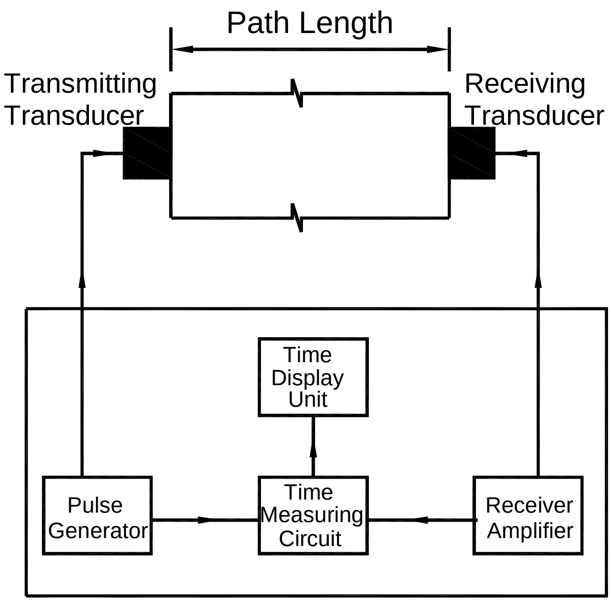


Limitations

- Difficult to interpret - at heavy congestion of reinforcement or depth of reinforcement is too great
- Embedded metals sometimes affect the reading
- Used to detect the reinforcing bars closest to the face

Ultrasonic Pulse velocity

It is used for determining the elastic constants (modulus of elasticity and Poisson's ratio) and the density. By conducting tests at various points on a structure, lower quality concrete can be identified by its lower pulse velocity. Pulse-velocity measurements can detect the presence of voids or discontinuities within a wall; however, these measurements cannot determine the depth of the voids.

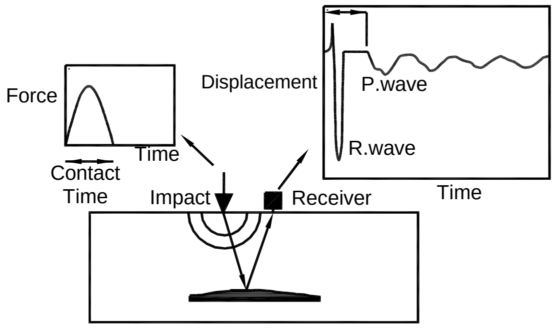


Limitations

- Moisture content: an increase in moisture content increase the pulse velocity;
- Presence of reinforcement oriented parallel to the pulse propagation direction- the pulse may propagate through the bars and result is an apparent pulse velocity that is higher than that propagating through concrete
- Presence of cracks and voids- these can increase the length of the travel path and result in a longer travel time

Impact Echo

Impact echo is a method for detecting discontinuities within the thickness of a wall. An impact- echo test system is composed of three components: an impact source, a receiving transducer, and a waveform analyzer or a portable computer with a data acquisition.



Limitations

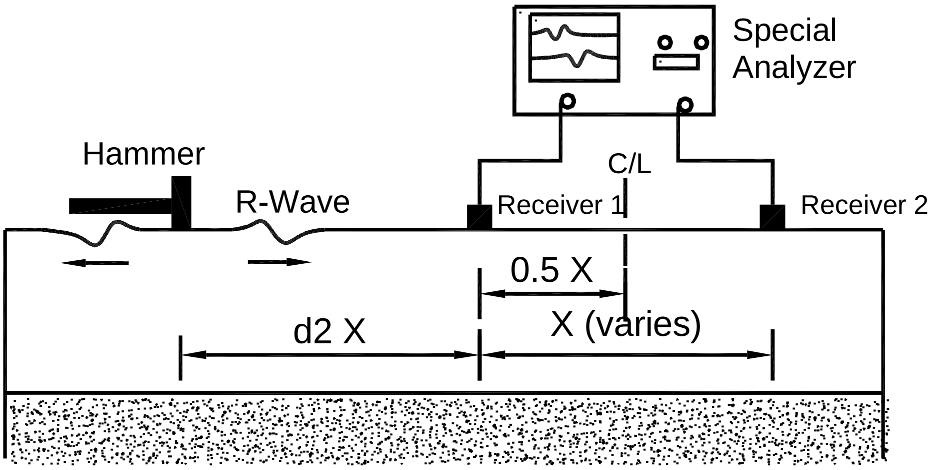
- Accuracy of results is highly dependent on the skill of the engineer and interpreting the results
- The size, type, sensitivity, and natural frequency of the transducer, ability of FFT analyzer also affect the results
- Mainly used for concrete structures

Spectral Analysis of Surface Waves (SASW)

To assess the thickness and elastic stiffness of material, size and location of discontinuities within the wall such as voids, large cracks, and delimitations of surface waves have been used.

Limitations

- Interpretation of results is very complex
- Mainly used on slab and other horizontal surface, to determine the stiffness profiles of soil sites and of flexible and rigid pavement systems, measuring the changes in elastic properties of concrete slab

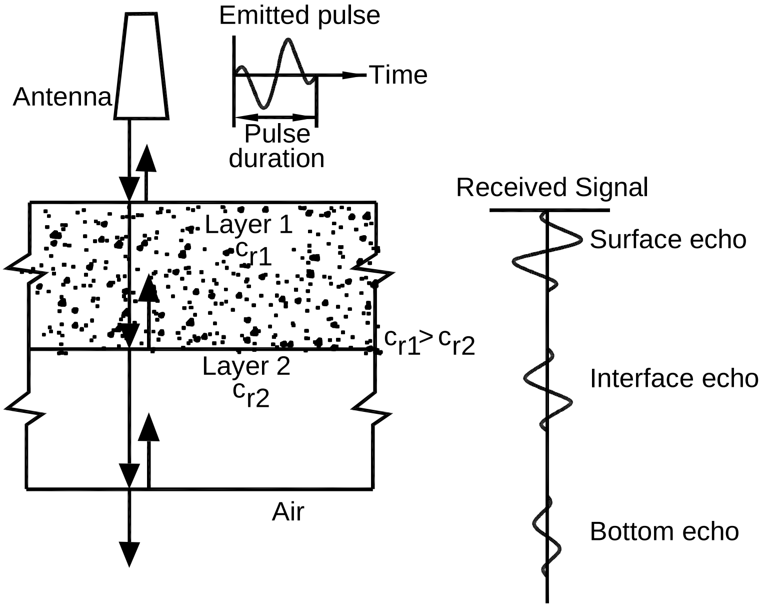


Penetrating Radar

It is used to detect the location of reinforcing bars, cracks, voids or other material discontinuities, Verify thickness of concrete;

Limitations

- Mainly used for detecting subsurface condition of slab-on-grade
- Not useful for detecting the small differences in materials
- Not useful for detecting the size of bars, closely spaced bars are difficult to detect features below the layer of reinforcing steel



Chapter 5

Determination of the Seismic Capacity of an Existing RC Building for Retrofitting

5.1 How to evaluate the seismic capacity of RC frame buildings using non-linear static pushover analysis

Problem Statement: The non-linear static pushover analysis is the most convenient as well as internationally accepted and recommended reliable tool for seismic evaluation of existing and new structures in terms of strength and deformations i.e. a quantitative method for seismic assessment of the building. This method is easy to apply and reasonably accurate to provide adequate information about the non-linear behavior of structure. But the accuracy of the estimate of seismic capacity strongly depends on input parameter such as; axial force-bending moment yield interaction, moment curvature and moment rotation characteristics accounting for appropriate non-linearity of an accurate constitutive material of reinforced concrete element for an accurate pushover analysis. The basic reference document for performing the pushover analysis is FEMA 273 entitled *"NEHRP Guidelines for the Seismic Rehabilitation of Buildings"* and its *Commentary* (FEMA 274) however it is later modified in FEMA 356 and FEMA 440.

PROCEDURE TO CARRY OUT NON-LINEAR STATIC PUSHOVER ANALYSIS

Pushover analysis is basically a step-by-step plastic analysis for which the lateral loads are applied to a structure and progressively increased until a target displacement is reached. It provides information on many response characteristics and gives an overall picture of the structural system behavior. These benefits however come at the cost of additional analysis effort associated with incorporating all of the important characteristics of the structural elements and performing incremental inelastic analysis. Dramatic advancements in the development of computational tools have made inelastic method of analysis easily accessible to the practicing engineers since a number of computer codes/softwares are available. In spite of this the accuracy of inelastic analysis depends upon the user since a large number of input parameter will require during the analysis and it is advisable that the user should go through the research papers in detail, for thorough understanding of different analysis parameters as well as procedural details. Here a few guidelines are provided to carry out the pushover analysis of a building frame;

- ✓ Evaluate the initial dynamic characteristics of the building by creating an elastic model of the building frame on the basis of the gross cross sectional geometry and the initial elastic properties of the concrete in which floor slab have been considered only for their in-plane action and modeled as rigid diaphragm. The columns have been considered fully fixed (restrained) at the base. In this space frame model, the masses are lumped at the appropriate location which is the dead weight of the building and % of live load to add to the mass of the building. From this elastic space frame model is used to compute the initial time period and mode shape of the building.

- ✓ To carry out pushover analysis, a lumped plastic model or concentrated plastic hinge model either in two or three dimensional elastic model of the building frame (as created earlier) has been used. Here plasticity can be lumped only at pre-defined cross section. A non-linear space frame model which includes the most aspects nonlinearities present in the behavior of a building structure under earthquake excitation would be far too complex and impractical; one must therefore aim at something capable of incorporating the most relevant aspects of the structural behavior while keeping the complexity at a bare minimum and assuring operational efficiency. For this reason the lumped plasticity models of the buildings have been used by the most of the popular/professional computer program used in structural dynamics and earthquake engineering.

- ✓ In this model, the plastic hinges are introduced at each of the member ends in the form of non- linear rotational spring since when RC frames are subjected to strong seismic excitation, the inelastic behavior occurs primarily near the member ends and the possibility of plastic hinge formation along the span has been ignored because under earthquake excitation at stress resultants, member ends are usually much bigger than the maximum in the span. The non-linear behaviors of rotational spring are represented in the form of bilinear moment rotation relationship of the section derived from the bi-linear modeling of moment–curvature relationships.

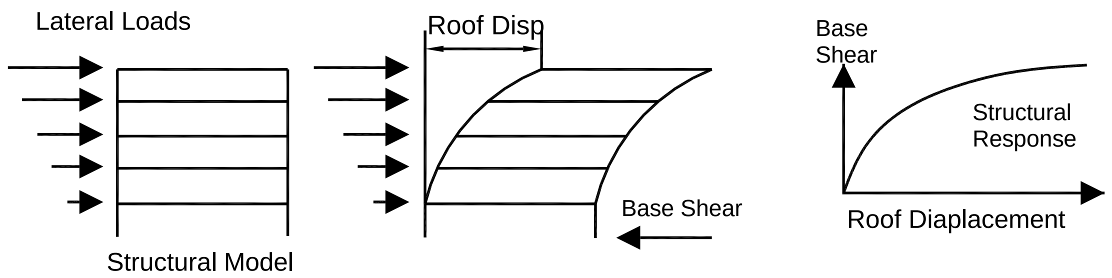
- ✓ Determine the moment –curvature relationship of the section and in turn moment rotation characteristic of the section. It is a very specialized work since it depends on the sectional characteristics, and the load and deformation histories. However, a number of software/spreadsheets are available to compute these relationships. It is always desirable, these input sectional properties must be on the basis of actual constitutive material like concrete and steel not to use the default properties as input (given in software as default) since a minor variation in the non-linear characteristics could result in an unsatisfactory solution leading to wrong assessment and interpretation.

- ✓ Now, this analytical model that accounts for all linear and non-linear response characteristics, applies gravity loads followed by lateral loads in predetermined patterns. The determination of lateral load distribution be too complex to evaluate, but for practical application the distribution of lateral load may be applied as per given equation, corresponding to the first mode of vibration.

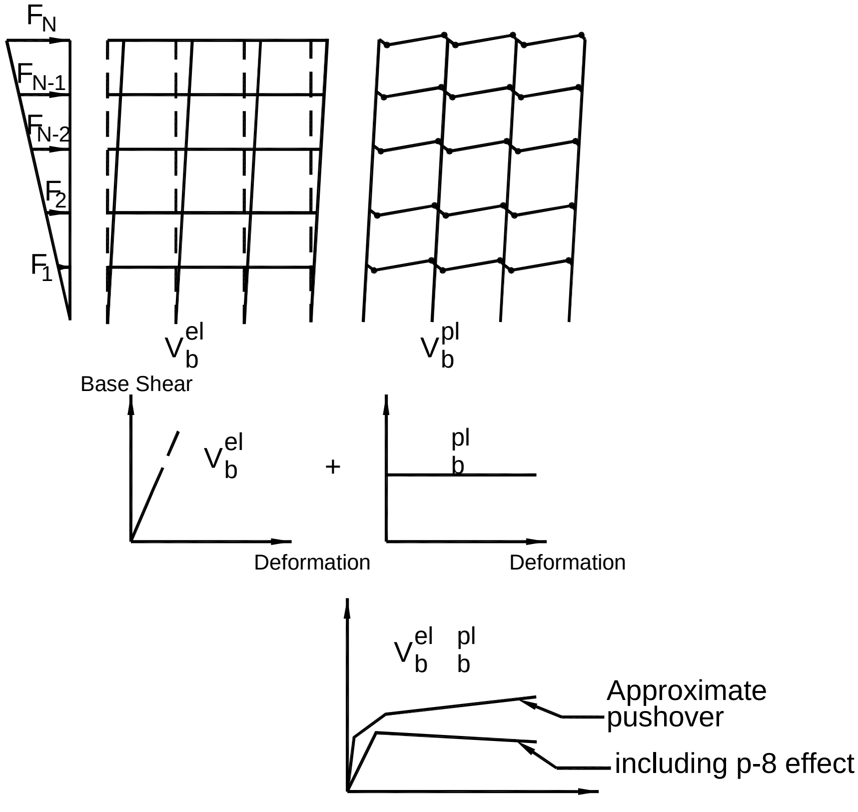
$$\Delta F_i = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \Delta V_b$$

ΔF_i is the force increment at Floor "i" where k is the parameter that controls the shape of the force distribution. The recommended value for k may be calculated as a function of the fundamental period of the structure (T): $1.0 < k = 1.0 + (T - 0.5)/2 < 2.0$. Note that k=0 produce a constant variation of the acceleration (uniform load distribution), while k =1 produced a linear variation (inverted triangular distribution), and k= 2 yields parabolic distribution. The implication of this is that for stiffer structure the higher mode response of the structure will be less significant and the lateral loading can enforce purely first mode response. As the structure becomes more flexible, the higher mode effects become much more important and the k values attempts to account for this by adjusting the lateral load distribution. The pushover analysis will not provide good predictions for tall structures, in which higher mode effects are important. The deformation estimation may be very inaccurate (on the high or low).

- ✓ A vertical distribution of static, monotonically increasing, lateral load is applied to a lumped plastic model of the structure. The loads are increased until the peak response of the structure is obtained. It is essential to apply the dead and live load on the frame before pushing the frame using lateral load. Do not apply pushover load before applying gravity load. The results could be erroneous. The resultant of these forces at each floor has been calculated in terms of base shear against the storey drift/displacement which is plotted, as pushover curve.

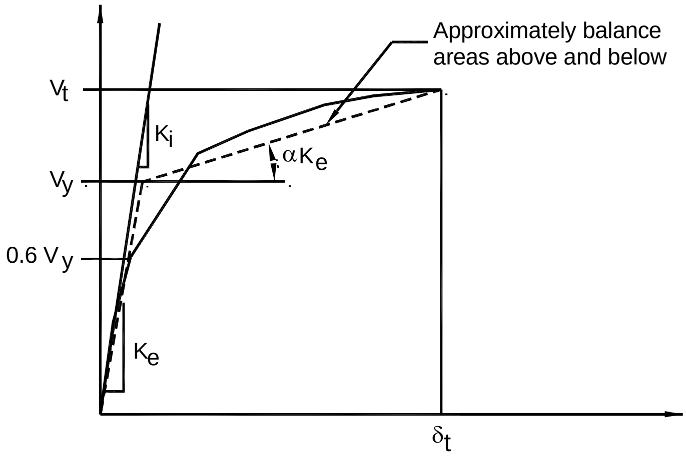


- ✓ The pushover analyses have been conducted to well past the point where the first cross section reaches its ultimate state and therefore breaks down. This point has been evaluated a posteriori and the pushover curves beyond this point represent only an ideal elastic-indefinitely plastic behavior. Therefore, the behavior of the structure up to the collapse may be distinguished in two different phases. The first one, of elastic behavior, identifies the structural response upto the formation of the first plastic hinge. The second one, plastic behavior, is characterized by the formation of a large number of plastic hinges and large inelastic deformations. The non-linear static pushover analysis encourages the design engineer to recognize important seismic response quantities and to use sound judgment concerning the force and deformation demands and capacities that control the seismic response close to failure; the strength and deformation capacity of the structure can be estimated, progressive damage in the structure. It also helps trace the load transfer among various load –resisting structural system in the non-linear range.

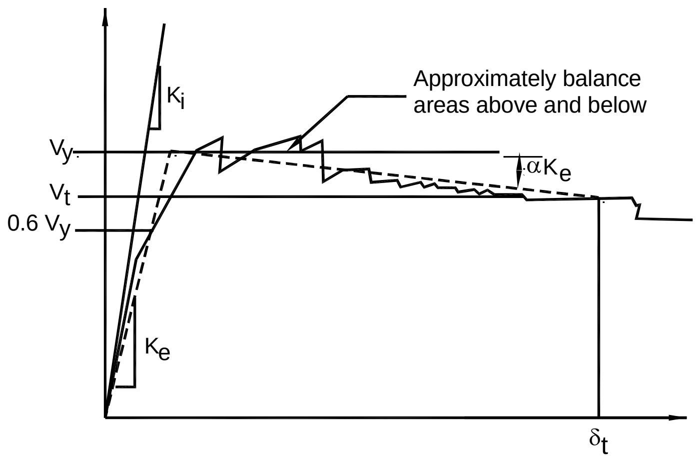


Idealized representation of Pushover capacity

- ✓ The nonlinear force-displacement relationship between base shear and displacement is replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength V_y of the different frames as per FEMA 356. This relationship shall be bilinear, with initial slope K_e and post-yield slope α. The line segments on the idealized force-displacement curve have been located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness, K_e, is taken as the secant stiffness calculated at a base shear force equal to 60% of the effective yield strength of the structure.



(a) Positive post-yield slope

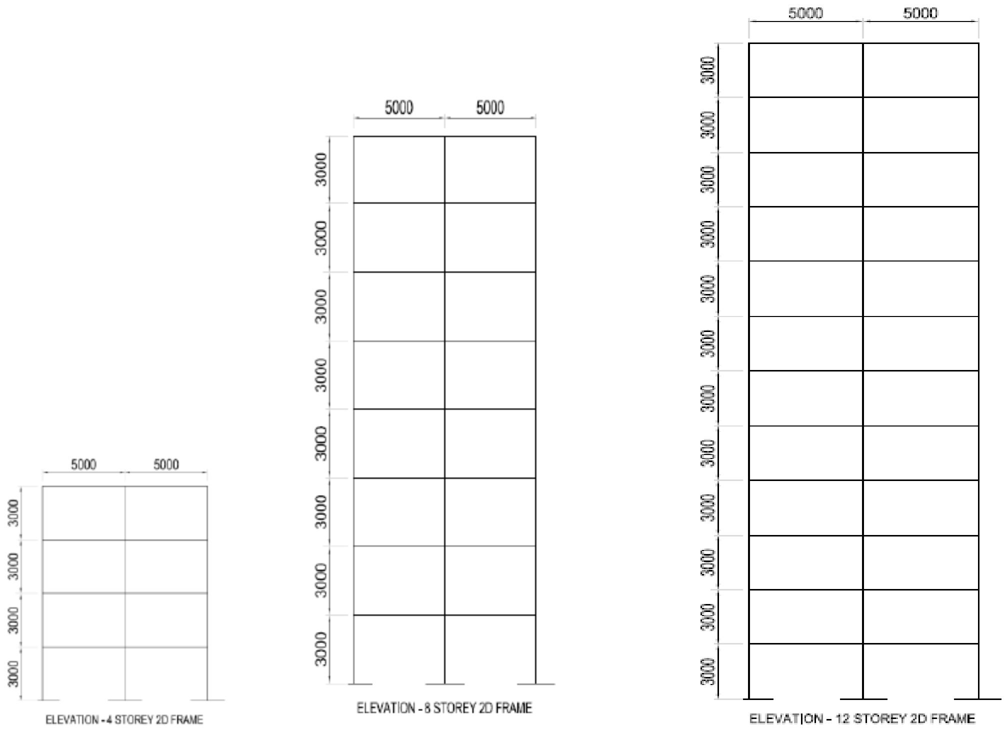


(b) Negative post-yield slope

Idealized Force-Displacement Curves

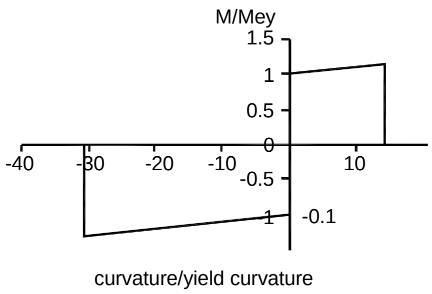
Example on the Non-Linear Static Pushover Analysis

In the present study three two bays RC frames are considered viz. 4 storey RC frame, 8 storey RC frame and 12 storey RC frame. All the three frames are designed using STAAD PRO software first for (a) gravity loading as per IS: 456(2000) and then for (b) seismic loading with ductile detailing as per IS: 13920(1993). Seismic evaluation of the designed frames is then carried out with nonlinear static (pushover) analysis using SAP2000 software. Nonlinear material modeling and ductility evaluation of frame sections are done using XTRACT software.

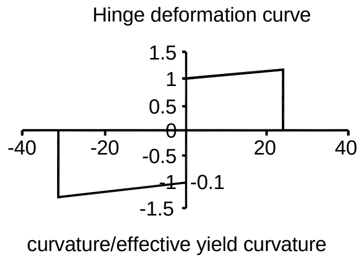


Non-Linear Modeling of Frames

Geometric nonlinearity is provided in the form of P-Δ effects of loading. Material nonlinearity is provided in the form of plastic hinges in the frame elements. Yielding and post yielding behavior is modeled using plastic hinge. Hinges are introduced into frame elements at the ends of the element. For each degree of freedom, a force-displacement (moment-rotation) curve is defined as shown below that gives the yield value and the plastic deformation following yield. Some of the hinge properties obtained using XTRACT are shown as



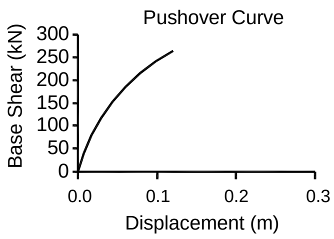
Plastic deformation curve for Beam



Plastic deformation curve for Beam

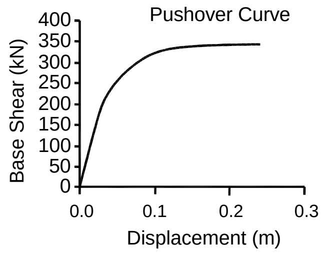
Hinge properties of beam of 4 storey RC frame designed with gravity loading and seismic loading

Evaluation and Results



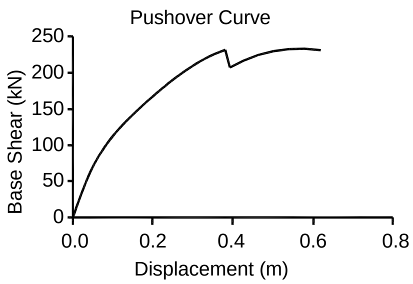
Capacity Curve-4 storey designed for gravity load

Performance point obtained = (164.873, .050)



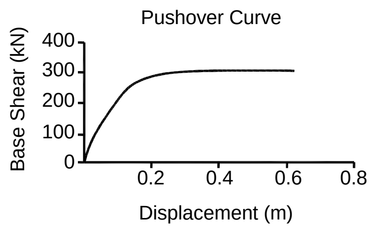
Capacity Curve-4 storey designed with seismic load

Performance point obtained = (244.2, .041)



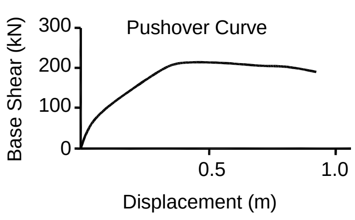
Capacity Curve-8 storey designed with gravity load

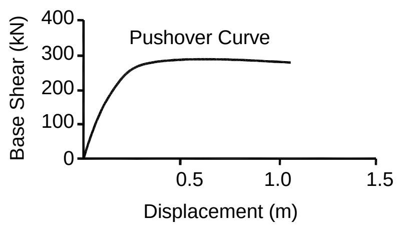
Performance point obtained = (130.43, 0.136)



Capacity Curve-8 storey designed with seismic load

Performance point obtained = (202.868, 097)





Pushover curves for 4, 8 and 12 storey buildings

Comparison of results

Three RC building frames of varying heights namely 4 storeys, 8 storeys and 12 storeys have been analyzed and designed with and without earthquake forces. A non-linear static (pushover) analysis has been carried out from these frames to evaluate the strength, deformation and location of hinges. A comparison of the results is as follows;

RC frame designed with gravity loading	**Base shear at yield point (kN)**	**Displacement at yield point (m)**
4 storey	164.873	0.05
8 storey	130.43	0.136
12 storey	141.83	0.202
RC frame designed with seismic loading	**Base shear at performance point (kN)**	**displacement at Performance point (m)**
4 storey	244.2	0.041
8 storey	202.868	0.097
12 storey	202.454	0.162

Further Reading

1. ATC-40, (1996), "Seismic Evaluation and Retrofit of Concrete Building," Applied Technology Council.
2. FEMA 273, 1998)"NEHRP Guidelines for the Seismic Rehabilitation of Buildings" Federal Emergency Management Agency
3. FEMA 274, (1998)"NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings" Federal Emergency Management Agency
4. Khah, M. A. (2008). "Seismic Evaluation of RC frame Buildings using Pushover and Non-linear Dynamic Analysis", M. Tech Thesis, Department of Earthquake Engineering, IIT Roorkee
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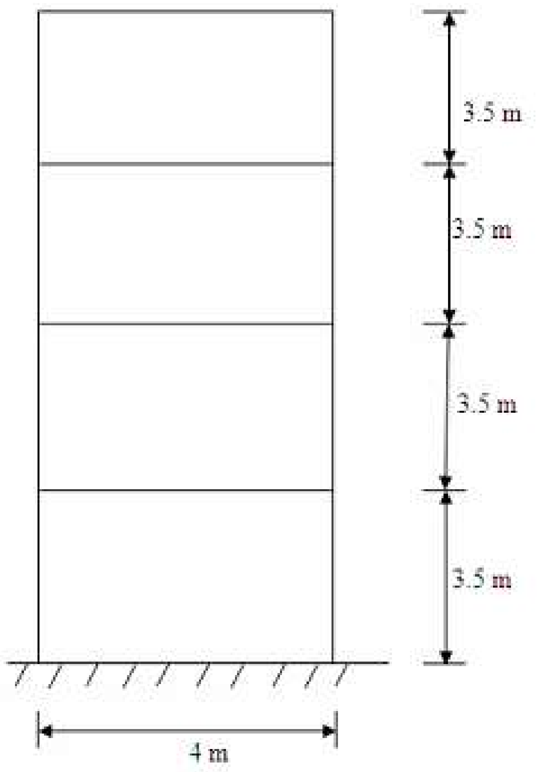
6. Calia, I. and Marletta, M. (2005). "Seismic Performance of a Reinforced Concrete Building not Designed to withstand Earthquake Loading", Seismic Prevention of Damage, WIT Press, Southampton, Boston
7. Oliveto, G., Calia, I. and Marletta, M. (2005). "Seismic Resistance and Vulnerability of a Reinforced Concrete Building not designed for Earthquake Motion", Innovative Approaches to Earthquake Engineering, WIT Press, Southampton, Boston
8. ASCE/SEI 41-06 (2007). "Seismic Rehabilitation of Existing Buildings", ASCE, USA.

5.2 How to evaluate the seismic capacity of an existing RC building before and after retrofitting?

Problem statement: The purpose of this example is to illustrate the procedure for determining the amount of retrofitting required in an RC building. The amount of retrofitting basically depends upon the current seismic capacity of the building and the capacity of the same building if designed with the present day code requirements. The difference between the two capacities will be narrowed down by adding suitable retrofitting scheme.

Evaluation the Seismic Capacity of an Existing RC Building

A single bay 4 storey (G+3) 2D frame has been considered. The storey height of all floors is 3.5 m. A solid slab of thickness 150 mm has been considered for all storeys. The frames have been analyzed and designed with different versions of the Indian codes (IS: 456 and IS: 1893) as given in Table 1. This gives a comprehensive picture of the seismic vulnerability of this structure in the light of the latest version which helps to provide retrofitting measures for those frames having inadequate capacity in comparison to latest code requirements.



Frame under consideration (Plan dimension of the frame = 4m X 4 m, Slab Thickness = 150 mm, Brick wall thickness = 250 mm)

Table 1: Five sequences used for analysis and design of RC frames

Sequence	Year	Concrete code	Seismic Code	Design Theory
1.	1964	IS:456-1964	N.A.	WSM
2.	1970	IS:456-1964	IS:1893-1970	WSM
3.	1975	IS:456-1964	IS:1893-1975	WSM
4.	1984	IS:456-1978	IS:1893-1984	LSM
5.	2002	IS:456-2000	IS:1893-2002	LSM

Load calculation of the frame

Sequence	Load at roof level (kN/m)		Load at floor level (kN/m)		Total earthquake load in terns of base shear (kN)	Remark
	DL	LL	DL	LL		
1.	11.5	3.0	8.7	8.0	Not considered	Base shear has been determine by equivalent static method as per respective codes in Appendix A.
2.	11.5	3.0	8.7	8.0	18.62	
3.	11.5	3.0	8.7	8.0	22.28	
4.	11.5	3.0	8.7	8.0	35.65	
5.	11.5	3.0	8.7	8.0	44.6	

Assume dead and live load on roof = 5.75 kN/m^2 and 1.5 kN/m^2; at floor level = 4.35 kN/m^2 and 4.0 kN/$m^2$and; weight of infill wall = 17.5 kN/m

Design of Frames

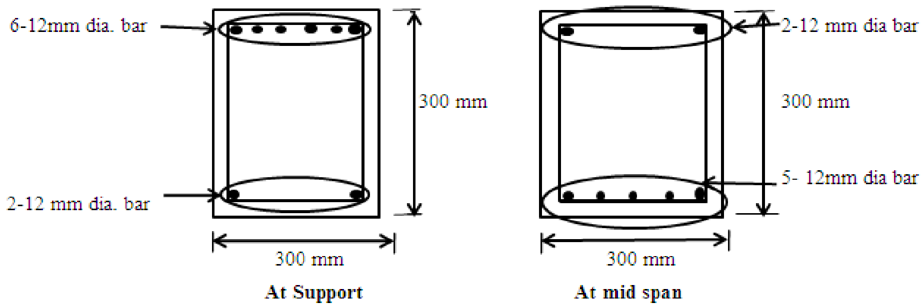
The frames are first designed for gravity loading and then for seismic loading according to different revisions of Indian codes. For analysis and design of RC frames STAAD-Pro software package is used. For limit state design for gravity loading following load combination cases are considered:

1. Dead Load.
2. Live load.
3. 1.5 (Dead Load+ Live load).

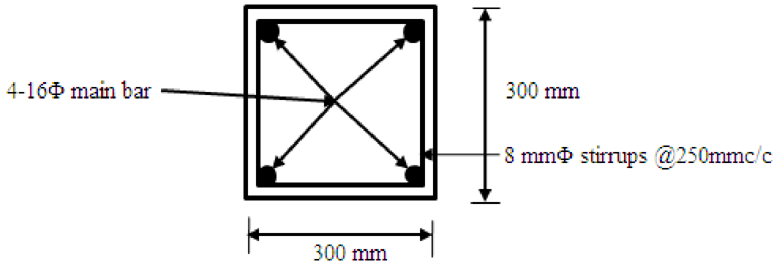
The following load combinations have been used for limit state design with seismic loading:

1. 1.5 (Dead Load+ Live load).
2. 1.2 (Dead Load+ Live load ± EL_X).
3. 1.5 (Dead Load ± EL_X).
4. 0.9 Dead Load ± 1.5EL_X.

Reinforcement details of the frame design as per sequence No. 1 (IS 456: 1964)



Details of Beams



Details of Columns

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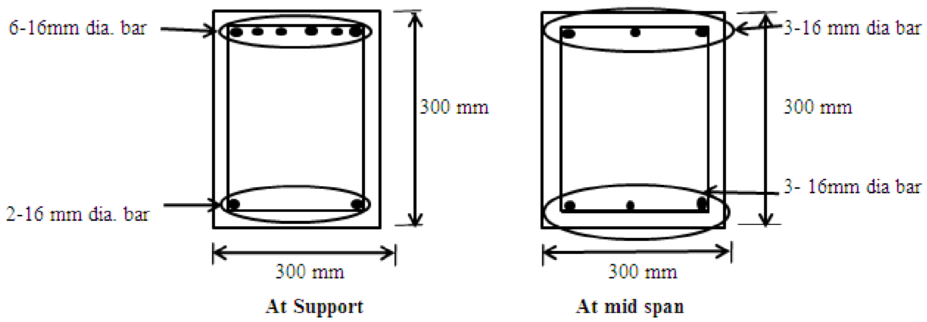

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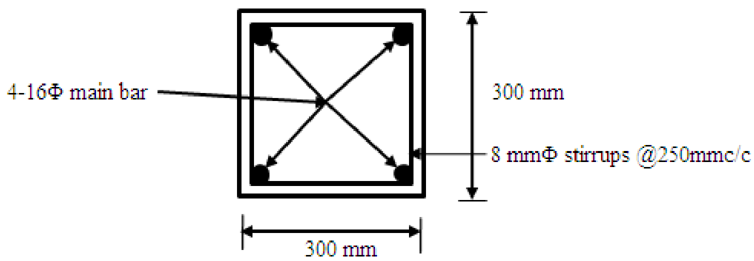

Details of Columns

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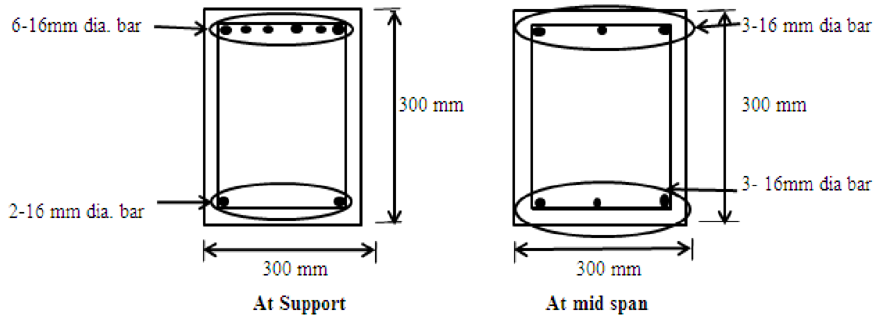
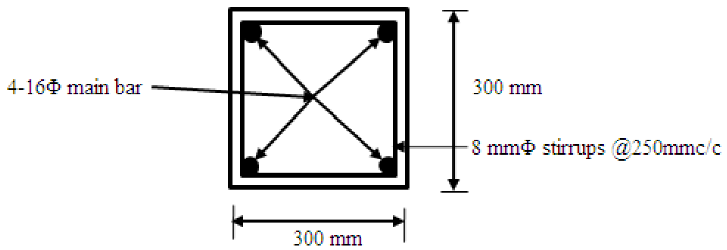


Details of Beams



Details of Columns

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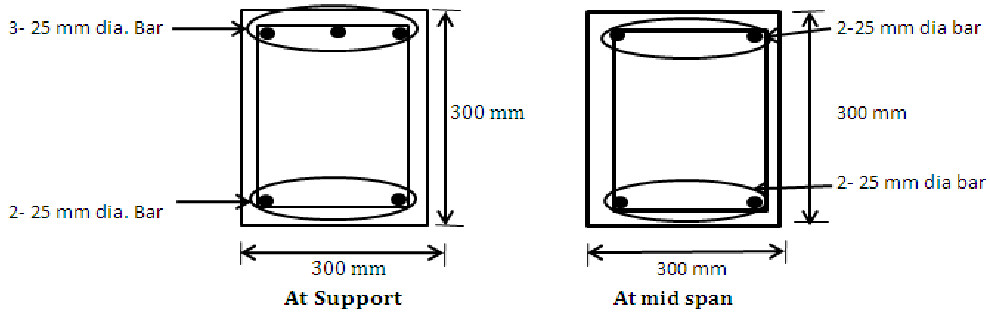

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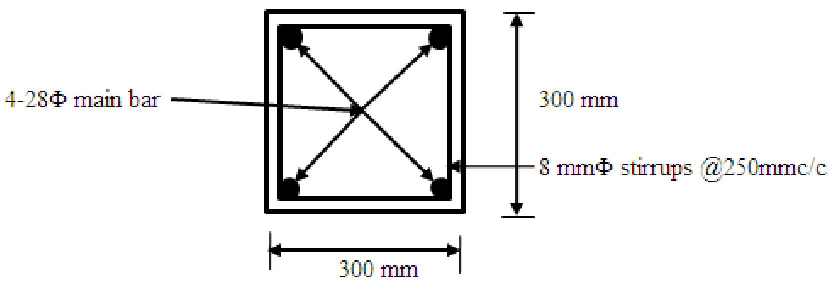
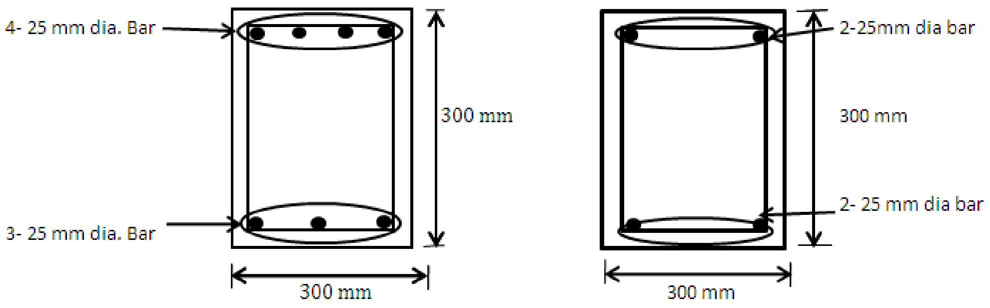


Details of Columns

Reinforcement details of the frame design as per sequence No. 5 (IS: 1893: 2002 and IS: 456: 2000)



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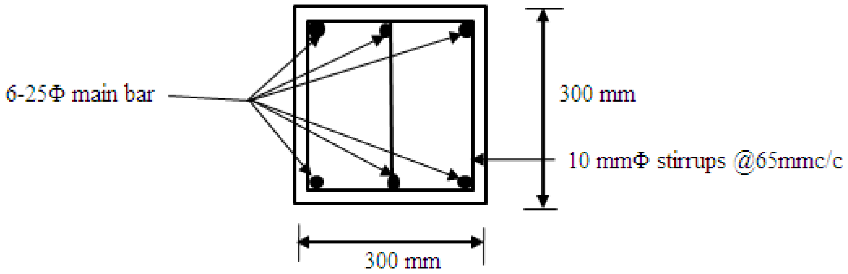
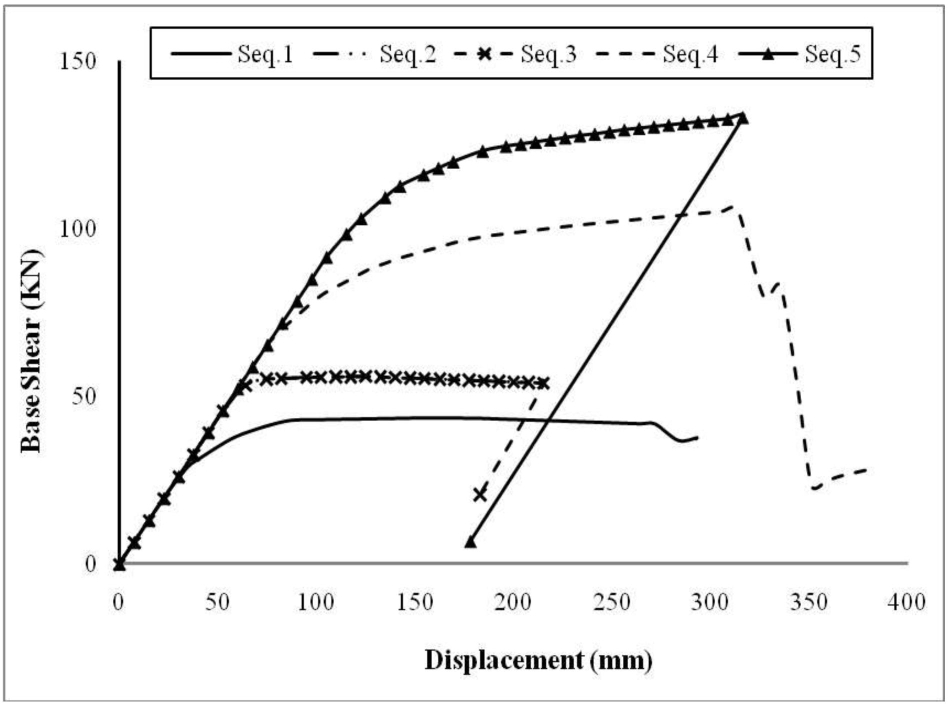


Details of Columns

NON-LINEAR STATIC PUSHOVER ANALYSIS OF THE FRAME

After designing and detailing nonlinear static (pushover) analysis of the 4 storied 2D frames which are designed according to different codes have been carried out using SAP2000 to find out its existing capacity. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The lateral loads are applied monotonically in a step-by-step nonlinear static analysis. Two types of nonlinearity, considered for modelling, are geometric nonlinearity and material non linearity. The capacity curves of the 4 storey 2D frame designed as per different codes are obtained as shown below. All the curves show similar features. They are initially linear but start to deviate from linearity as the beams and the columns undergo inelastic actions. When the buildings are pushed well into the inelastic range, the curves become linear again but with a smaller slope. These curves could be approximated by a bilinear relationship.

It is clear from the plotted curves that the capacities of frames designed with only gravity (DL+LL) loading [Seq.1] and the frames designed according to earlier codes [Seq.2 to Seq.4] are less than the capacity of the frame designed according to present day code.



Calculation of different pushover parameters

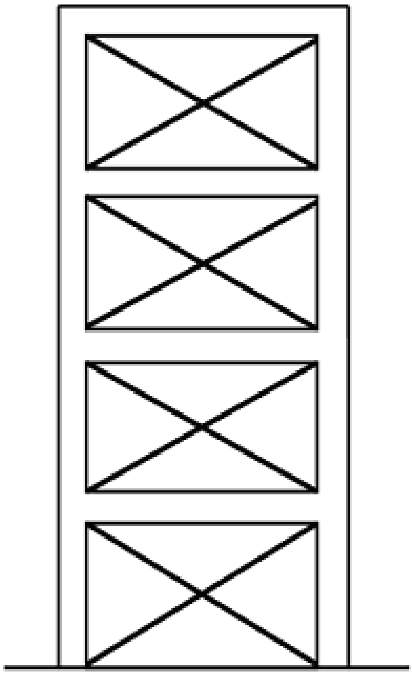
The nonlinear force-displacement relationship between base shear and displacement is replaced with an idealized bi-linear relationship [as discussed earlier] to calculate the different parameters. Different pushover parameters including strength and ductility are extracted from the capacity curves for different frame which are tabulated below. It is concluded that the existing building in seismic zone IV, designed and constructed using previous Indian standards is found inadequate to withstand the revised present day code. The maximum base shear calculated at the roof level of the 4 storey frame according to IS :1893 (Part-I) -2002 increases upto about 2.40, 2.0 and 1.25 times as compare to the values in 1970, 1975 and 1984 respectively.

Table 2: Different pushover parameters extracted after idealization of pushover curves

Design Sequences	Sequence 1	Sequence 2	Sequence 3	Sequence 4	Sequence 5
Yield base shear (KN)	39	54	54	86	116
Ultimate base shear (KN)	43	55.9	55.9	105.6	133
Effective Stiffness (N/mm)	866.7	900	900	860	859.3
Post-Yield Stiffness (N/mm)	36.7	29.7	29.7	92.0	94.4

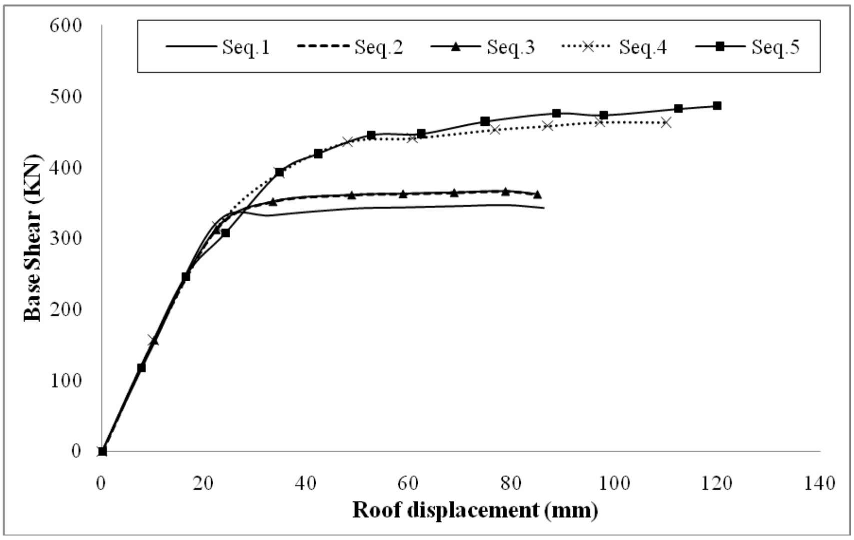
RE-EVALUATION OF THE SEISMIC CAPACITY OF RETROFITTED RC BUILDING USING STEEL BRACING TECHNIQUES

It has been observed that the building frames designed from the earlier codes are inadequate in terms of strength and ductility to present day code requirements. The seismic capacity of these frames can be enhanced by applying any suitable retrofitting techniques. Steel bracing techniques for seismic retrofitting have been used in this study. The steel bracing used is of cross pattern (X-bracing) of size 2 ISA 60x60x8 connected back to back at a spacing 8 mm is used as a retrofitting measure.



PUSHOVER CURVES OF DIFFERENT STEEL BRACED FRAMES

Steel bracing members (double angle back to back) are modelled as truss member. X bracing (cross bracing) system has been considered. In the cross pattern of steel bracing, additional joints (nodes) are created at intersection point of diagonal braces. The connection between steel brace and frame have been made rigid by providing end length offset with rigid zone factor 1, i.e. the entire connected zone has been made rigid. The pushover curves of the 4 storey 2D frame retrofitted with steel bracing are obtained by nonlinear static analysis using SAP 2000. A comparative pushover curve of different frames is shown below:



Idealization of the Pushover Curves of Retrofitted Frames

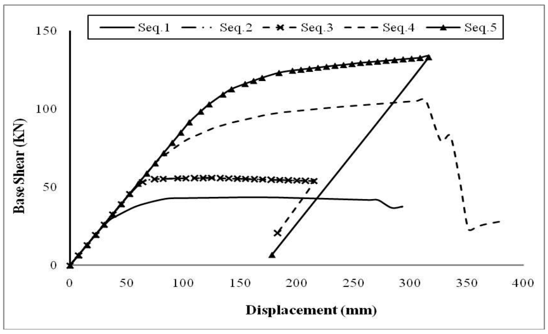
The nonlinear force-displacement relationship between base shear and displacement of the retrofitted frames is replaced with an idealized bi-linear relationship to calculate the different parameters. Different pushover parameters including strength and ductility are extracted from the capacity curves for different frame that are tabulated below:

Table 3: Different Pushover parameters of frame retrofitted with steel braces

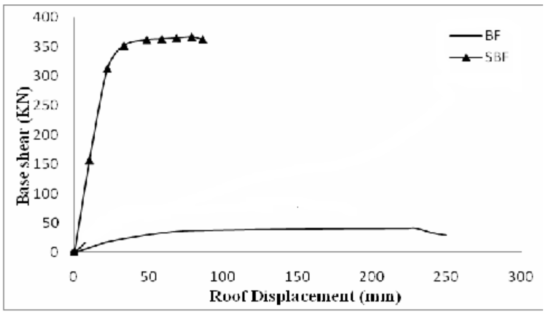
Design Sequences	Sequence 1	Sequence 2	Sequence 3	Sequence 4	Sequence 5
Yield base shear (KN)	330	340	340	390	395
Ultimate base shear (KN)	342	366	366	463	485
Ductility	3.54	3.59	3.59	4.07	4.62
Effective Stiffness (N/mm)	15000	15454	15454	15000	15192
Post-Yield Stiffness (N/mm)	214.3	456	456	698	957

COMPARISON OF THE PUSHOVER CUVES BEFORE AND AFTER RETROFITTING

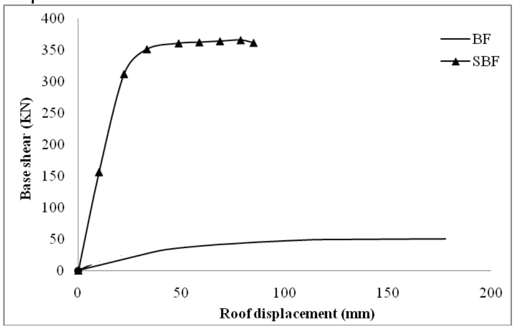
The capacities of the frames designed as per different sequences (Seq.1 to Seq.5) before retrofitting (as bare frame) and after retrofitting (using steel bracing) are compared. By comparing the curves it can be concluded that by adding steel bracings the lateral strength of ordinary moment resisting frame can be enhanced in the lateral strength of the system.



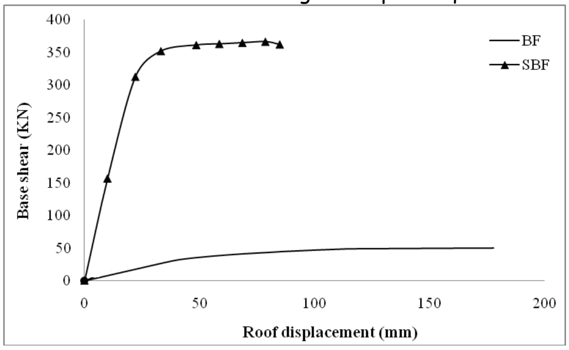
Pushover curves before retrofitting under different sequences



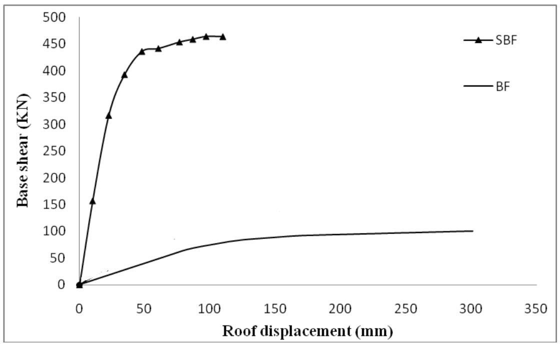
Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 1



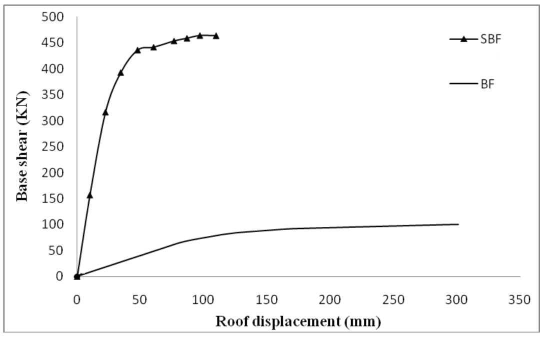
Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 2



Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 3



Comparison of pushover curves before and after the retrofitted of the frame designed as per Sequence 4



Comparison of pushover curves before and after the strengthening of the frame designed as per Sequence 5

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Appendix A: Earthquake Load Calculations

Determination of Base Shear as per IS 1893:1970

The total base shear (V_B) = $C.\alpha_h.\beta.W$
T = Time period = 0.1n =0.1 x 4 = 0.4 (Here n= 4 as four storey frame)
$C = 0.5/T^{1/3} = 0.677$
α_h = 0.05 (Roorkee, Zone- IV)
β = 1.2 (For type-II soil)
The base shear is given by -

$$V_B = C.\alpha_h.\beta.W$$

W = Seismic Weight = 87 +123.8+123.8 +123.8 = 458.4 KN
So, Total Base shear = (0.677 x 0.05 x 1.2) x 458.4 = 18.62 KN

Base shear distribution along the height of frame (as per IS:1893-1970)

Floor	h_i(m)	W_i(KN)	$W_ih_i^2$	V_B (kN)	Qi (kN) = V_B x(W_i h_i^2/ΣW_i h_i^2)
Roof	14	87	17052	8.29	8.29
3rd	10.5	123.8	13648.95	14.93	6.64
2nd	7	123.8	6066.2	17.88	2.95
1st	3.5	123.8	1516.55	18.62	0.74
		Σ=	38283.7		

Determination of Base Shear as per IS 1893:1975

Horizontal Seismic co-efficient (α_h) = $\beta.I.F_0.S_a/g$
β = 1.2 (For type-II soil)
I = Importance factor = 1.0 (Residential building)
F_0 = 0.25 (Roorkee, Zone- IV)
T = Time period = 0.1n =0.1 x 4 = 0.4(Here n= 4 as four storey frame)
S_a/g = 0.18 (For T=0.4 and 5% damping)

The base shear is given by -

$$V_B = C.\ \alpha_h.W$$

C = A co-efficient defining the flexibility of the structure = 0.9
W= Seismic Weight = 87 +123.8+123.8 +123.8 = 458.4 KN
So, Total Base shear = 0.9 x (1.2 x 1.0 x 0.25 x 0.18) x 458.4 = 22.28 KN

Base shear distribution along the height of frame (as per IS: 1893-1975)

Floor	h_i(m)	W_i(KN)	$W_ih_i^2$	V_B (kN)	Qi (kN) = V_B x(W_i h_i^2/ΣW_i h_i^2)
Roof	14	87	17052	9.93	9.93
3rd	10.5	123.8	13648.95	17.88	7.95
2nd	7	123.8	6066.2	21.41	3.53
1st	3.5	123.8	1516.55	22.3	0.88
		Σ=	38283.7		

Determination of Base Shear as per IS 1893:1984

Horizontal Seismic co-efficient (α_h) = $\beta.I.F_0.S_a/g$
β = 1.2 (For type-II soil)
I = Importance factor = 1.0 (Residential building)
F_0 = 0.25 (Roorkee, Zone- IV)
T = Time period = 0.1n =0.1 x 4 = 0.4 (Here n = 4 as four storey frame)
S_a/g = 0.18 (For T = 0.4 and 5% damping)
The base shear is given by -

$$V_B = K.C.\ \alpha_h.W$$

K= Performance factor = 1.6
C = A co-efficient defining the flexibility of the structure = 0.9
W= Seismic Weight = 87 +123.8+123.8 +123.8 = 458.4 KN
So, Total Base shear = 1.6 x 0.9 x (1.2 x 1.0 x 0.25 x 0.18) x458.4 = 35.65 KN

Base shear distribution along the height of frame (as per IS: 1893-1984)

Floor	h_i(m)	W_i(KN)	$W_ih_i^2$	V_B (kN)	Qi (kN) = $V_B \times (W_i\ h_i^2/\Sigma W_i\ h_i^2)$
Roof	14	87	17052	15.88	15.88
3rd	10.5	123.8	13648.95	28.6	12.71
2nd	7	123.8	6066.2	34.24	5.65
1st	3.5	123.8	1516.55	35.65	1.41
		∑=	38283.7		

Determination of Base Shear as per IS 1893 (Part-I):2002

Total Base shear (V_B) = A_h x W

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

Z = Zone factor = 0.24
I = Importance factor = 1.0 (Residential building)
R = Response Reduction factor = 3.0 (Ordinary moment resisting frame)
T = Time period = 0.075 $h^{0.75}$ = $0.075(14)^{0.75}$ = 0.56 sec.
S_a/g = 1.36/T = 1.36/0.56 = 2.43
$\therefore$ A_h = (0.24/2) x (1/3) x (2.43) = 0.0972
Total Seismic weight of building (W) = 87 +123.8+123.8 +123.8 = 458.4 KN

Base shear distribution along the height of frame (as per IS: 1893-2002)

Floor	h_i(m)	W_i(KN)	$W_ih_i^2$	V_B (kN)	Qi (kN) = $V_B \times (W_i\ h_i^2/\Sigma W_i\ h_i^2)$
Roof	14	87	17052	19.87	19.87
3rd	10.5	123.8	13648.95	35.77	15.90
2nd	7	123.8	6066.2	42.84	7.07
1st	3.5	123.8	1516.55	44.60	1.76
		∑=	38283.7		

So, Total Base shear = 0.0972 X 458.4 = 44.6 KN

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