Lecturer notes for
Training of Practicing Architect

NATIONAL PROGRAMME FOR CAPACITY BUILDING FOR ARCHITECTS

IN

EARTHQUAKE RISK MANAGEMENT

(NPCBAERM)

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INDIAN INSTITUTE OF TECHNOLOGY ROORKEE

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PREFACE

As an integral part of National Programme for Capacity Building of Architects for Earthquake Risk Management (NPCBERM), initiated by the Ministry of Home Affairs, Government of India, the seven identified National Resource Institutes have completed their assigned task of training of around 250 faculty members (and government architects) from more than 100 institutions of architecture , designated as the State Resource Institutes (SRIs) in the past 18 months under Training of Teachers (TOT) phase. The next stage of the Programme envisages the SRIs to undertake the training of 10,000 practicing architects in one week training modules to be arranged in the respective regions of the SRIs. A few of the architectural institutes have already started their work in this direction.

This compendium of resource material has been prepared to serve the teachers of the SRIs, the institutions of architecture and all the professional architects as a sort of extended notes for the training modules. But its usefulness goes beyond the training sessions. As this reference volume has been compiled and edited from the lecture notes of a number of distinguished experts in the field of earthquake architecture and engineering, its value, from the point of view of gaining good theoretical and professional understanding of the subject of earthquake architecture, is highly enhanced. One of the prime objectives of NPCBAERM is to ensure seismically safer habitats through the training of practicing architects and this resource volume contributes directly and gainfully to that end.

A lot of productive work has gone into the making of this note which has been made possible only with the involvement of a number of individuals and we are happy that many of them are from the Department of Architecture and Planning, Indian Institute of Technology, Roorkee.

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CHAPTER 1

BUILDING SAFETY FROM NATURAL HAZARDS

AN INTRODUCTION

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BUILDING SAFETY FROM NATURAL HAZARDS: AN INTRODUCTION

1.1 INTRODUCTION

The unique geo-climatic conditions of India make this region particularly vulnerable to natural disasters. Indian subcontinent is among the world's most disaster prone areas. Hazard profile of India tells us that 59% of land is vulnerable to hazards like earthquakes, 8% of land is vulnerable to cyclones and 5% of land is vulnerable to Floods. These disasters along with others occur with unfailing regularity and the losses caused by them continue to mount year after year. This fact emphasizes the importance of protecting our buildings from hazards to prevent disastrous situations.

This chapter introduces various types of disasters occurring in India and general building safety guidelines to reduce their impact. Guidelines for protecting buildings from earthquake disaster are discussed in detail in others chapters of the book and hence are not covered here.

1.2 DISASTERS

<u>Disaster is the occurrence of a sudden or major misfortune, which disrupts the basic fabric, and normal functioning of a society.</u>

WHO defines disaster as 'any occurrence that causes damage, economic destruction, loss of human life and deterioration in health and health services on a scale sufficient to warrant an extraordinary response from outside the affected community or area.' A disaster is the product of a hazard such as earthquake, flood or windstorm coinciding with a vulnerable situation, which might include communities, cities or villages.

1.2.1 Hazards

Hazards are defined as "<u>Phenomena that pose threat to people, structures or economic assets and which may cause a disaster</u>". They could be either man made or naturally occurring in our environment (UNDRO). A natural hazard pertains to a natural phenomenon which occurs in proximity and poses a threat to people, structures and economic assets caused by biological, geological, seismic, hydrological or meteorological conditions or processes in the natural environment.

1.2.2 Classification of Hazards

There are four basic types of hazardous events that put societies at risk. Those are:

i. Those based in ***nature:*** Earthquake, Flood, Drought, Cyclone, Tsunamis, Heat /cold wave, Landslides, Hailstorm, Avalanche.
ii. Those based in ***violence***: War, armed conflict, physical assault, etc.
iii. Those based in ***deterioration***: Declining health, education and other social services, environmental degradation, etc.
iv. Those based in the ***failing of industrialized society***: Technological failures, oil spillage, factory explosions, fires, gas leakage, transport collisions.

We are however concerned only with hazards which pose threat to buildings viz. earthquake, floods, cyclones, tsunamis, landslides, fire etc.

1.2.3 Hazards Affecting Buildings

***E**arthquake:* Earthquake is shaking of earth due to release of strain energy due to breaking of crust or movement of tectonic plate.

***F**lood:* Flood is submergence of a wide area because of heavy local rainfall or overflowing of river or damage to some water-body nearby.

***C**yclone:* Cyclone is a large-scale closed circulation system in the atmosphere with low barometric pressure and strong wind that rotate counter-clockwise in northern hemisphere and clockwise in southern hemisphere.

***T**sunamis:* Tsunami is a shallow water waves that propagate in great speeds transferring energy from the source across oceans and towards land.

***L**andslides:* Any rapid down - slope movement of a mass of regolith and/or bedrock, under influence of gravity

***F**ire:* Fire is uncontrolled burning of forest trees or train or building or shop.

1.3 BUILDING SAFETY AGAINST HAZARDS

Characteristics of different hazards and precautionary measures to be taken to protect our built environment from different hazardous events are given here. The earthquake disaster is not covered here since it has been discussed in detail in the subsequent chapters.

1.3.1 Floods

India is highly vulnerable to floods and every year Ganga and Brahmputra basins experience floods affecting states of Uttar Pradesh, Bihar, West Bengal, Assam and Orissa on regular basis. Flood hazard and disaster are brought on by human development, affecting both the immediate floodplain and properties downstream and along the shore. Though traditionally human settlements occurred near sources of water for drinking water, agriculture, transport and power generation, in recent decades development along waterways and shorelines has been spurred by the aesthetic and recreational value of these sites. The result has been the formation of a downward spiral consisting of:

1. An increasing exposure to damage and destruction wrought by the natural forces of flooding on human development.
2. The forces of flooding being amplified by development activity insensitive to the dynamics of flooding.

Causes of Large Floods

Different types of floods are flash floods, single-peak flood, multiple flood and synchronized flood. Causes for these are:

1. Prolonged, heavy and wide spread monsoons;
2. Avulsion and channel shift;
3. River bed aggradations due to siltation and damming of rivers;
4. Back flooding and seawater flooding;
5. Reduction in the water carrying capacity and
6. Failure of natural and man made dams.

Measures for flood mitigation

The various measures adopted for flood mitigation may be categorized into two groups:

1. **Structural**

   The general approach is aimed at preventing floodwaters from reaching the potential damage centers, as a result of which a large number of embankments came up along the various flood prone rivers. The mainthrust of the flood protection programme in the form of structural measures is grouped into the following:

   - Dams and reservoirs;
   - Embankments, flood walls, sea wall;
   - Natural detention basin;
   - Channel improvement;
   - Drainage improvement and
   - Diversion of flood waters.

   For effective functioning of all the physical measures taken, it is necessary that pre- and post monsoon checks must be made and special repairs must be carried out prior to flood period.

2. **Non – Structural**

   The non structural measures on the other hand, aim at modifying the susceptibility of flood damage as well as modifying the loss burden. The various non – structural measures are:

   - Flood Plain Management;
   - Flood proofing including disaster preparedness and response planning;
   - Flood forecasting and warning;
   - Disaster Relief and
   - Flood fighting including Public Health Measures.

1.3.2 Cyclones

The term cyclone denotes all tropical storms. It is known as 'Hurricanes' in the Atlantic and the eastern pacific; 'Typhoons' in western pacific, 'Willy Willy' in Australia and 'Bagius' in Philippines. Damage is limited to 50-60 kms beyond coast line.

In India from the year 1891 to1990 total 262 cyclones occurred within a 50 km wide strip on the East Coast out of which 92 were severe. Less severe cyclonic activity is on West Coast which is 33 cyclones in the same period out of which 19 were severe. The Indian subcontinent is the worst cyclone affected due to the following reasons:

- Low depth ocean bed topography;
- Coastal configuration and
- Wind and Cyclones.

The Orissa Cyclone (October 1999)

Two massive cyclones hit the Orissa coast spaced in just two weeks. The events have been traumatic and demoralizing for Orissa. The first cyclone on October 17 and 18, 1999, inundated the areas of Gopalpur, Brahmapur, Chhatrapur and Ganjam with more than 150 miles of wind and torrential rain. The super cyclone of Friday, October 29, 1999, beat all records. The winds went beyond 250 miles an hour, the sky went all dark and there was frightening lightning with incessant rains. The loss of life and property is estimated at five billion US dollars.

The main reasons of failure in non – engineered buildings are as follows:

- Poor site selection;
- Inappropriate shape of building especially slope of the roof;
- Large roof projections;
- Poor construction practices;
- Lack of continuity in walls and roof;
- Poor connections;
- Corrosion of steel bars and connections, and
- Bad fabrication of wooden or steel trusses.

These defects must be corrected in order to minimize the impact of cyclones in future.

Cyclone Shelters

One of the most successful means of reducing loss of human lives during cyclones is the provision of cyclone shelters. In densely populated coastal areas, where large scale evacuations are not always feasible, community buildings and buildings used for gathering of large number of persons, like schools, dharamshalas, hospitals, prayer halls, etc. can be used as cyclone shelters. These buildings can be so designed, so as to provide bland façade, with a minimum of apertures in the direction of prevailing winds. The shorter side of the building should face the storm. Alternately these buildings can be designed on a circular / ellipsoidal plan, so as to impart least wind resistance. Earth berms and green belts can be used in front of these buildings to reduce the impact of the storm. Mangrove, Palmyra, Casuarina trees are found to be good barriers for cyclones. These shelters should be located in relatively elevated areas with provision for community kitchen, water supply and sanitation.

Another alternative although expensive is to have individual cyclone shelters attached for installation on flat ground adjacent to the house. The shelter is installed 4 feet in the ground, the dirt from excavation is placed around the shelter to help increase the shelter's effectiveness in protecting its occupants.

1.3.3 Landslides

Landslides are defined as the mass movement of rock, debris or earth down a slope. They are influenced by geological, geo-morphological, climatic, environmental, hydrological and seismological conditions and by biological factors. In India the North Sikkim and Garhwal Himalayas are often affected by landslides with the average being 2 landslides / sq.km./year, with soil loss of 2500 ton/sq.km./year. In this region almost 80% of the landslides occur in 4 months of monsoon where annual rainfall is from 3500 to 5000 mm. Situations of cloudburst (more than 1000mm rain in 24 hours) also trigger landslides. Landslides are also very common in Nilgiri hills and Western Ghats of India. The unprecedented rains in 1978 triggered over 100 landslides in Nilgiri hills while in Western Ghats also, the frequency of landslides is on increase.

Causes of Landslides:

Various causes of landslides are:

- Unstable geological conditions;
- Indiscriminate construction activity;
- Exceptionally heavy rainfall, cloudbursts, flash-floods;
- Poor drainage due to urbanization and
- Deforestation.

Categories of landslides

All landslides can be grouped into two categories

- Rain induced slope failures and
- Earthquake induced landslides.

Mechanism of land sliding

Mechanisms of land sliding include:

- Development of fissures (mostly subsequent to seismic events);
- Widening of these and also inherent joints in the laterites occupying the plateau due to tremors/percolation of rainwater;
- This gradually leads to a state of in-equilibrium.

Prior symptoms of landslides:

Some prior symptoms of landslides are:

- Development of new cracks or unusual bulges in probable crown area;
- Development of springs, seepages or saturated ground in the areas not wet earlier;
- Change in position, amount of discharge or amount of turbidity of springs – a year or two prior to landslides;
- Tilting of electricity poles, trees, fences, rods etc.;
- Sinking of roads or ground surfaces;
- Cracking of floors, foundations or compound walls and
- A decrease in the population of burrowing animals.

Remedial measures for landslides

Two most important remedial measures which could be taken for landslides are:

1. Control Work: modification of the natural conditions such as topography, geology, groundwater etc.
2. Restraint Work: construction of structures such as surface and sub- surface drainage works, removal of earth from the unstable area, building buttress walls, piles, anchors, retaining walls.

Four-fold strategy for control of landslides:

- Reducing slope angle;
- Placing additional supporting material at foot of slope;
- Reducing load on slope by removing rock/soil high on slope and
- Use of retention structures and removal of fluid by drainage systems.

1.3.4 Tsunami

Tsunami is a Japanese word meaning 'The Harbor Wave.' Tsunami is a shallow water waves that propagate in great speeds transferring energy from the source across oceans towards the land. They have great destructive potential, as they are increasing in height as they approach land.

Causes of Tsunamis

Tsunamis are usually caused by underwater earthquakes. These often occur offshore at subduction zones (places where a tectonic plate that carries an ocean is gradually slipping under a continental-plate). Hence, a receding sea usually precedes a tsunami wave. In most cases there is also drawdown of sea level preceding crest of the tsunami waves. Landslides can also cause tsunamis by displacing large volumes of water. (For e.g.: On July 17 1998, more than 2100 people were killed in Papua, New Guinea) If Volcano collapses and slides into the ocean it may also create a very large tsunami wave. Hence we can say that:

- They are caused by earthquakes or landslides.
- Tsunamis can be generated when the seafloor abruptly deforms vertically displacing a large volume of water.
- Displaced water mass under the influence of gravity forms waves around it in order to reach equilibrium.
- Tectonic earthquakes can cause tsunamis when a tectonic place subsides or rises.
- Along plate faults is where vertical movements of plates take place. Subduction zones are usually sources of large tsunamis.
- During subduction earthquake, the offshore ocean bottom lifts up the land along the coast lowers down.

The December 2004 Tsunami

Sunday 26 December 2004: at 0100 GMT, an 8.9 magnitude earthquake occured on the seafloor near Aceh in northern Indonesia. It created the fourth largest tsunami since 1900 hitting the coasts of Indonesia, Malaysia, Thailand, Myanmar, India, Sri Lanka, Maldives and even Somalia. The epicentre of this great earthquake was near the triple point junction of three major tectonic plates (see section 2.2.4) where large catastrophic earthquakes and tsunamis have occurred for millions of years. It occurred along the boundary where the great tectonic plates of India and Australia collide and subduct underneath the Sunda and Eurasian plates. The total death toll was above 2.5 lakh, which was aggravated by the fact that there was no warning.

Reasons for Disastrous Effects of Tsunamis

Tsunamis throughout history have been associated with tragedy and loss of human lives. The disastrous effects of Tsunamis are because of the following facts:

- Tsunamis behave like shallow water waves because of the very small ratio between water depth and their wavelength.
- The velocity of waves is dependent on depth. In 4000m water depth tsunami can travel at 720 kmph.
- Even though these waves transfer enormous amounts of energy in jetliner speeds they are completely harmless in deep waters.
- Because of the low ratio between water depth and wavelength, the slope of tsunami waves is very small giving them the ability to travel across oceans unnoticed.
- Because the rate of energy loss of a wave is inversely related to its wavelength, tsunamis not only can travel transoceanic distances in jetliner speeds, they can also travel great distances without loosing significant amounts of energy. (159 people died in Hawaii from tsunami waves generated almost 3700 km away in Alaska's islands.)

- As tsunami reaches the shore the water depth decreases and speed decreases significantly.
- Flux of energy carried remains almost constant, its crest height increases to account for the kinetic energy lost.
- Sudden decrease of the wave speed from 700 kmph to 100kmph can result to an increase in crest height from a couple of m to 20 m high wave. (During Okushiri tsunami in Japan in 1993 wave upto 31 m high were reported. It killed 239 people.)
- The tendency of tsunami waves to align parallel to the shoreline they approach, may present a disadvantage to a protruding headland as the waves tend to wrap around it before they smash into it with great force.

Destructions caused by Tsunamis

Tsunami can cause destruction in three ways:

- Wave impact on structures
- Inundation
- Erosion

Major forms of destructions brought out by Tsunamies are:

- Flotation and drag forces have moved houses and overturned railroad cars.
- Tsunami associated wave forces have demolished frame buildings and other structures.
- Tsunamis propagate inland with tremendous amount of power causing great destruction to structures as well as coast morphology and vegetation.
- Strong tsunami induced currents have led to erosion of foundations and the collapse of bridges and seawalls.

Tsunami Prediction and Alert System

Earthquake could not be predicted in advance and hence there is no direct technology which can predict a Tsunami. However, with new technological advancements, once the earthquake is detected it is possible to give alert well before tsunamis reach land. Such a system of warnings is in place across the Pacific Ocean but not in the Indian Ocean. The mechanism is as given below:

- Deep-ocean assessment and reporting of tsunamis (DART) System consists of special buoys strategically placed along the seismic zone.
- At ocean floor a sensitive pressure detector is installed that can detect pressure differences created by tsunamis with wave heights of only a few centimeters. They can distinguish tsunamis from other waves.
- Detector sends the signal to the buoy through an acoustic link and buoy transmits it through satellite to DART stations along the coast.
- This process takes place in less than a minute. Sophisticated computer software at the stations analyses the signal and they calculate magnitude, direction and speed of tsunami in a matter of minutes.
- This allows plenty of time for communities to evacuate and also offers more confidence to the officials and coastal residents eliminating the false alarms associated with older systems.
- Even though DART technology is a great advancement, but alone cannot save lives. Since future destructive tsunamis are inevitable coastal residences around the world should learn to recognize signs associated with incoming tsunamis.

Precautions during a Tsunami

During peace times proper education must be given to high risk residents. And during tsunami people should be told to save themselves by moving quickly to higher grounds.

1.3.5 Fires

In India the direct & indirect annual losses due to fire is estimated at Rs 1200 crore and 20,000 fatal injuries. These fire accidents & resulting losses are increasing with rapid rise in industrialization and urbanization, due to several factors as high rise construction, congestion in squatter and slum pockets, inner city areas and changing life styles and materials used in constructions. Lack of proper building guidelines, enforcement of the codes, improper installation& maintenance of active systems, are aggravating our vulnerabilities to fires. The death and destruction due to fire triggered by earthquakes are the designer's inability to understand the sensitive relationship between the built infrastructure and forces of nature/environment. Fires are caused as a combined effect of three elements forming the triangle of fire (Fig. 1.1).

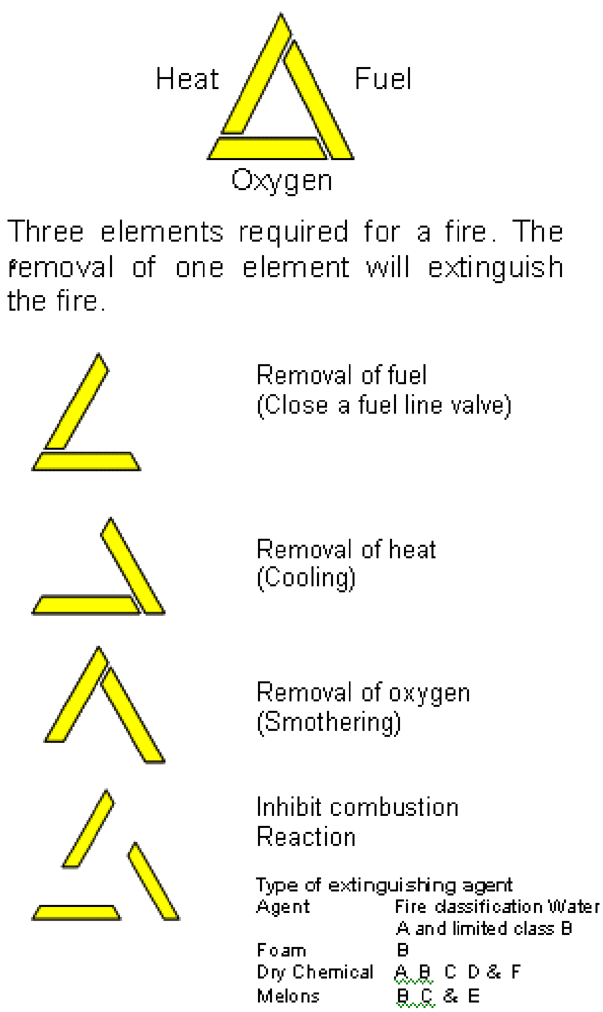

Figure 1.1: Triangle of Fire

Classification of Fires

Building fires can be classified as

- Class A- Fires in solid fuel
- Class B- Fires in inflammable liquids
- Class C- Fires in inflammable gases
- Class D- fires in inflammable metals (e.g. Aluminum, Uranium)
- Class E- Fires caused by electrical faults.

Fundamental Safety Mechanisms

1. Structural fire protection
2. Means of escape in case of fire

Structural fire protection

1. <u>Design objective</u>:
   Provide fire resistance for a reasonable period of time which being anticipated in the given scenario & thereby will provide for
   - Delay to the spread of fire;
   - Ultimate collapse of the structure;
   - Reaction and egress time for persons in danger to escape and
   - Enable fire fighting to be commenced.

2. <u>Building classification</u>:
   Buildings can be graded as to the amount of overall fire resistance required by taking into account the size of building, function character of building and the fire load.
   - **Grade 1** – (1 hr). Typical buildings within this grade are flats, offices, restaurants, hotels, hospitals, schools, museums and public libraries.
   - **Grade 2** – (2hr) Moderate fire load, typical examples are retail shops, factories and workshops.
   - **Grade 4** – (4 hr) High fire load, Typical examples are certain types of workshops and warehouses

3. <u>Fire resistance of materials</u>:
   Fire resistance of a building element is given in minutes and is the time interval from the start of the test until failure occurs in stability, integrity or insulation of the building.

4. <u>Building Regulations</u>:
   The basic aim of relevant Building Regulations is to limit the spread of fire. This is achieved by considering the
   - Functional use of building;
   - Fire resistance of structural elements and surface finishes;
   - Size of the building or parts of a building and
   - The degree of isolation between buildings or parts of buildings.

5. <u>Functional Classification of buildings</u>:
   - Residences;
   - Institutional - hospitals, schools and similar establishments;
   - Office - any premises used for office purposes;
   - Shops - commercial activities;
   - Factory - generally as defined by relevant Factories Act;
   - Public /private spaces;
   - Storage and general premises - used for goods, materials, parking etc. and
   - Any premises not included in purpose groups mentioned above would be placed in this classification.

6. <u>Area Zoning</u>:
   - *Escape route* – a continuous path by way of a space, room, corridor, staircase or other means of passage but excluding a lift, escalator, escape or fireman's ladder
   - *Place of safety* – an unenclosed space in the open air at ground level
   - *Protected zone* – comprises approach lobbies, stairway and final exit lobby
   - *Unprotected zone* – sometimes called a clearway and is that portion of an escape route leading to a protected zone.

Means of escape in case of fire:

In the context of means of escape in the event of fire break out, the building and its contents are of secondary importance. The provision of a safe escape route should, however, allow at the same time an easy access for the fire brigade using the same routes. Since these routes are protected, the risk of fire spread is minimized. In practice the provision of an adequate means of escape and structural fire protection of the buildings and its contents are virtually inseparable.

1. Common Factors for fire escapes:

Each building has to be considered as an individual exercise but certain common factors prevail in all cases for e.g.

- An outbreak of fire does not necessarily imply the evacuation of the entire building;
- Persons should be able to reach safety without assistance using the protected escape routes;
- Rescue facilities of the local fire brigade should not be considered as part of the planning of means of escape, and
- All possible sources of an outbreak and the course the fire is likely to take should be examined and the escape routes planned accordingly

2. Planning of escape routes:

The various aspects to be considered while designing means of escape are:

- The user type;
- Levels of risk;
- Unfamiliarity with building layout;
- Problems of smoke and
- The reaction time available to evacuate the building.

3. Fire precautions:

The basic fire escape principles embodied in the National Building Code (NBC) are:

- Limitation of travel distances;
- Escape route considered in 3 stages
  - Travel distance within rooms;
  - Travel distance from rooms to a stairway or final exit;
  - Travel within stairways and final exit.
- Provision of a protected route which is defined as a route for persons escaping from fire which is separated from the remainder of the building by fire-resisting doors (except doors to lavatories), fire resistant walls, partition and floors.

4. Planning and designing escape routes:

Considerations for planning and designing of escape routes are:

- Number of persons who may reasonably be expected to be on the premises; typical population densities are as follows:
  Offices – 3.7 m2 per person where layout consists of individual rooms and taken for the net floor area or 5 m2 per person for open planned offices calculated on the gross floor area
  Shops – 4.5 to 7 m2 per person of gross sales floor area but for ground floors of department and self service food stores allow 0.5 to 1 m2 of net circulating area
- Fire resistance of the structure and of materials used in the construction and finishes
- Vulnerability of proposed escape routes to smoke and fire.
- Fire load
- Existence of openings in the floor, such as stairwells, which would permit the passage of smoke, fire and hot gases.

Fire protection systems

There are two types of fire protection systems viz. active and passive. These systems do not require power or water to contain fires.

Active: Involves the control of smoke spread, detection and communication process that informs the fire outbreak and triggers some sort of counteraction towards extinguishing the fire.

Passive: A proactive approach taken at the building design stage, aimed at addressing a comprehensive solution to the fire problem. It is an all encompassing fire safety concept which embraces the passive measures in fire containment design in addition to augmenting the active measures.

Passive fire protection

Scope:

- Structural fire protection
- Safe escape routes and refuge for occupants
- Compartmentation & containment of fire spread
- Preserving the function of active fire safety measures
- Life safety of fire personnel

Application of passive fire protection:

- Structural Steel Protection - Strength reduces drastically when temperature rises beyond 500^0c, which is most common in uncontained fire. Risk of collapse increased.
- Mechanical And Electrical Enclosures - Electrically operated system is well protected.
- Fire Rated Partitions - Fire safe lobby and safe escape passage ways are critical to life safety.
- Fire Rated Ceilings - Engineering services passing above the unprotected ceiling – a source of fire propagation
- Fire Rated Ducts - When the smoke temperature is high the sheet metal ducts become distorted, making it useless for air flow or smoke extraction
- Fire Stops/Seals – Barriers to prevent fire propagation.

Present scenario

- Passive fire protection market in India is in the developing stage, concentrated on the passive fire protection for load bearing structures using concrete mostly in the industrial segment.
- In the commercial segment, virtually no passive fire protection concept exists.
- Although fire regulations followed in Indian market are based on National Building Codes (NBC) which follows the basic guidelines of the British Standards (BS), they are not product or application specific.

1.4 EMERGENCY MANAGEMENT

Emergencies do not just appear one day, rather, they exist throughout time and have a life cycle of occurrence, and hence the management strategy should match the phases of an emergency in order to mitigate, prepare, respond and recover from its effects. Emergency management requires a close working partnership among levels of the government (national, state, district and taluka), the private sector (business and industry, voluntary organizations) and the general public.

1.4.1 Emergency Management Cycle

Mitigation, Preparedness, Response and Recovery are visualized as having a circular relationship to each other as shown in fig 1.2. Each phase results from the previous one and establishes the requirements of the next one. The activities in one phase may overlap those in the previous. Four phases of Emergency Management are

(A) **Response** - occurs during and immediately following a disaster. They are designed to provide emergency assistance to victims of the event and reduce the likelihood of secondary damage

(B) **Recovery -** the final phase of the emergency management cycle. It continues until all systems return to normal, or near normal. Short-term recovery returns vital life support systems to minimum operating standards. Long-term recovery from a disaster may go on for years until the entire disaster area is completely redeveloped; either as it was in the past or for entirely new purposes that is less disaster prone.

(C) **Mitigation** – activities, which actually eliminate or reduce the vulnerability or chance of occurrence or the effects of a disaster.

(D) **Preparedness** - *planning* how to respond in case an emergency or disaster occurs and working to increase resources available to respond effectively.

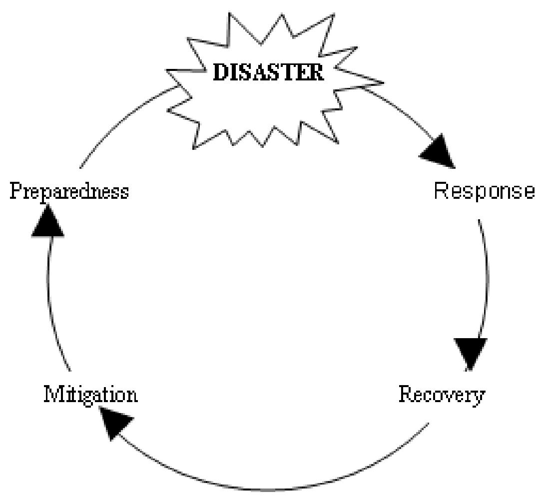

Figure 1.2: Emergency management Cycle

1.5 STATUS OF EARTHQUAKE MONITORING

Indian Meteorological Department (IMD) is the nodal agency dealing with measuring earthquakes in India. At present IMD is maintaining a network of 51 seismological observatories. Twenty-four seismological observatories were upgraded with broadband digital seismographs during 1996-99. A Central Receiving Station (CRS) and a National Seismological Data Base Centre (NSDC) have been established at New Delhi to receive analyze and systematically archive the seismic data.

1.6 CONCLUSIONS

India is prone to various types of natural hazards which pose threat to our buildings in different ways. The losses caused by them every year are continuously mounting and hence protection of buildings from these hazards to minimize their disastrous effects is need of the time.

Different types of measures have to be taken at the building design stage in order to increase there resilience to different hazards, which they are prone to. Disasters gives rise to emergency situations which have response, recovery, mitigation, preparedness and disaster as cyclic phases. Lot of research is being conducted in different aspects of disasters especially earthquakes owing to their peculiar un-predictive nature. Guidelines for the construction of earthquake resistant buildings have been provided in this book.

CHAPTER 2

ELEMENTARY SEISMOLOGY

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ELEMENTARY SEISMOLOGY

2.1 INTRODUCTION

An earthquake is a spasm of ground shaking caused by a sudden release of energy in the earth's lithosphere. As most earthquakes arise from stress build –up due to deformation of the earth's crust, understanding of seismicity depends heavily on aspects of geology, which is the science of earth's crust, and also calls upon knowledge of the physics of the earth as whole i.e. geophysics. The particular aspect of geology which sheds most light on the source of earthquakes is tectonics, which concerns the structure and deformations of the crust and the processes which accompany it; the relevant aspect of tectonics is referred to as seismo-tectonics. The chapter deals with basic understanding of all these fields for better understanding of earthquakes.

2.2 EARTHQUAKES

<u>Earthquakes are defined as, 'Ground shaking and radiated seismic energy caused mostly by sudden slip on a fault, volcanic or any sudden stress change in the earth'.</u>

2.2.1 Important Definitions

Some of the important definitions involved in the understanding of seismology are given below:

<u>Epicenter:</u> It is the point on the (free) surface of the earth vertically above the place of origin (hypocenter) of an earthquake. This point is expressed by its geographical latitude and longitude. (Fig 2.1)

<u>Hypocenter or Focus:</u> It is the point within the earth from where seismic waves originate. Focal depth is the vertical distance between the hypocenter and epicenter. (Fig 2.1)

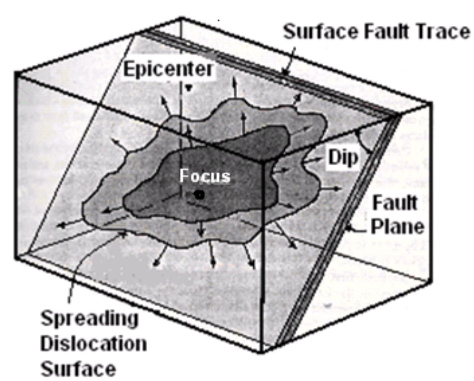


Figure 2.1: Origin of an earthquake

<u>Magnitude:</u> It is the quantity to measure the size of an earthquake in terms of its energy and is independent of the place of observation.

<u>Richter Scale</u>: Magnitude is measured on the basis of ground motion recorded by an instrument and applying standard correction for the epicentral distance from recording station. It is linearly related to the logarithm of amount of energy released by an earthquake and expressed in Richter Scale.

<u>Intensity:</u> It is the rating of the effects of an earthquake at a particular place based on the observations of the affected areas, using a descriptive scale like Modified Mercalli Scale.

2.2.2 Earthquake Occurrence in the World

All places on the earth are not equally seismic. Earthquakes are generally found to occur along specific regions called 'Seismic Belts'. There are three main belts around the globe along which majority of earthquakes have occurred. They are:

1. Circum Pacific Belt or Ring of Fire;
2. Alpide Belt and
3. North and South in the Middle of the Atlantic Ocean.

On a global scale, the present day seismicity pattern of the world is illustrated in general terms by the seismic events plotted in Fig 2.2

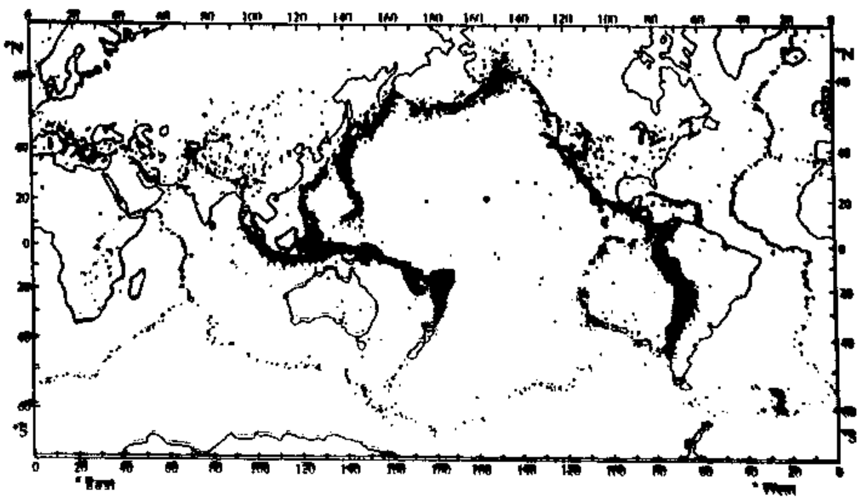


Figure 2.2 Map showing global distribution of earthquake epicenters.

2.2.3 Earthquake Occurrence in India

In India, the main seismic zone runs along Himalayan mountain range, northeast India, Andaman-Nicobar islands and Rann of Kutch region. Fig 2.3 shows the earthquake distribution in and around India and table 2.1 shows some recent significant earthquakes in India.

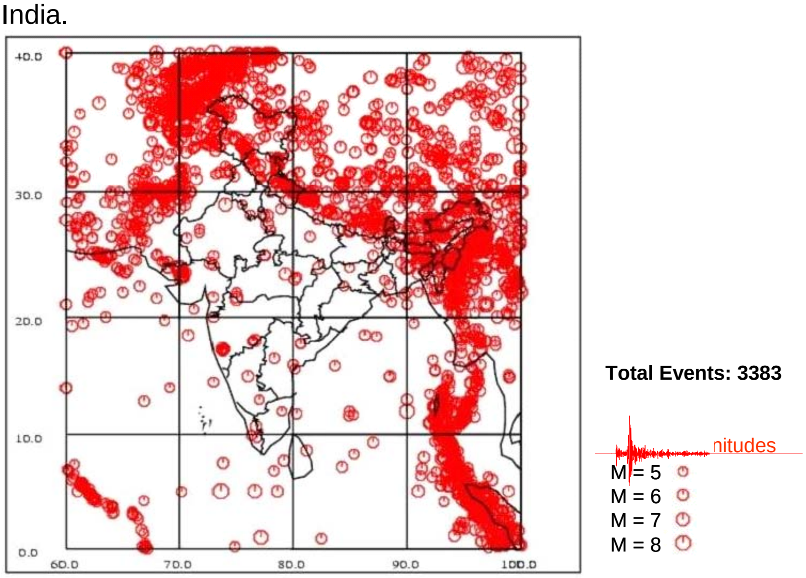


Figure 2.3 Plot of Earthquakes (M>= 5.0) From IMD Catalogue for the period from 1800 to Sept, 2001

Table 2.1: Recent Significant Earthquakes

Year	Place	Magnitude	Lives Lost
2005	Pakistan	7.4	1,309 in India
2004	Sumatra	9.3	10,749 in India
2001	Bhuj	7.7	13,805
1999	Chamoli	6.8	103
1997	Jabalpur	6.0	39
1993	Latur	6.3	7,601
1991	Uttarkashi	6.6	769
Lives lost in last about 15 years			34,375

2.2.4 Plate Tectonics

The theory of plate tectonics was originally proposed in 1912 by a German scientist, A. Wegner. Plate Tectonics is the theory supported by a wide range of evidence that considers the earth's crust and upper mantle to be composed of several large, thin, relatively rigid plates that move relative to one another. It is based on some theoretical assumptions that explain the forces, which cause accumulation of stresses inside the earth (Fig 2.4). These assumptions are as given below:

- Drifting of continents and mountain building process
- Shortening of Earth’s crust due to cooling and contraction.
- Disturbance of mass distribution on the Earth’s surface as a result of erosion of high lands and deposition of sediment in the sea.
- Generation of heat by radioactive material inside the Earth’s crust.

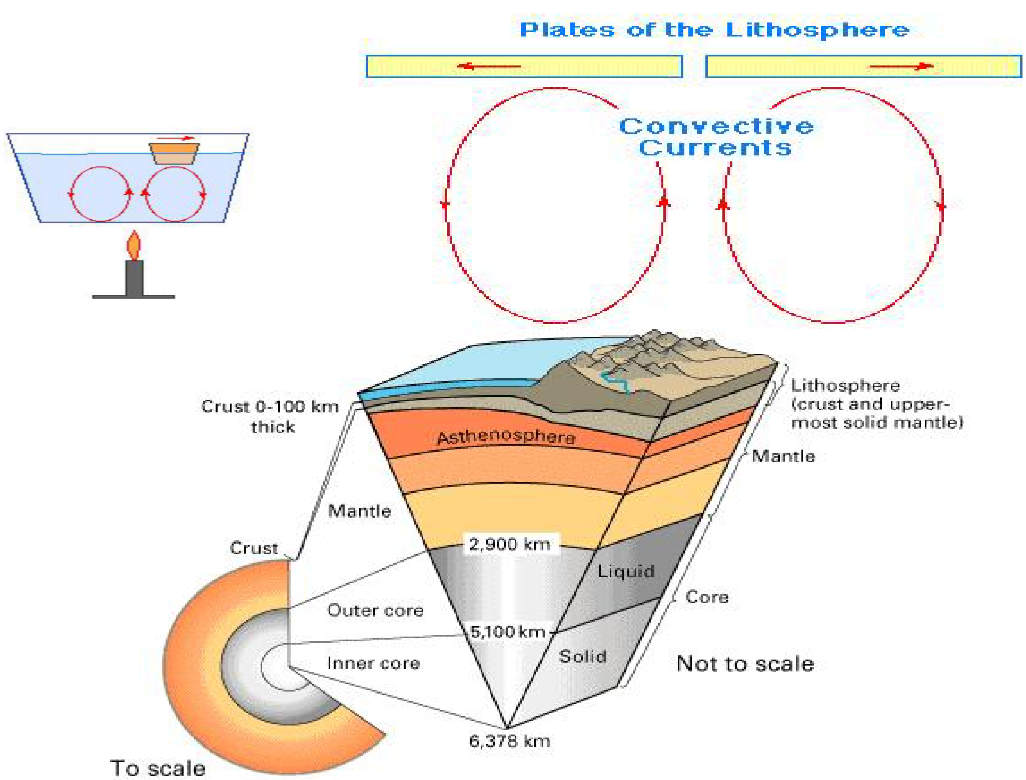


Figure 2.4: Theory of Plate Tectonics

The edges of the oceanic and/or continental plate boundaries mark the regions of destructive earthquake activity and volcanic activity. Fig 2.5 shows the plate boundaries across the globe.

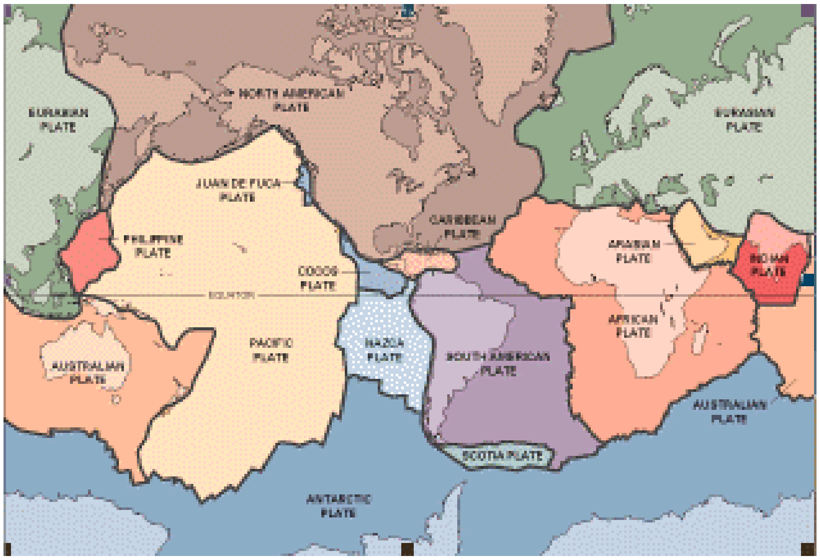


Figure 2.5: Plate Boundaries

About 80% of the seismic energy is released by earthquakes occurring along the plate boundaries. These earthquakes are called as inter-plate earthquake, directly associated with forces related to the interaction of the plates. (Circum-Pacific belt, Mid-Atlantic ridge and Alpine- Himalayan belt)
Sporadically, earthquakes also occur at rather large distances from the respective plate margins, these so called intra-plate earthquake, show a diffuse geographical distribution. (Central USA: New Madrid, 1812, Northeastern Continental China: Tangshan, 1976 and central India: Latur, 1993).

Earthquakes are usually caused when the underground rocks suddenly break along a plane of weakness, called fault.

Types of Plate Boundaries

There are three types of plate boundaries as explained in the Fig 2.6 to 2.8 given below:

1. Convergent Plate Boundaries

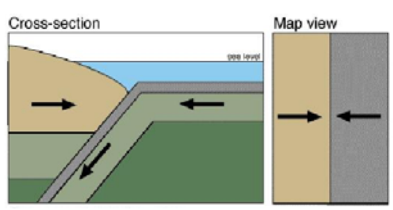


Fig 2.6 a

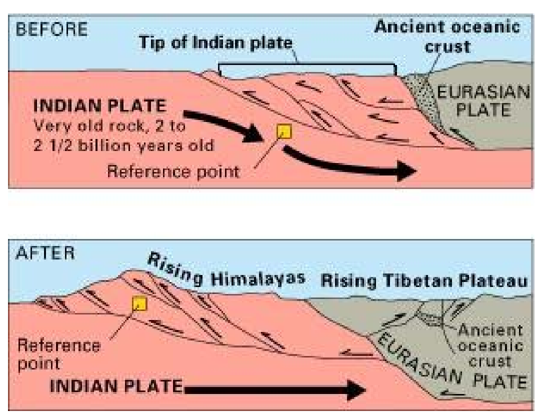


Figure 2.6 a: Convergent Plate Boundaries
Figure 2.6 b: Example of Convergent Plate Boundaries

Figure 2.6 b

2. <u>Divergent Plate Boundaries</u>

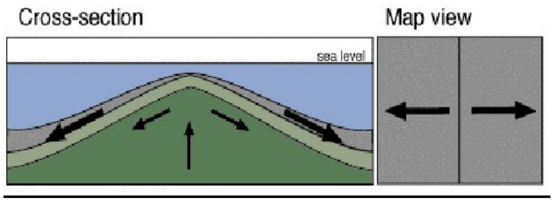


Figure 2.7: Divergent Plate Boundaries

3. <u>Transform Plate Boundaries</u>

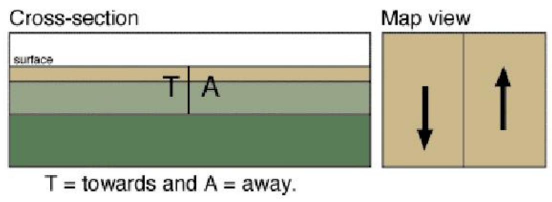


Figure 2.8: Transform Plate Boundaries

2.2.5 Faults

A fault is nothing but a crack or weak zone inside the Earth. When two blocks of rock or two plates rub against each other along a fault, they don't just slide smoothly they stick a little.
As the tectonic forces continue to prevail, the plate margins exhibit deformation as seen in terms of bending, compression, tension and friction. The rocks eventually break giving rise to an earthquake, because of building of stresses beyond the limiting elastic strength of the rock.
The building up of stresses and subsequent release of the strain energy in the form of earthquake is a continuous process, which keeps on repeating in geological time scale.

Types of Faults:

Different types of faults are (Fig 2.9):

- Dip Slip Faults;
- Normal;
- Reverse;
- Strike Slip Faults;
- Right Lateral and
- Left Lateral.

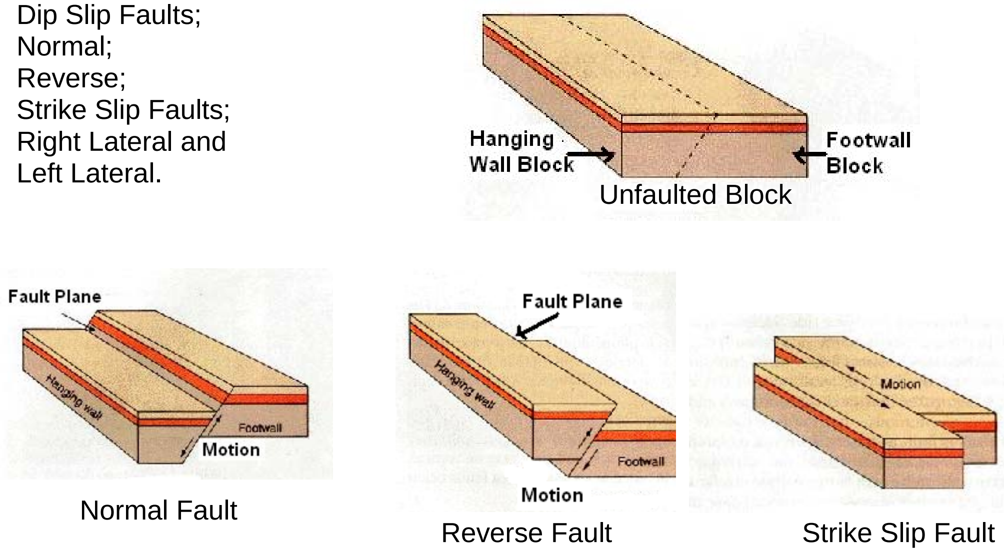
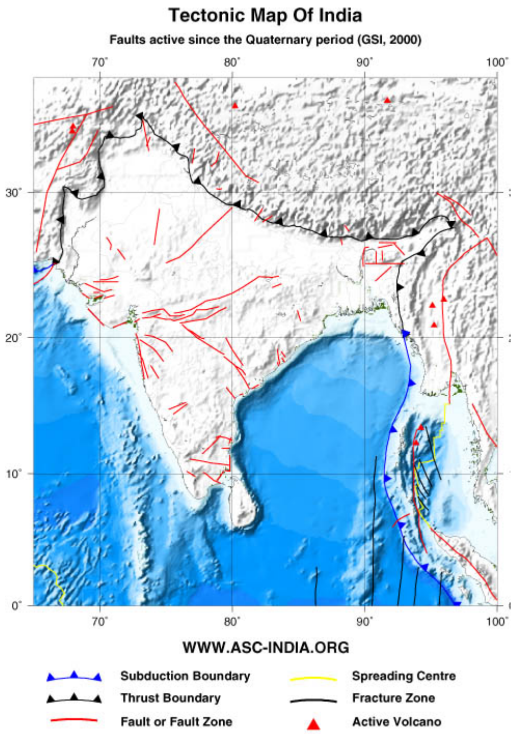


Figure 2.9: Types of Faults

Fig 2.10 shows the tectonic map of India with known fault planes.



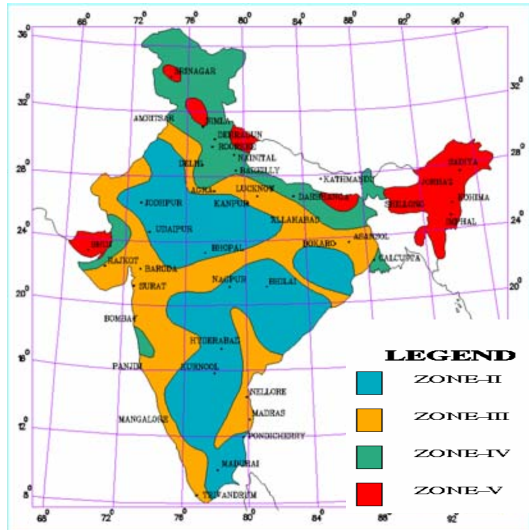


Figure: 2.10: Tectonic map of India showing known fault planes

Figure: 2.11: Seismic Zoning map of India - 2002

2.2.6: Earthquake Hazard maps

Under the initiative of the Ministry of Urban Development, a Vulnerability Atlas of India was prepared in which the earthquake, cyclone and flood hazard maps for every state and Union Territory of India have been prepared to a scale of 1:2.5 million. The seismic zoning map was periodically updated and the latest (2002) map is shown in fig 2.11.

2.3 SEISMOLOGY

The term 'Seismology' (Science of Earthquakes) is derived from Greek word Seismo, which means earthquake and logos which means science; hence the Seismology is Science of Earthquakes

Seismology can be defined in two ways:

1. The science of earthquakes and the physics of the earth's interior
2. The science of elastic wave (seismic waves) i.e.
    a) Their origin (earthquakes, explosions etc.)
    b) Their propagation through the earth's interior and
    c) Their recording, including the interpretation of records

The elastic waves, emanating from an earthquake permit the most reliable studies and conclusions about the internal constitutions of the earth using the records of the seismograph stations around the world. Source parameters of earthquake help us to evaluate the tectonic force.

In addition there is **applied seismology,** where we can also distinguish between several branches, such as seismic prospecting, i.e., the search by seismic method for economically significant occurrences of salt, oil, mineral ores. Furthermore, depth to bedrock measurements for construction purposes, etc. The problem of distinguishing between earthquakes and explosions can be considered as another branch of seismology.
In brief, seismology deals with the following:

- The practical problem of understanding, reacting and living with earthquakes
- The use of earthquakes and other natural excitations of the earth to understand the nature of the terrestrial forces involved and the structure of earth
- The technology of seismic prospecting

2.3.1 Causes of earthquakes

An earth shaking may occur due to various reasons: tectonic plate movements, volcanic activity, impact of meteorites, collapse of caves, rock-burst in mines, land slides/ rock-falling, nuclear explosion etc.
<u>An earthquake is a phenomenon related to **strong vibrations** occurring on the ground due to sudden release of energy.</u>

2.3.2 Classification of Earthquakes

Most earthquakes originate within the crust. At depth beneath the Moho, the number falls abruptly and dies down to zero at a depth of about 700 km.

Classification based on focal depth

1. Shallow-focus: Shallow-focus earthquakes, which constitute about 80% of total activity, have their foci at a depth between 0 to 70 km and occur at oceanic ridges, collision and subduction zones and transform faults
2. Intermediate focus: Intermediate-focus earthquakes (focal depth between 71 and 300 km) and
3. Deep-focus: Deep-focus earthquakes (focal depth greater than 300 km) occur at subduction zones

Classification based on magnitude

Classification	**Magnitude** (on Richter Scale)
Micro earthquake	less than 3.0
Slight	3.1-4.9
Moderate	5.0-6.9
Great	7.0-8.0
Very Great	Greater than 8.0

Classification based on epicentral distance

Classification	**Range**
Local shock	< 4.0
Near shock	4.0 to 10.0
Distant shock	10.0 to 20.0
Teleseismic shock	> 20.0

2.3.3 Earthquake Size

There are two methods of describing how large an earthquake is, as given below:

1. <u>The intensity of an earthquake</u>: It is a subjective parameter that is based on an assessment of visible effects. It is therefore depends on factors other than the actual size of the earthquake.
2. <u>The magnitude of an earthquake</u>: It is determined instrumentally and is more objective measure of its size.

Intensity:

Intensity is the rating of the effects of an earthquake at a particular place based on the observations of the affected areas, using a descriptive scale. Large earthquakes produce alterations to the Earth's natural surface features or severe damage to the man-made structures such as buildings, bridges and dams. Even small earthquakes can result in disproportionate damage to the edifices when inferior construction methods or materials have been utilized. The intensity of earthquake at a particular place is classified on the basis of the local character of the visible effect it produces. Various types of scales have been developed for the classification of intensity, one originally proposed in its original, known as Modified Mercalli (MM) scale is in common use (Annexure I). In map intensity is represented by lines representing equal intensities called as isoseismals. (Fig 2.12)

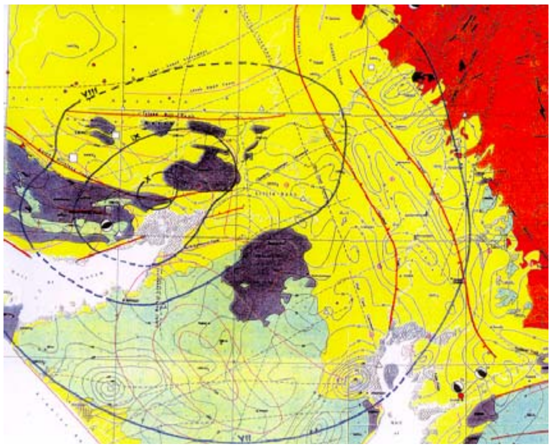


Figure 2.12: Isoseismals drawn for Gujarat Earthquake (January 26, 2001)

Magnitude:

The strength of an earthquake or strain energy released by it is usually measured by a parameter called 'magnitude' determined from the amplitudes and periods of seismic waves of different types.

A magnitude is a logarithmic measure of size of an earthquake or explosion base on instrument measurement.

Depending upon the level of magnitudes, epicentral distance and the characteristics of seismographs, there are mainly four magnitude scales in use. They are:

- Local (Richter) magnitude (M_L)
- Body Wave magnitude (m_b)
- Surface wave magnitude (M_S)
- Moment Magnitude (M_W)

2.3.4 Seismic waves

We know that, sudden release of energy causes an earthquake. Part of energy released during an earthquake, at its origin, fractures the rock in that region. The rest travels away from the focus in all directions in the form of elastic waves. These are called seismic waves. The velocity of propagation of these waves depends upon the density and the elastic properties of the medium through which they travel. Different types of seismic waves are described below:

Body Waves

Those waves that travel through rocks are called body waves. Body waves are of two kinds, longitudinal and transverse.

Longitudinal waves: these are sometimes referred to as P waves, or primary waves or push waves. As the wave advances each particle in the solid medium is displaced in the direction of motion of these waves, as in the case of sound waves. (Fig 2.13 a)

Transverse waves: these are also known as S waves or secondary waves. These are like ripples observed in a pond. The particle motion within the transmitting medium is at right angles to the direction of wave propagation. (Fig 2.13 b) For example, the ripples one observes when a stone is thrown in a pond. If a cork is placed in water it moves up and down while the wave travels at right angles to the cork movement. So in any medium the longitudinal wave travels faster than the transverse wave and hence at any point of observation one first observes longitudinal waves. And as liquids do not have any rigidity, the transverse waves cannot travel through them.

Surface Waves

Those waves that travel on the surface of the earth or elastic boundaries are called surface waves. They travel only at the surface or at the boundary of two different media and not into earth's interior. Two types of surface waves are Love waves and Rayleigh waves as shown in fig 2.13 c Fig 2.13 d.

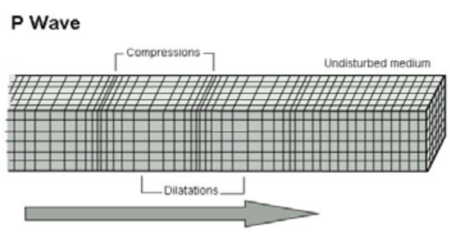


Fig 2.13 a: P Wave

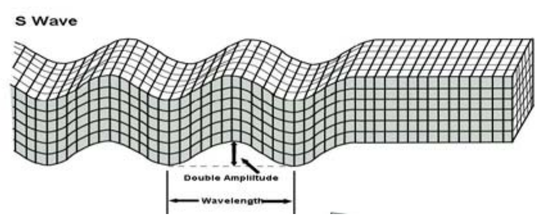


Fig 2.13 b: S Wave

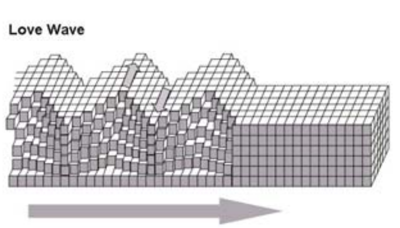


Fig 2.13 c: Love Wave

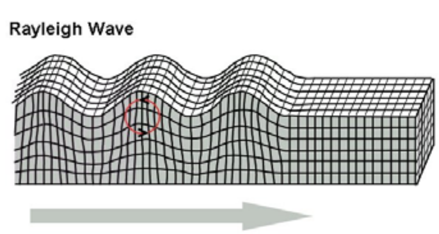


Fig 2.13 d: Rayleigh Wave

Fig: 2.13: Seismic Waves

2.3.5 Energy release

The energy released at the time of earthquake is not same for all earthquakes. Some earthquakes are so small that they can be detected only with the help of very sensitive instruments. However, the energy released at the time of a large earthquake is indeed enormous.
To measure the size of an earthquake, seismologists use the Richter magnitude scale. It is a measure of the total energy released during an earthquake and is determined by the maximum amplitude of recorded seismic waves, instrumentally recorded, plus an empirical actor that takes into account the weakening of seismic waves as they spread away from the focus. This logarithmic scale is expressed in Arabic numerals. If the magnitude is increased by a factor of one, the energy released is increased by a factor of 30. There is no longer limit to the magnitude but the upper limit seems to be about 8.9, as earthquakes with magnitude greater than this have not yet been recorded.

2.4 SEISMOLOGICAL INSTRUMENTS

Many instruments have been designed to measure ground shaking in detail. Some of them which are used in seismology are given below:

2.4.1 Seismograph

Elastic waves transmitted from a single earthquake can be recorded all over the world using earthquake recording instruments called seismographs.
The prototype of the modern seismograph was built in Japan about 100 years ago. Basically, the seismograph has a mass which is loosely coupled to the earth through a spring. The inertia of the mass keeps it fixed in position as the earth moves.
Modern seismographs are quite complex in the construction and can record very feeble ground motions which have traveled long distances. They can magnify the ground motion upto million times before recording it. A study of seismograms, that is, the records produced by seismographs, can yield information not only about the time and place of occurrence of an earthquake, but also about the rocks through which earthquake energy travels.

2.4.2 Accelerograph

Rate of change of velocity with time is known as acceleration and a strong motion earthquake instrument recording accelerations is called as accelerograph.
The record from an accelerograph showing acceleration as a function of time is accelerograms.

2.4.3 Seismoscope

The first earthquake recorder described in any detail was an artistic device invented by the Chinese scholar Chang heng about 132 A.D. Balls were held in dragons' mouths connected by linkages to a vertival pendulum. Shaking released the balls. The instrument was seismoscope, because unlike a seismograph, it could not give the complete time history of the earthquake shaking but simply the direction of the principal impulse due to earthquake.

2.5 CONCLUSIONS

Study of elementary seismology tells us the overall underlying level of seismic hazard which may differ from the available evidence of historical seismicity, notable in areas experiencing present day quiescent periods. India is highly prone to earthquakes and the main seismic zone runs along Himalayan mountain range, northeast India, Andaman-Nicobar islands and Rann of Kutch region**.**

The occurrences of earthquakes worldwide is best explained by theory of Plate Tectonics supported by a wide range of evidence that considers the earth's crust and upper mantle to be composed of several large, thin, relatively rigid plates that move relative to one another. The tectonic forces build up when two plates rub against each other along a fault which is a crack or weak zone inside the earth. The building of such stresses and subsequent release of the strain energy in the form of earthquake is a continuous process, which keeps on repeating in geological time scale

Earthquakes can be classified on the basis of focal depth, magnitude and epicentral distance. Size of earthquake can be measured in terms of magnitude and intensity. Many instruments like seismograph, acceleraograph and seismogram etc. have been designed to measure ground shaking in detail.

Although lot of research is being conducted in the field of prediction of earthquakes, till date there is no reliable mechanism for earthquake prediction. Hence earthquakes occur without warning and cause widespread damages and destructions.

CHAPTER 3

THEORY OF VIBRATIONS

CHAPTER 3
THEORY OF VIBRATIONS

3.1 INTRODUCTION

Vibration is a common phenomenon. Machines and engines in workshops or factories and household-appliances generate vibrations which often go unnoticed. Vibrations become more perceptible under unusual circumstances like strong gusts during turbulent winds, pressure waves due to explosions, operation of heavy machinery with moving parts, ground motions due to earthquakes, etc. These strong excitations have far reaching implications for the safe design of structures. Though the external excitations that lead to significant vibrations act only for a finite duration and that too only once in a while, structure has to be designed with consideration for them to avoid damage and collapse. Structures can be made to withstand these unusual dynamic loads with only an incremental increase in investment.

3.2 STRONG GROUND MOTION

Ground motion is reached to a building in different waveforms: Body and Surface. The ground motion can be quantified through different parameters like Acceleration, Velocity and Displacement. To determine forces and assessing relative seismic impact at different locations these are prerequisite.
Both the amount of ground motion and the time to develop the motion are critical to the structure. The amount of ground motion at any particular location is basically a function of the magnitude of the earthquake, the epicentral distance, and the site-soil conditions. The time-factor or frequency of the ground motion is basically a function of the characteristics of the earthquake waves.

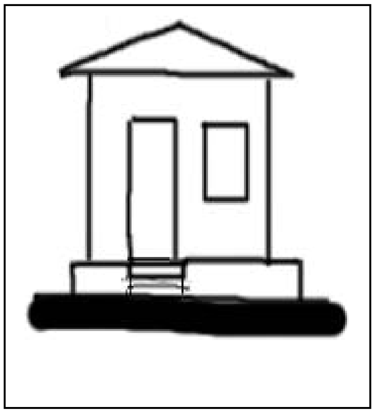
Understanding of effects of ground motion on a building would explain the building's response for the seismic force.

Due to ground shaking, buildings are subjected to inertia force, which is described here:

3.3 INERTIA FORCE

Building's mass is one of the most important parameter as vibration of the building's mass generates inertial forces while damaged by shaking. This leads to common saying that Earthquake load is generated by the structure itself.

Hence, Inertia forces are the product of mass and acceleration, i.e. F = M X A.



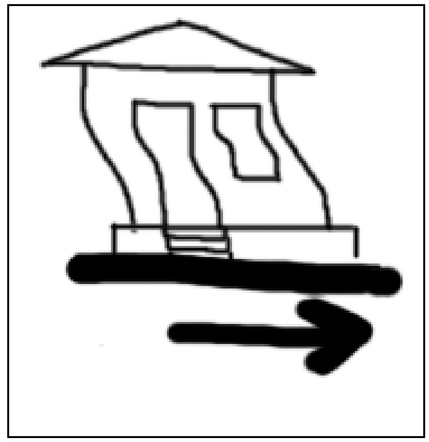


Figure 3.1: Effect of inertia in a building when shaken at base

Earthquake causes shaking of the ground. So a building resting on it will experience motion at its base and the roof has a tendency to stay in its original position because of inertia. If the building were rigid, then every point in it would move by the same amount as ground. In normal building the walls and columns are flexible and hence the motion of roof is different from that of the ground (Figure 3.1).

There is another impact of this inertia force. As the building bents or moves, vertical elements like column and wall moves out of plumb and get subjected to buckling. Thus they loose their capacity to carry gravity load as they are eccentrically loaded now and their load path assumption has changed. This displaces the vertical loads from P to P_1 (Fig 3.2). Extreme displacement can cause collapse. This is to remember that a structural system, by definition, is required to provide a safe path to transfer the externally applied loads to the foundation level.

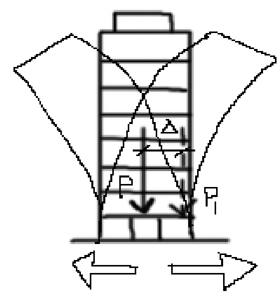


Figure 3.2: P- Δ Effect of inertia in a building as a result of drift Δ

<u>While this load path is available naturally for the loads applied vertically, (as in case of gravity loads like dead loads), a safe load path has to be provided for the loads applied horizontally or laterally.</u>

3.4 STRUCTURES AS PERSONALITIES

Every structure has a distinct personality. In the parlance of theory of vibrations we term those as the natural period(s) and damping. The natural periods (or natural frequencies, being the reciprocal of natural period) are by far the most important set of personality traits that distinguish a structural system. The significance of these natural frequencies can be ascertained from the fact that at only this finite number of frequencies, a simple harmonic motion of entire structure is possible.

3.5 NATURAL PERIOD

To understand Natural Period of a building, a simple example can be cited – that is of a pendulum. A pendulum is a heavy bob suspended from a (supposedly) weightless string, as shown in Fig 3.3.

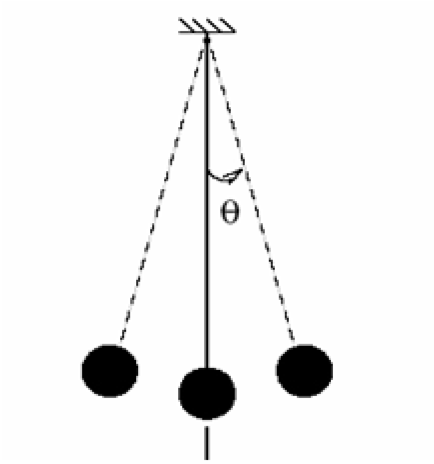


Figure 3.3: A pendulum

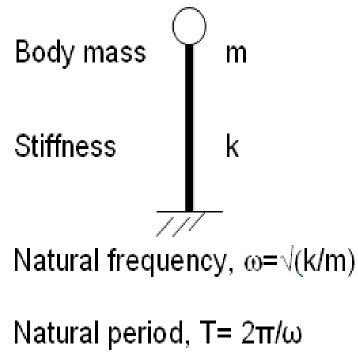


Figure 3.4: Natural Period, T

The pendulum is set into vibratory motion (simple harmonic motion, in this case) by releasing the bob from one of the extreme positions with no slack in the string. As the bob goes from one extreme position, crosses the mean position, goes to another extreme position, comes back towards the mean position, and then goes to the first extreme position again, it completes one cycle or oscillation. The time taken to complete one cycle/ oscillation is known as the time period or **natural period,** T of the pendulum (Fig. 3.4).

Natural frequency, $\omega = \sqrt{(k/m)}$ …(3.1)

Natural period, $T = 2\pi/\omega$ …(3.2)

The natural period depends on the length of the string with which the bob is suspended. It was demonstrated first by Galileo and as the name suggests, the natural period is a characteristic of a dynamical/vibratory system. At either of the extreme positions, the potential energy of the bob is at its maximum, while its kinetic energy is zero. As the bob reaches its mean position, the kinetic energy is at its maximum with potential energy reducing to zero. This interchange in the form of energy continues throughout the vibration.

This is similar for different structures too. Typically tall, slender structures have longer natural periods and are more flexible in comparison with the shortly squat structures, which are characterized by short natural periods and are relatively stiffer (Fig 3.5). Though this analogy is a little too simplistic, it does help to elucidate the core issue, i.e. changes in dynamic characteristics of structures with configuration plan and elevation.

Due to these differences in natural characteristics it is important to avoid any coupling of such vastly different structural types. If such a coupling cannot be avoided then it is necessary to account for their different dynamic characteristics in the structural design calculations.

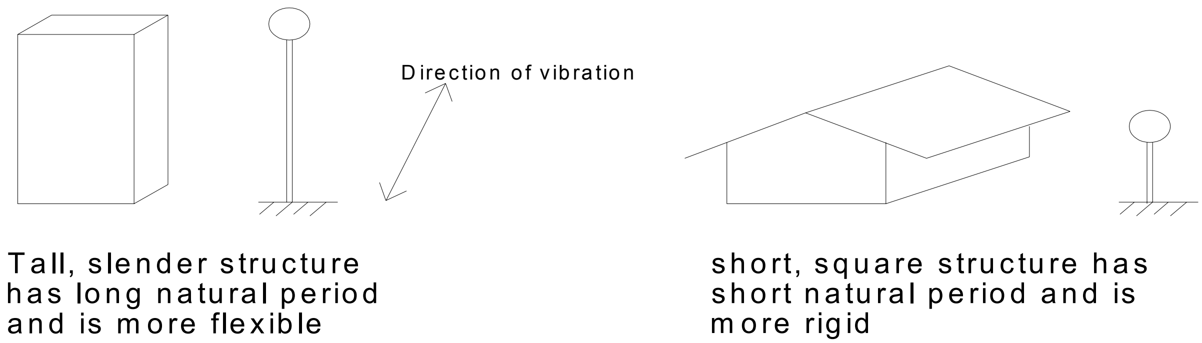


Figure 3.5: Flexibility of long and short period structures

Natural Period of any building primarily depends on height. Other than height attributor, construction material, building configuration and proportioning influence any building's time period.

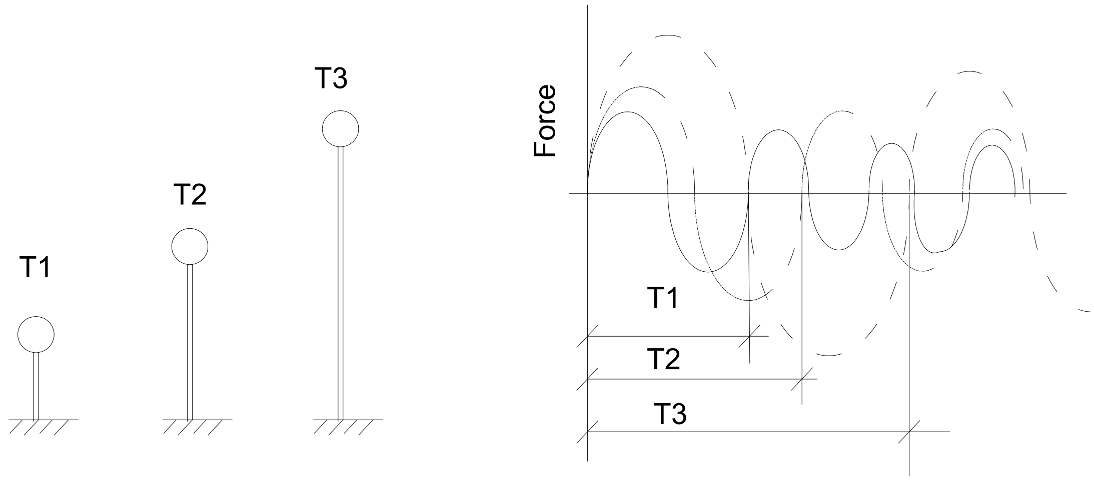
As a rule of thumb, T can be calculated as Building storey number divided by 10.

3.6 RESONANCE

If Natural Period of any vibrating structure matches with ground natural period (which is also shaking at its natural period), then vibrations would be amplified and would result in resonance in the structure.

The very act of tuning a radio receiver to a particular radio station (identified by the broadcast frequency) is an example of resonance. Here, the dynamic characteristic of receiver set is changed by means of a tuner (a variable capacitor) so that the natural frequency of the internal *LRC* circuit/oscillator is tuned to the frequency at which that particular radio station is broadcasting the signals. This situation represents the case of receiver oscillator being excited by the radio signals of the same frequency as the natural frequency of the oscillator resulting into a multi-fold amplification of the input signal. The radio signals of other frequencies broadcast by other stations are also present in the environment but the receiver listens to only those signals, which are of the same frequency as the natural frequency of the receiver. The signals of other frequencies are not amplified, as the receptivity of the receiver for other frequencies is very low.

The situation of a structural system subjected to dynamic loads is quite similar to that of a radio receiver. Fig 3.6 shows three structural systems with different natural periods being excited by an identical dynamic load. The condition of resonance occurs if the natural period T_n of a structural system coincides with the time period T_S of the excitation. In such case, the structure with same natural period would experience large amplitude vibrations and will probably not survive those vibrations unless specifically designed to care of this eventuality. The response of structures with natural periods different from the time period of the excitation is not alarming as the receptivity of these structures at the frequency (ω) of the applied loading is low.



Different types of structures

Resonance would be there when ground natural period, Ts matches with T1, T2, T3 or Tn.

Figure 3.6: Resonance in structures

<u>Resonance may occur to a structure if periods of both the structure and ground vibration matches, which is unlikely</u>

The ground vibration depends on nature of soil. For a soft-clay ground, period is as high as 2 seconds. Stiffer soil may end up with a vibration of 0.4 seconds. So if the natural period of a building is more or less equal with the ground soil, even a mild intensity shaking would result in large building response.

3.7 DAMPING

Every structure is blessed with inherent quality termed as damping. This is nothing but the same quality which stops the half-door at the entrance of your principal’s room. If we don’t provide a hinge-stopper, the door would swing for an indefinite period. Damping helps in attenuating the shaking vibration induced in the structure. It is expressed as % and represented by ξ. (See section 6.3.8)

3.8 SINGLE DEGREE OF FREEDOM (SDOF) STRUCTURE

The dynamic excitation of structures under earthquake can be considered in a simple manner consisting of a mass on a spring and under vibration, elastic or inelastic response of the structure can be assessed. This is referred as Single- degree-of-freedom system (SDOF) as shown in Fig 3.7.

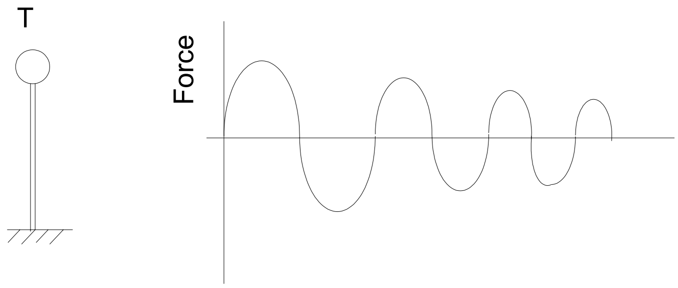


Figure 3.7: Damping of a structure diminishes the excitement

The dynamic characteristics of such system are simply described by its natural period of vibration T (or frequency, ω) and its damping ξ. It can be analyzed as subjected to harmonic base motion or a more complex irregular and transient excitation of earthquake.

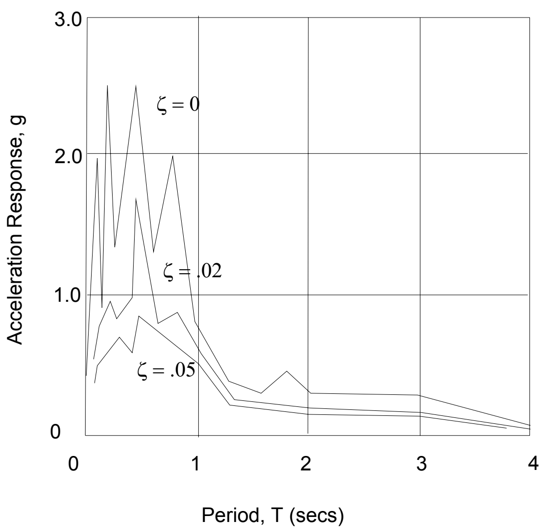


Figure 3.8: Elastic acceleration response spectra of north-south component of the 1940 El Centro earthquake

The Figure 3.8 can be studied as an example. For a SDOF structure with a period of 0.8s and damping ξ = 0.02 has a maximum acceleration of approximately 0.9g compared with a peak input ground motion of about 0.33g (from accelerogram records). This represents an amplification of 2.7 times at ξ = 0.02, whereas if the damping ξ = 0.05 the amplification can be seen to reduce to 1.8.

3.9 Multi-degree-of-freedom (MDOF) Systems

In the dynamic analysis of structures assumptions made in SDOF is no longer valid. Discrete lumps of masses are assumed at the levels of different floors, which would be subjected to lateral displacements now. In Fig 3.9 a case of a three storeyed building has been illustrated to discuss concepts lying behind multi-degree-of-freedom systems.
Each storey-mass represents one-degree- of- freedom with individual dynamic equilibrium.

Fundamental Time Period of any structure refers to the Time period of multi-degree-of-freedom systems at first mode.
Fundamental time period of different scales of structures are given in Fig 3.10.

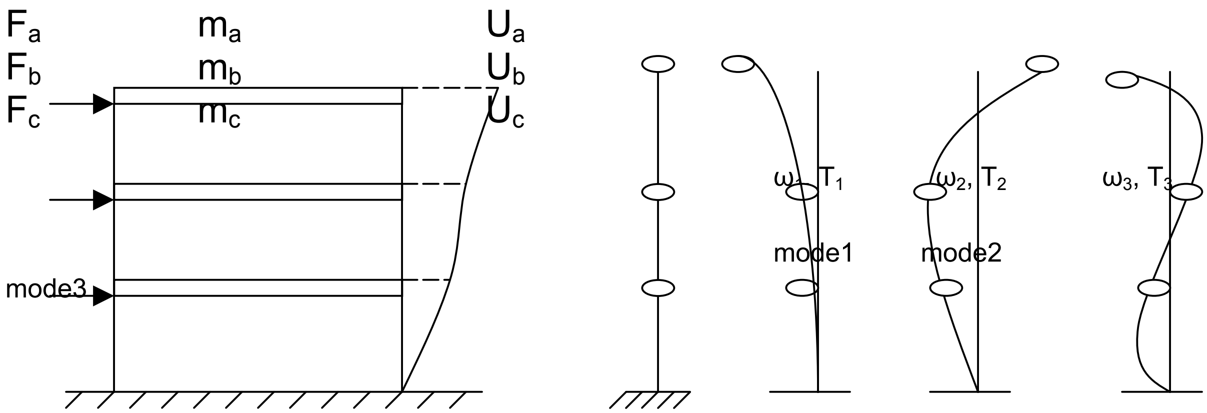


Figure 3.9: Multi-degree-of-freedom system subjected to dynamic loading

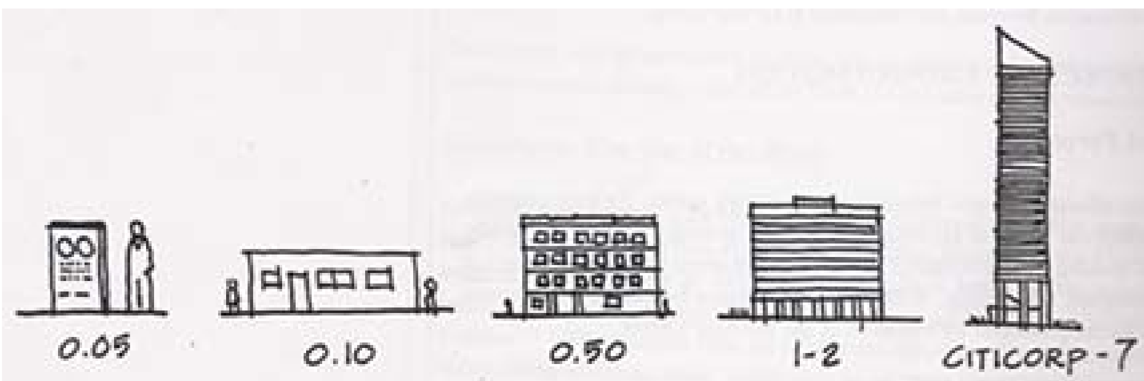


Figure 3.10: Fundamental Periods of different types of structures

3.10 METHODS OF SEISMIC ANALYSIS FOR SDOF SYSTEMS

There are many methods for determining seismic forces in structures; broadly they can be categorized in two divisions: equivalent static force analysis and dynamic analysis. In equivalent static force analysis, the “total” horizontal force (base Shear) V is determined on a structure with following equation:

$$V = ma \qquad \ldots(3.3)$$

Where, m is the mass of the structure
and, a is the seismic horizontal acceleration.

“a” is generally in the range of 0.05 to 0.20g.

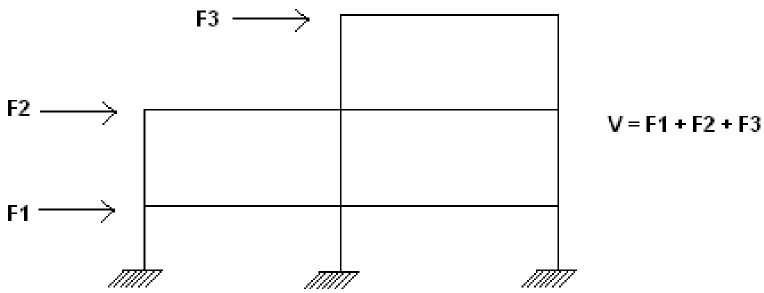
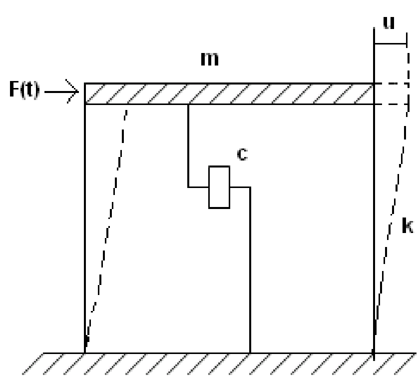


Figure 3.11: Frame with equivalent static forces applied at floor levels

An important feature of equivalent static load calculation method is that the calculated seismic forces are considerably less than those, which would actually occur, in higher earthquake risk-prone areas. (Fig 3.11)
Dynamic analysis involves different techniques like:

- direct integration of the equations of motion
- normal mode analysis
- response spectrum analysis.

In the Response Spectrum Analysis, modes of vibration are determined in period and shape. Maximum response magnitudes corresponding to each mode are found with reference to a response spectrum. An arbitrary rule is then used for superimposition of the responses in the various modes. The resultant moments and forces in the structure `correspond to the envelopes of maximum values, rather than a set of simultaneously existing values. The response spectrum method has the great virtues of speed and economy.



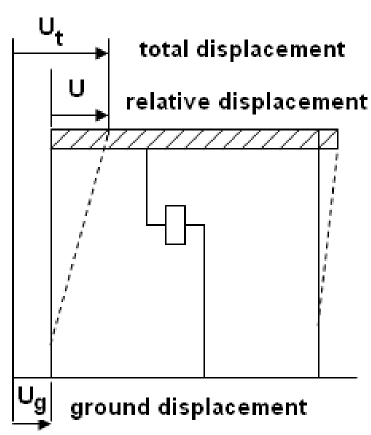


Figure 3.12 (a) Idealized SDOF system

Figure 3.12 (b): SDOF system subjected to ground motion

3.11 NDAMPED AND DAMPED SDOF

3.11.1 Seismic responses

Seismic responses of undamped and damped SDOF systems are different. In the Figure 3.12 (a), F(t) is a force varying with time, k is the total stiffness (spring) constant of resisting elements, c is the damping coefficient and u is the displacement.
Generally the equation of dynamic equilibrium is

$$\mathbf{F_I + F_D + F_S = F\ (t)} \qquad \ldots(3.4)$$

where the inertia force, $F_I = m\ddot{U}$; the damping force, $F_D = c\dot{u}$; and the elastic force, $F_S = ku$.

Thus,

$$\mathbf{m\ddot{U} + c\dot{u} + ku = F\ (t)} \qquad \ldots(3.5)$$

For the case of earthquake force as shown in Figure 3.12 (b), the only external loading is in the form of an applied motion at ground level, i.e. ground displacement u_g. Due to inertia, relative displacement of ground, u has occurred to the structure. This results in Total displacement of U_t, and

$$\mathbf{u_t = u + u_g} \qquad \ldots(3.6)$$

Thus, the total acceleration of the mass m is

$$\mathbf{\ddot{U}_t = \ddot{U} + \ddot{U}_g} \qquad \ldots(3.7)$$

3.11.2 Free vibrations (undamped)

In the case of free vibration system, these equations can be solved, and the result is a simple harmonic motion (SHM) as shown in Fig 3.13. Here u_0 and $\dot{u}_0$ are the initial displacement and velocity respectively.

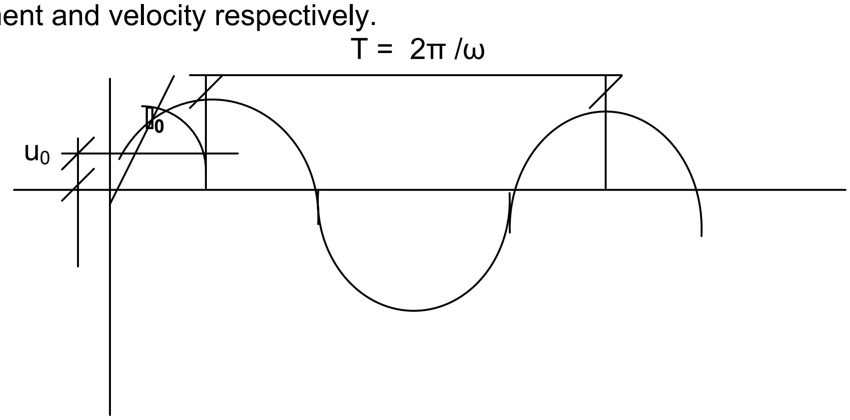


Figure 3.13: Undamped SHM of SDOF system given initial displacement and velocity

The period of the above motion is T = 2π /ω and the amplitude is $R = \sqrt{\{(\dot{u}_0 / \omega)\ 2 + u^2_0\}}$

3.11.3 Damped Vibration

The response of a damped SDOF structures to earthquake motion can be simplified and the effective earthquake force on the structure can be presented as :

$$\mathbf{Q(t) = m\ \omega\ V\ (t)} \qquad \ldots \quad (3.8)$$

Thus the effective earthquake force (or base shear) is found in terms of the mass of the structure, its circular frequency and the response function V(t) .

3.12 RESPONSE SPECTRA

Typically, in earthquake ground motions, the frequencies in the dynamic excitation can range from 0.5-25.0 Hz. Thus, it can be a very tedious exercise to calculate the maximum vibration response of a structural system to the applied excitation using different frequencies. The response of a structural system is governed by several waveforms in the applied excitation with frequencies in the close vicinity of the natural frequency of the structural system. The required maximum response can be computed in a routine manner by solving the governing differential equation of motion for the vibration. This can be a very tedious, even if a straight forward, exercise.

In order to simplify this and for a quick assessment of maximum effect of an applied dynamic load on a structure a graphical tool, known as the response spectrum, is used. In development of this tool, a set of different structures (or, oscillators with different natural periods) is selected. The vibration response for each oscillator of this set is calculated for the applied dynamic load. The absolute maximum value of the response of an oscillator is plotted with respect to its natural period as shown in Fig 3.14.

This maximum value is called the spectral velocity S_v, or more accurately the spectral pseudo- velocity because it is not exactly the maximum velocity of a damped system.

So, the maximum displacement or spectral displacement, $S_d = S_v / \omega$;

And the spectral acceleration (or, spectral pseudo-acceleration) $S_a = \omega S_v$.

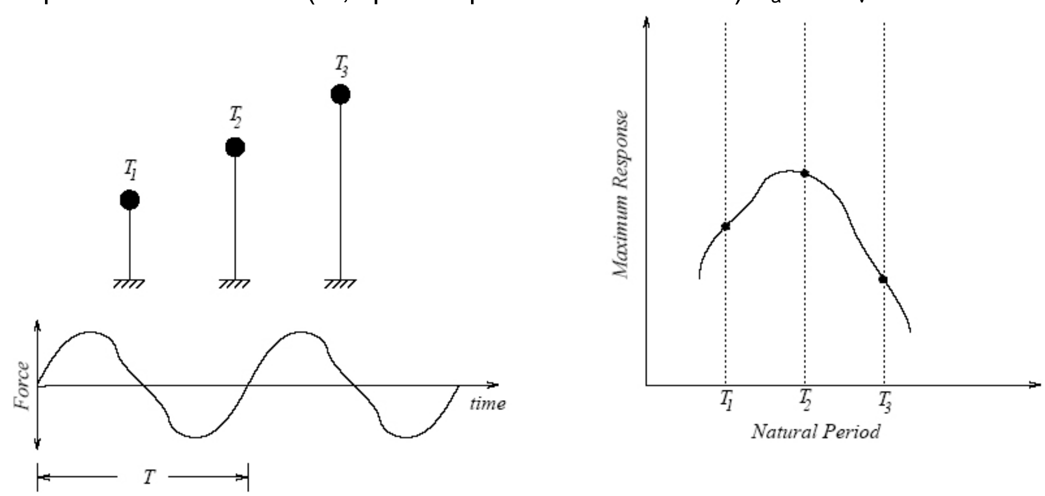


Figure 3.14: Response Spectrum

From these relationships, the maximum earthquake displacement response, $u_{max} = S_d$

And, the maximum effective earthquake force or base shear is $Q_{max} = m. S_a$

If these equations are evaluated for SDOF structures of varying natural periods, a maximum velocity response curve can be plotted. The curve resulting from joining these points on the graph is known as the *response spectrum.* Now the maximum response of any structure with natural period within the range of curve can be easily read from this graph. The response spectrum method is very convenient, for a quick estimation of the maximum response of a structural system to an applied dynamic excitation and is invariably used in all preliminary design calculations.

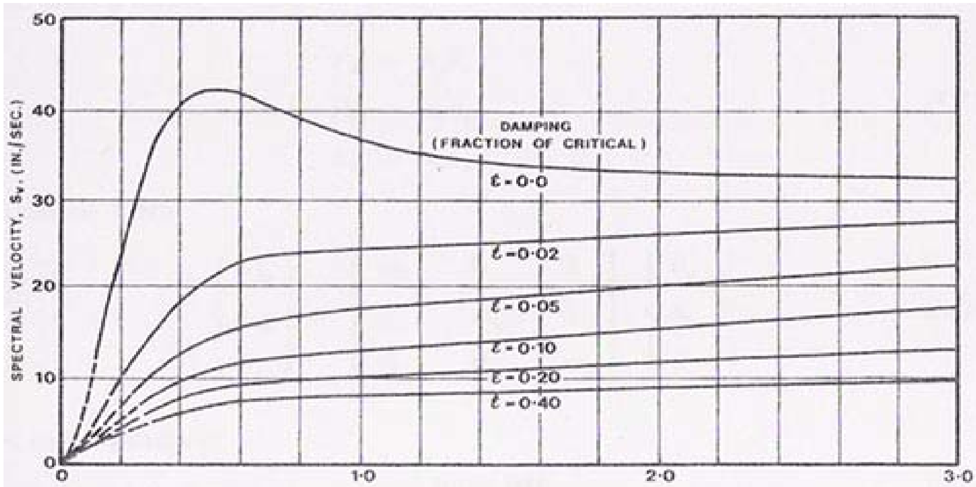


Figure 3.15: Averaged velocity response spectra

The velocity spectrum shown of the Fig 3.15 depicts averaged velocity response spectra based on the spectral intensity of the 1940 El Centro earthquake. For general design purposes an averaged spectrum as shown in the figure would be more appropriate.

3.13 CONCLUSIONS

This is forgoing text is a simplified compilation of the basic concepts of the dynamics of structures concentrating on bare essentials only. Several issues complicate the actual vibration analysis and a more rigorous, mathematical treatment is necessary to deal with those issues as desirable to seek an expert opinion on the dynamic behavior of structures at the planning stage itself.

This could lead to a considerable savings as against, trying to make a design conform to acceptable vibration performance levels at a later stage.

CHAPTER 4

PERFORMANCE OF GROUND AND BUILDINGS IN PAST EARTHQUAKES

CHAPTER 4
PERFORMANCE OF GROUND AND BUILDINGS IN PAST EARTHQUAKES

4.1 INTRODUCTION

Lives and properties of hundreds of millions of people throughout the world are at a significant risk due to earthquakes. Human settlements in highly seismic regions often display the influence of construction practices in less seismic areas, making them more vulnerable to earthquakes. Earthquakes have occurred from millions of years and will continue in future. It is impossible to prevent earthquakes from occurring, but it is possible to mitigate the effects of strong earthquake shaking to reduce loss of life and damage. Clearly, there is a need to recall lessons repeatedly taught by past earthquakes.

4.2 EARTHQUAKE EFFECTS ON GROUND AND SEA

During an earthquake, seismic waves radiate away from the source and travel rapidly through the earth's crust. When these waves reach the ground surface, they produce shaking. The strength and duration of shaking at a particular site not only depends on the size and location of the earthquakes but also on the characteristics of the site. Extent of damage caused by an earthquake depends on the level of ground shaking. At sites near the source of a large earthquake strong ground shaking can cause tremendous damage. Common earthquake damage on ground may be grouped into two categories:

Damages due to tectonic surface processes: includes ground surface fault rupture or simply surface rupture.

Secondary effects: which are defined as non-tectonic surface processes that are directly related to earthquake shaking. Examples of secondary effects are liquefaction, earthquake-induced slope failures, landslides and tsunamis.

Hence, the effects of earthquakes on ground (and sea) which involve surface rupture, liquefaction, landslides, and tsunamis are discussed in detail.

4.2.1 Surface Rupture

Most earthquakes will not create ground surface fault rupture. For example, there is an absence of surface rupture for small earthquakes and earthquakes generated at great depths. On the other hand, large earthquakes at transform boundaries will usually be accompanied by ground surface fault rupture on strike-slip faults. Figs. 4.1 and 4.2 show examples of surface fault rupture. Examples of very large surface fault rupture are the 11 m of vertical displacement in the Assam earthquake of 1897 and the 9 m of horizontal movement during the Gobi-Altai earthquake of 1957. The length of the fault rupture can be quite significant e.g. the estimated length of surface faulting in the 1964 Alaskan earthquake varied from 600 to 720 km.

Figure 4.1 Surface fault rupture associated with the El Asnam (Algeria) earthquake on October 10, 1980.

Figure 4.2 Surface fault rupture associated with the Izmit (Turkey) earthquake on August 17, 1999.

Surface fault rupture associated with earthquakes is important because it has caused severe damage to buildings, bridges, dams, tunnels, canals and underground utilities. There were disastrous examples of surface rupture associated with the Chi-Chi (Taiwan) earthquake on Sept. 21, 1999. With a magnitude of 7.6 the earthquake was the strongest to hit Taiwan in decades. The earthquake also triggered at least five aftershocks near or above magnitude 6. Surface fault rupture associated with this earthquake caused severe damage to civil engineering structures, as shown in Fig 4.3 to 4.6.

Figure 4.3: Overview of a dam damaged by surface fault rupture associated with the Chi-chi earthquake.

Figure 4.4: Close up view of the location of the dam damaged by surface fault rupture, shown in Fig 4.3

Figure 4.5: Close-up view of bridge pier (Wu-His Bridge) damaged by surface fault rupture-Chi-chi EQ.

Figure 4.6: Surface fault rupture and roadway damage associated with the Chi-chi earthquake.

January 26, 2001, Bhuj (India) earthquake was also a large magnitude (M_w =7.7) earthquake but no primary surface fault rupture was identified. Continuous geologic and geomorphic surfaces and cultural features provide evidence that no major fault rupture occurred in the epicentral area as a result of the Bhuj earthquake. Minor ground deformation, with a possible right lateral component of slip has been identified. Many ground failures reported are due to liquefaction related ground deformation.

4.2.2 Liquefaction

Most critical damage (due to earthquakes) on ground is when soil deposits have lost their strength and appeared to flow as liquids. This phenomenon is termed as liquefaction in which strength of soil is reduced, usually to the point where it is unable to support structures. Liquefaction occurs only in saturated soils, therefore most commonly observed near water bodies. It typically occurs in soil with a high groundwater table, its effects are most commonly observed in low-lying area or area adjacent to rivers, lakes, bays and oceans. Liquefaction phenomena can affect buildings, bridges, buried pipelines, and other constructed facilities in many different ways. Liquefaction phenomena can be divided into two main groups: flow liquefaction and cyclic mobility. Flow liquefaction can produce massive flow slides and contribute to the sinking or tilting of heavy structures, the floating of light buried structures and to the failure of retaining structures. Cyclic mobility can cause slumping of slopes, settlement of buildings, lateral spreading and retaining wall failure. In general the effects of liquefaction involves several related phenomena. For example, flow failures, lateral spreading, and sand boils, which are discussed in following paragraphs.

Flow failures:

Flow failures occur when the strength of the soil drops below the level needed to maintain the stability under static conditions. Flow failures are driven by static gravitational forces and can produce very large movements. Flow failures have caused the collapse of earth dams and other slopes, and the failure of foundations. Fig 4.7 to

4.9 shows examples of settlement and bearing capacity failures due to liquefaction during the Niigata (Japan) earthquake on June 16, 1964.

Figure 4.7: Kawagichi-cho apartment buildings suffered liquefaction-induced bearing capacity failure during the Niigata (Japan) earthquake on June 16, 1964.

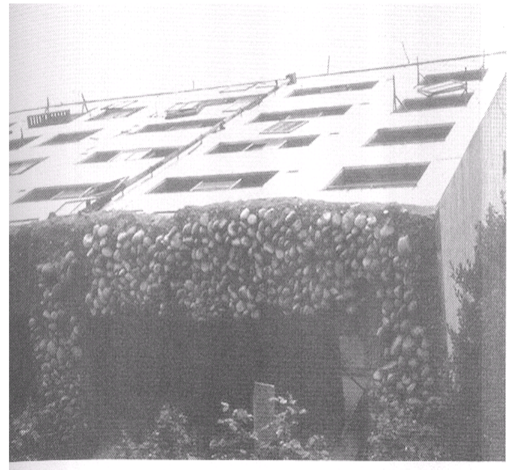


Figure 4.8: View of the bottom of a Kawagichi-cho apartment building suffered a liquefaction-induced failure during Niigata earthquake.

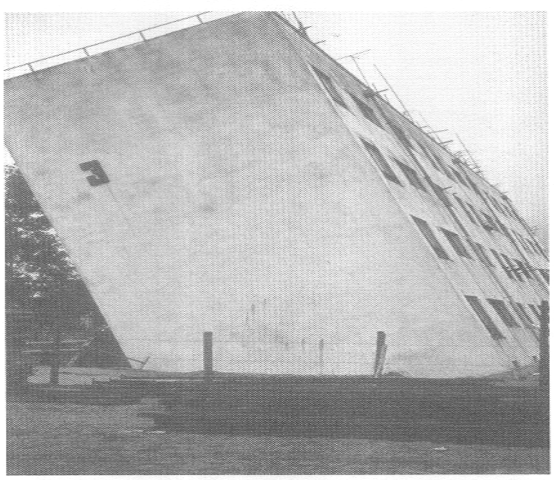


Figure 4.9: Liquefaction-induced settlement and tilting of an apartment building - Niigata earthquake

Damages to Waterfront Structures: Port and wharf facilities are often located in areas susceptible to liquefaction. The ports and wharves often contain major retaining structures, such as seawalls, anchored bulkheads, gravity and cantilever walls, and sheet-pile cofferdams, that allow large ships to moor adjacent to the retaining walls and then load or unload their cargo. Some spectacular examples of damage to waterfront structures due to liquefaction occurred during the (M_w = 6.9) Kobe earthquake (known as Hyogo-ken Nanbu earthquake) of January 17, 1995, are shown in Fig 4.10 to 4.14.

Figure 4.10: The interiors of Rokko and Port islands settled as much as 1 m with an average of about 0.5 m due to liquefaction caused by the Kobe earthquake. This liquefaction-induced settlement was accompanied by the eruption of large sand boils that flooded many areas and covered much of the island with sand boil deposits. In this photograph, a stockpile has been created out of this sand.

Figure 4.11: Ground cracks caused by lateral retaining wall movement due to liquefaction during the Kobe earthquake. The site is near Nishinomiya Port and consists of reclaimed land.

Figure 4.12: Damage on Port Island caused by retaining wall movement during the Kobe earthquake.

Figure 4.13: Collapse of a crane due to about 2 m of lateral movement of the retaining wall of Rokko Island during the Kobe earthquake.

Figure 4.14: Settlement caused by lateral retaining wall movement during the Kobe earthquake. The industrial building is supported by a pile foundation.

Lateral spreading:

Lateral spreading is a liquefaction related phenomenon characterized by incremental displacements during earthquake shaking. Depending on the number and strength of the stress pulses that exceed the strength of the soil, lateral spreading can produce negligible to quite large displacements. Lateral spreading is quite common near bridges and the displacements it produces can damage the abutments, foundations, and superstructure of bridges as shown in Fig 4.15a and 4.15b.

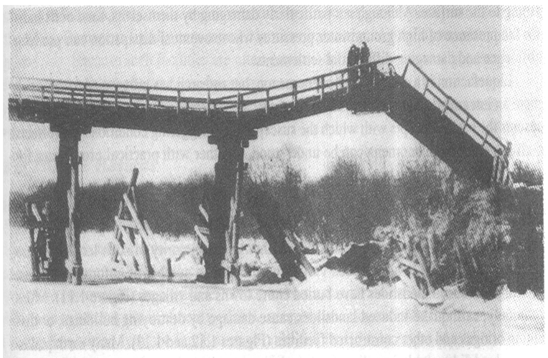


Figure 4.15a: Effect of lateral spreading on a small bridge in Japan following Tokachi-Oki earthquake of 1952. Lateral spreading of the soil at the abutment buckled the bridge deck.

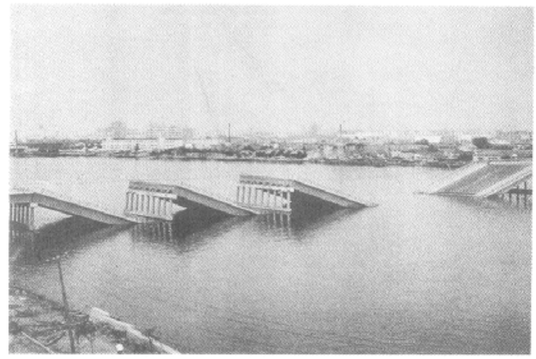


Figure 4.15b: The Showa Bridge following the 1964 Niigata earthquake. Lateral spreading caused bridge pier foundation to move and rotate sufficiently for simply supported bridge span to fall.

Sand boils

Sand boils produced by ground water rushing to the surface are present in the level-ground liquefaction, which does not involve large lateral displacements. Sand boils are not damaging by themselves but indicates the presence of high ground water pressures, whose eventual dissipation can produce subsidence and damaging differential settlements (Fig 4.16)

Figure 4.16 Sand boil in rice field following the 1964 Niigata earthquake.

Liquefaction during Bhuj earthquake (January 26, 2001)

The earthquake induced liquefaction and related ground failures over an area of greater than 15,000 square km. Surface manifestations of liquefaction include sand blows, sand blow craters, and lateral spreading. Areas where widespread liquefaction occurred include the Great Rann of Kachchh, Little Rann, Banni Plain, Kandla River and Gulf of Kachchh (Figs. 4.17 and 4.18). These areas contain low-lying salt flats, estuaries, intertidal zones, and young alluvial deposits (meizoseismal area), which are typically considered to have a very high susceptibility to liquefaction.

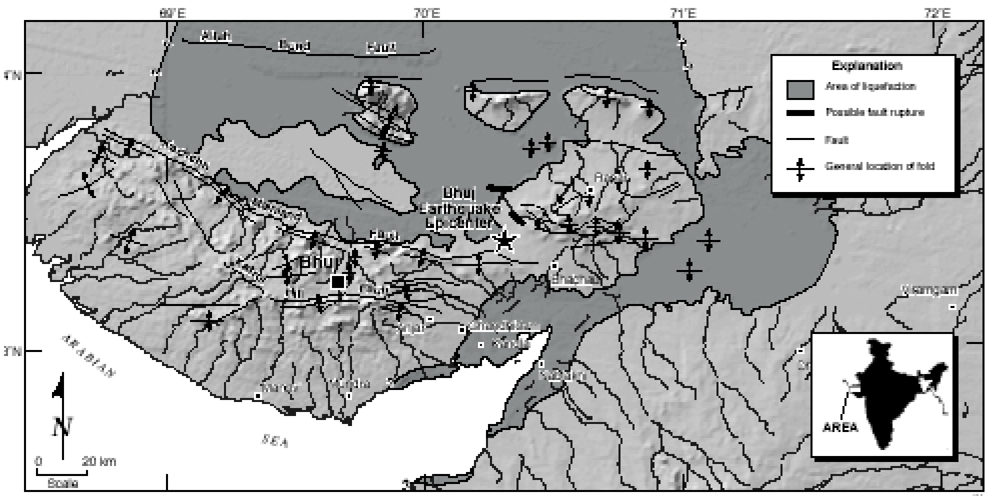


Figure 4.17: Map showing general distribution of liquefaction resulting from the Bhuj earthquake.

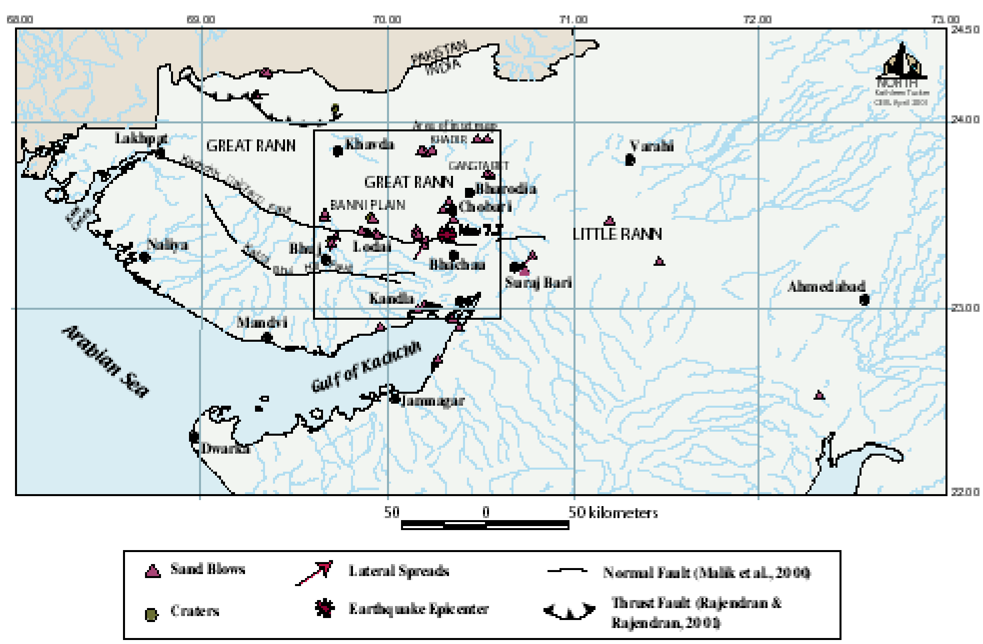


Figure 4.18: Map of the Kachchh region showing locations of documented liquefaction features relative to epicenter of the Bhuj earthquake.

According to many residents in the meizoseismal area, fountains of water ranging from 1 to 2 m in height formed during and immediately following the Bhuj earthquake. So much water vented to the surface in the Banni Plain and Great Rann that temporary streams flowed in previously dry channels. The surface water was so extensive that the media proclaimed the return of a mythical river, possibly the Sarasvati. Satellite imagery suggests that liquefaction may have occurred near Naliya and Lakhpat along the coast about 180 km west of the epicenter (Fig. 4.19). In addition there are reports of ground failure indicative of liquefaction as far away as the Sabaramati River south of Ahmedabad, about 240 km east of the earthquake epicenter.

Features Resulting from Subsurface Liquefaction (Bhuj Earthquake):

Sand Blows: Sand blows were the most common liquefaction features observed in the Bhuj earthquake. Sand blows are constructional cones of mostly sand vented with water to the ground surface through ground cracks (Fig. 4.19).

Sand-Blow Craters: These are characterized by constructional cones of vented sand and also have large central craters formed by the removal of surface soil and sediment (Fig. 4.20).

Lateral Spreading: It was common on gentle slopes (1 to 2°) in the epicentral area and along rivers and bays at greater distances (Fig. 4.21).

Figure 4.19: Moderate-size sand blow in Great Rann

Figure 4.20: Sand-blow craters in a dry riverbed

Figure 4.21: Parallel ground cracks in the epicentral area probably related to lateral spreading

Liquefaction-Related Damage to Facilities (Bhuj Earthquake):

Pipelines: Significant settlement of the backfill above a natural gas pipeline was observed over many km in a stretch of desert between the Little Rann and Great Rann, Fig. 4.22.

Figure 4.22: Settlement of backfill above natural gas pipeline

Bridges: A four span, two-lane reinforced concrete bridge on National Highway 8A was under construction at the time of earthquake and was severely damaged. Significant damage occurred at the east abutment to the support bent and wing walls. This could be attributed to liquefaction resulting in lateral spreading near the abutment and causing a rotational failure of the abutment and first pier. The Surajbadi Bridges; a railway bridge and two highway bridges suffered damages due to liquefaction.

Engineering Analysis:

The potential liquefaction hazards can be evaluated by addressing the following questions:

1. Is the soil susceptible to liquefaction?
2. If the soil is susceptible, will be liquefaction be triggered?
3. If liquefaction is triggered, will damage occur?

If the answer to the first question is no, the liquefaction hazard evaluation can be terminated with the conclusion that liquefaction hazards do not exist. If the answer is yes, the next question must be addressed. If the answers to all three are yes, a problem exists; if the anticipated level of damage is unacceptable, the site must be abandoned and improved or on-site structures strengthened.

4.2.3 Landslides

Strong earthquakes may cause landslides. In majority of the cases landslides are small but earthquakes have also caused very large slides. In a number of cases, earthquake-induced landslides have buried entire towns and villages (Fig. 4.23). Earthquake induced landslides cause damage by destroying buildings or disrupting bridges and other facilities. Many earthquake landslides result from liquefaction phenomena, but many other simply represent the failures of slopes that were marginally stable under static conditions (Figs. 4.24 to 4.26).

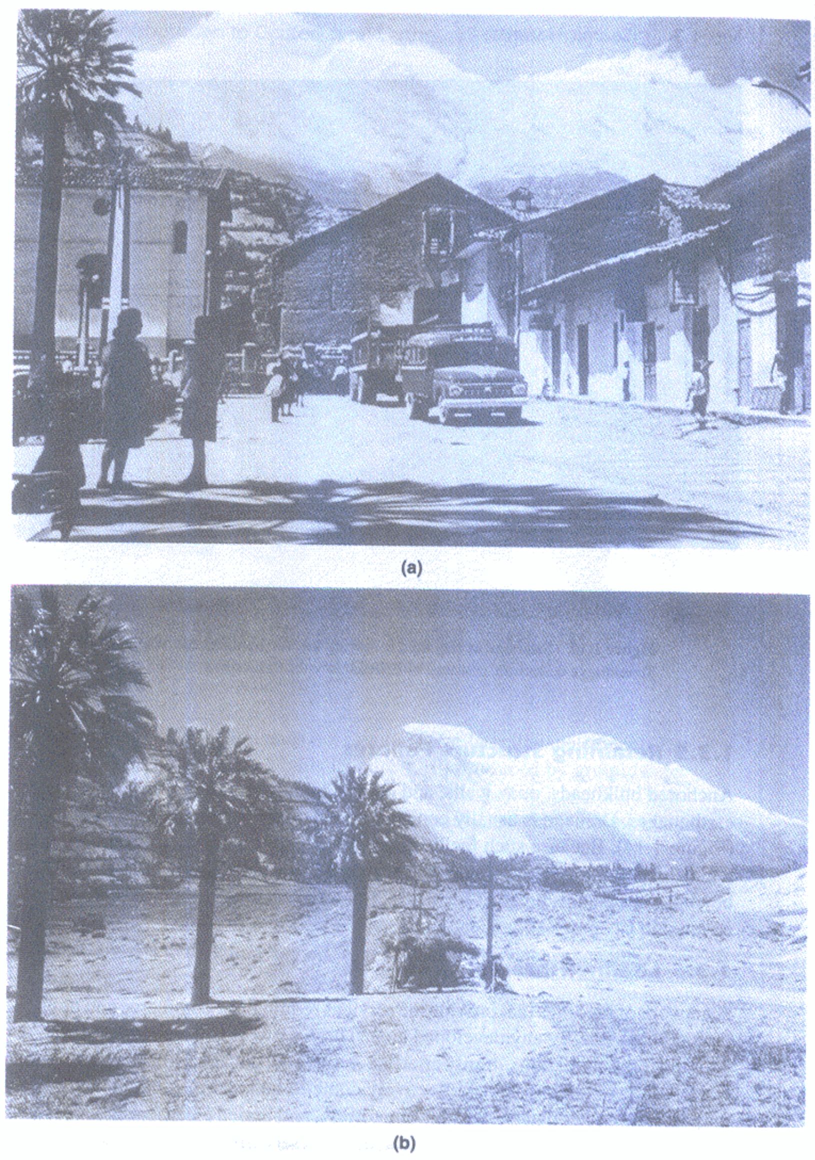


Figure 4.23: Village of Yungay, Peru, (a) before and (b) after being buried by a giant landslide in the 1970 Peruvian earthquake. The same palm trees are visible at the left side of both photographs. The landslide involved 50 million cubic meters of material that eventually covered an area of some 8000 square kilometers. About 25,000 people were killed by this landslide, over 18,000 in the villages of Yungay and Ranrahirca.

Figure 4.24: Damage caused by movement of the Turnagain Heights landslide during the Alaskan earthquake on March 27, 1964.

Figure 4.25: Damage caused by movement of the Turnagain Heights landslide during the Alaskan EQ.

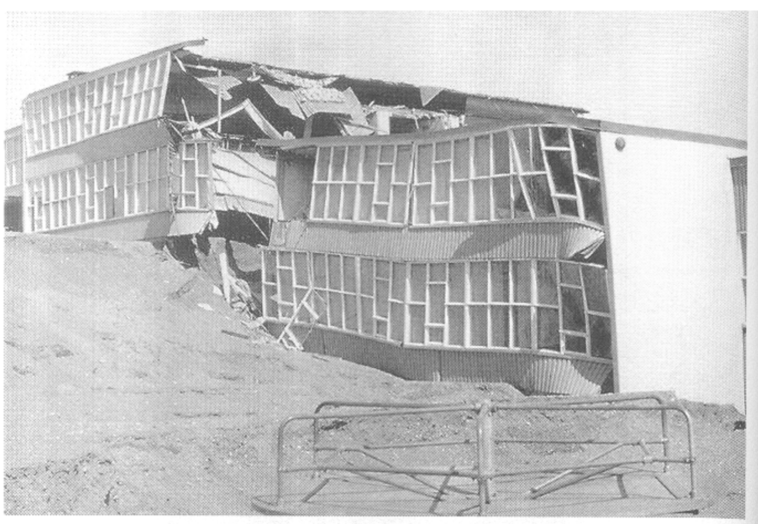


Figure 4.26: Close-up view of damage to the Government Hill School located at the head of a landslide caused by the Alaskan EQ.

The Bhuj earthquake also produced numerous rockfalls from steep slopes and road-cuts. Rockfalls included topple failures and surfacial raveling. Blocks up to 2 m across were displaced on the north side of the Island Belt near Khadir Island. Failures of embankments and cut-slopes were also widespread. Slope failures were most highly concentrated in the area near Bhuj and Bhachau. No large-scale rotational failures were observed on native slopes.
In Chamoli (Himalaya, India) earthquake on March 29, 1999, ground cracks at several places developed as part of slope failure and these pose threat to the down-slope settlements. Cracks were seen in asphalt roads at some locations, indicating the possibility of failure due to ground slippage. At several sites, large-scale earthquake-induced landslide/rock falls were observed as shown in Fig. 4.27.

Figure 4.27: Chamoli (India), earthquake: A major landslide about 1 km north of Gopeshwar. It blocked the road traffic to Okimath for a considerable period. (Source: NICEE, IITK, India, website)

4.2.4 Tsunamis

Rapid vertical seafloor movements caused by fault rupture during earthquakes can produce long-period sea waves called tsunamis. We have already learnt about tsunamis in section 1.3.4. The Great Hoei-Tokaido-Nonhaido tsunami killed 30,000 people in Japan in 1707. The 1960 Chilean earthquake produced a tsunami that not killed 300 people in Chile but also killed 61 people in Hawaii and 22 hours later 199 people in distant Japan. Fig 4.28 and 4.29 shows tsunami damage caused by the 1964 Niigata (Japan) earthquake and 1960 Chile earthquake, respectively. Recent tsunamis caused by 9.0 magnitude Sumatra earthquake of December 26, 2004 have killed more than 220,000 people in Indonesia, Sri Lanaka, Thailand, India and as far as Somalia in Africa. Fig 4.30 to 4.37 shows the tremendous damage caused by these tsunamis.

Figure 4.28: Tsunami damage caused by the 1964 Niigata earthquake in Japan.

Figure 4.29: Tsunami damage caused by the 1970 Chile earthquake.

Figure 4.30: Collapse of the adobe houses in Pondicherry (India) due to tsunami caused by December 26, 2004 Sumatra earthquake.

Figure 4.31: Erosion of a pakka road in Pondicherry (India) due to tsunami caused by Sumatra EQ.

Figure 4.32: Damage to brick masonry construction in Cuddalore (India) due to tsunami caused by Sumatra EQ.

Figure 4.33: Damage to a structure used for drinking water near Cuddalore (India) due to settlement of soil caused by tsunami generated by Sumatra EQ.

Figure 4.34: Collapse of Adobe houses in a village south of Cuddalore (India) due to tsunami caused by Sumatra earthquake.

Figure 4.35: Damage to a police check post due to settlement of soil below at Nagapattinam beach (India) due to tsunami caused by Sumatra earthquake.

Figure 4.36: Damage and mess-up of boats and steamers in Nagapattinam (India) due to tsunami caused by Sumatra earthquake.

Figure 4.37: Due to tsunami caused by Sumatra earthquake, all the four spans of the deck of this bridge was washed away. One span was moved more than 20 m. Bridge is over the sea and connects Keelamanakudai and Melamanakudai villages in Kanyakumari district in South India.

4.3 BEHAVIOR OF STRUCTURES DURING EARTHQUAKES

Earthquakes throughout the world cause a considerable amount of death and destruction. Earthquake damage can be classified as being either structural or non-structural. According to FEMA, USA, Structural damage means a situation where the building's structural support has been impaired. Structural support includes any vertical and lateral force resisting systems, such as building frames, walls and columns. Non-structural damage does not affect the integrity of structural system. Examples of non-structural damage include broken windows, collapsed or rotated chimneys, and fallen ceilings.

During an earthquake, buildings get thrown from side to side, and up and down. Heavier buildings are subjected to higher forces than lightweight buildings, given the same acceleration. Damage occurs when structural members are overloaded, or differential movements between different parts of the structure strain the structural components. Larger earthquakes and longer shaking duration tend to damage structures more. The level of damage resulting from a major earthquake can be predicted only in general terms, since two buildings undergo the exact same motions during a seismic event. However, past earthquakes have shown that some buildings are likely to perform more poorly than others.

Structural damage is the leading cause of death and economic loss in many earthquakes. This damage is not only to unreinforced masonry and adobe structures but also to more modern constructions. Structures need not collapse to cause death and damage during earthquakes. In many earthquakes, falling objects such as brick facings and parapets on the outside of a structure or heavy pictures and shelves within a structure have caused casualties. Interior facilities such as piping, lighting and storage systems can also be damaged during earthquakes.

There are four main factors that cause structural damage during an earthquake:

1. Strength of shaking: For small earthquakes (magnitude less than 6), the strength of shaking decreases rapidly with distance from the epicenter of the earthquake. In the case of a small earthquake, the center of energy release and the point where slip begins are not far apart. But in the case of large earthquakes, which have a significant length of fault rupture, these two points may be hundreds of miles apart. Thus for large earthquakes, the strength of shaking decreases in a direction away from the fault rupture.
2. Duration of shaking: The duration of shaking depends on how the fault breaks during the earthquake. It usually varies from 10 sec to 1 minute. The longer the ground shakes, the greater the potential for structural damage. In general, the higher the magnitude of an earthquake, the longer the duration of the shaking ground.
3. Type of subsurface conditions: Ground shaking can be increased if a site has a thick deposit of soil that is soft and submerged. Many other subsurface conditions can cause or contribute to structural damage. For example, as discussed in section 4.2.2 there could be structural damage due to liquefaction of loose submerged sands.
4. Type of building: Certain types of buildings and other structures are especially susceptible to the side-to-side shaking common during earthquakes. For example, sites located near (about 16 km) the epicenter or location of fault rupture are

generally subjected to rough, jerky, high-frequency seismic waves that are often more capable of causing short buildings to vibrate vigorously. For sites located at greater distance, the seismic waves often develop into longer-period waves that are more capable of causing high-rise buildings and buildings with large floor areas to vibrate vigorously.

As many diseases will attack the weak and infirm, earthquakes damage those structures that have inherent weakness or age-related deterioration. Those buildings that are not reinforced, poorly constructed, weakened from age or underlain by soft or unstable soil are most susceptible to damage. Over the years, considerable advances have been made in earthquake-resistant design of structures and seismic design requirements in building codes have steadily improved. Earthquake-resistant design concept has moved from an emphasis on structural strength to emphasis on both strength and ductility.

According to IS 1893 (Part 1): 2002 – To perform well in an earthquake, a building should possess four main attributes, namely simple and regular configuration, adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation, suffer much less damage than buildings with irregular configurations.

In the following sections, behavior of various building structures during earthquakes has been examined. Major causes of damage to buildings during earthquakes has been discussed which include plan asymmetry, soft story, settlement, torsion, pounding damage and resonance.

4.3.1. Plan Asymmetry

The buildings that are asymmetric, such as T or L shaped buildings, can experience more damage as different parts of the building vibrate at different frequencies and amplitude. This difference in movement of different parts of the building is due to the relative stiffness of each portion of the building. For example, for the T shaped building the two segments that make up the buildings are usually much stiffer in their long directions then across the segments. Thus damage tends to occur where the two segments of the T join together.

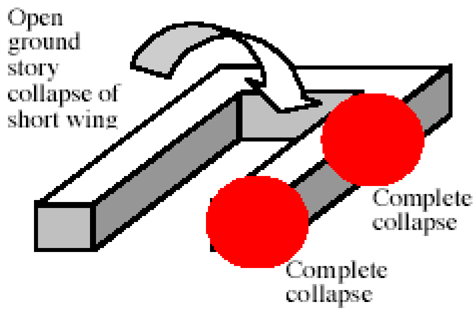


Figure 4.38: U-shaped plan of commercial- residential building in Bhuj

Figure 4.39: Open ground story collapse of short wing of same building

During Bhuj earthquake (Jan. 2001) many buildings were damaged due to plan asymmetry. In Bhuj, a five-story U shaped plan apartment-commercial building had a tall ground story to accommodate shops. Infills were made of sandstone blocks of random sizes and mud mortar. Use of non-ductile columns, coupled with a small number of infill walls in mud mortar in the already flexible ground story, caused complete collapse in some portions of one of the long blocks. The shorter and stiffer wing of the U-shaped building was separated from the rest of building, and underwent ground story collapse. Plan asymmetry, flexible ground story, and weak infills seem responsible for this collapse (Fig 4.38 to 4.40).

Buildings with L-shaped plans in Bhuj also sustained severe damage and collapsed. A 5-story residential building in Bhuj with partially open ground story also collapsed (Fig. 4.41).

Figure 4.40: Shear cracking in infill masonry in weak cement mortar and sandstone blocks

Figure 4.41: Collapse of a partially open ground story building in Bhuj, coupled with vertical split of the building at midline

During Bhuj earthquake, a 4-story RC frame building with brick infills under construction in Gandhidham sustained partial collapse of the upper two stories of the left block (Fig. 4.42). By the time of the earthquake, infills were provided everywhere except in upper two stories of the left block. The absence of infill walls in the left block created plan asymmetry. Consequently, the flexible upper stories of the left block experienced larger deformations during the earthquake. This, coupled with the existence of nonductile columns, led to the collapse of the upper two stories. A 3-story commercial building on a corner plot in Gandhidham sustained partial collapse (Fig. 4.43). The ground story of one of its wings collapsed and pulled apart from the other.

Figure 4.42: Collapse of the upper stories of this Gandhidham apartment building was due to plan irregularity. The upper stories of the left block experienced larger deformations due to the absence of infill walls. The building was under construction at the time of the earthquake

Figure 4.43: Collapse of corner owing to failure of diaphragm at the re-entrant corner of an L-shaped building in Gandhidham

<u>Short Column Effect (Bhuj):</u> Numerous examples of the short-column effect were noted, mostly owing to the unintended use of partial infills. In Bhuj and other towns, open spaces are not always left between the plot boundary and the ground story columns of the multistory RC frame buildings. Unreinforced brick masonry walls were raised between the perimeter frame columns to form the boundary wall. Again, for ventilation purpose these infills were stopped at 1.5 m from the floor level. This created a short column effect on the adjoining RC columns, which suffered significant diagonal shear cracking (Fig. 4.44).

<u>Water Tanks Atop Buildings (Bhuj):</u> A variety of damages were associated with water tanks placed on top of buildings. Some tanks sustained minor cracks in their walls and their connections with the building frame. Some tanks simply toppled from the elevated pedestals onto roof slabs, owing to lack of adequate connection between tank and building. Many tanks dislodged from the RC columns to which they were weakly connected (Fig. 4.45).

Figure 4.44: damage to perimeter columns of building with varying lateral supports from masonry infills: shear failure in short columns and flexure/shear failure in others.

Figure 4.45: Damage to small service water tank on top of RC frame building in Bhuj. Tanks are either nominally or not connected to building frame.

4.3.2 Soft Story

A soft story, also known as a weak story, is defined as a story in a building that has substantially less resistance, or stiffness than the stories above or below it. In essence, a soft story has inadequate shear resistance or inadequate ductility (energy absorption capacity) to resist the earthquake-induced building stresses. Although not always the case, the usual location of the soft story is at the ground floor of the building, this is because many buildings are designed to have an open first-floor area that is easily accessible to public. Thus the first floor may contain large open areas between columns, without adequate shear resistance. The earthquake-induced building movement also causes the first floor to be subjected to the greatest stresses, which compounds the problem of a soft story on the ground floor.

Concerning the soft stories, the National Information Service for Earthquake Engineering, USA (2000) states: ‘In shaking a building, an earthquake ground motion will search for every structural weakness’. These weaknesses are usually created by sharp changes in stiffness, strength and/or ductility and the effects of these weaknesses are accentuated by poor distribution of reactive masses. Severe structural damage suffered by several modern buildings during recent earthquakes illustrates the importance of avoiding sudden changes in lateral stiffness and strength. Inspection of earthquake damage as well as the results of analytical studies have shown that structural systems with a soft story can lead to serious problems during severe earthquake ground shaking. Many examples illustrate such damage and therefore emphasize the need for avoiding the soft story by using an even distribution of flexibility, strength and mass.

Effect of soft story on the ground floor during Bhuj earthquake:

A large number of open ground story in Ahmedabad, Bhuj, Gandhidham, and other towns suffered severe damage or dramatic collapse. For example, of the 130 buildings that collapsed in Ahmedabad, most were of open ground story configuration. Among those that did not collapse, the damage was confined mostly to the open ground story columns with nominal frame infill separation in the upper stories. When elevator core walls were of RC (Reinforced Concrete) and well connected with the floor slab, they added some lateral strength and stiffness to the ground story.

The RC frames with masonry infills formed a relatively stiff and strong lateral load resisting system in the upper stories, in contrast to the columns with few or no infill walls in the ground story. As a result, almost the entire lateral deformation is concentrated in the ground story columns, and the upper story moved laterally as a rigid block (Fig. 4.46).

Moreover, unlike the upper story columns, the ground story columns in such buildings could not share the lateral share with the infill walls. Since these columns were neither designed for lateral forces nor detailed for ductile behavior, many of them sustained brittle shear failure or flexural failure resulting from large moment and axial load (Fig. 4.47). Once the ground floor columns failed, the gravity load-carrying capacity of the building was partially/completely lost resulting in partial/complete collapse of many buildings.

Figure 4.46: Open ground story buildings with masonry infills only in the upper stories are subject to severe deformation demands on ground story columns.

Figure 4.47 Shear cracking of non-ductile columns of an open ground story building in Bhuj

Figure 4.48: RC elevator cores saved many buildings with open ground stories from collapsing. These stiff elements drew large seismic shears and sustained severe cracking

Similarly, the RC walls of elevator cores, where provided, also experienced severe diagonal cracking at the ground floor (Fig. 4.48). In commercial buildings with shops in the ground story, the damage was of varying levels. Shops were closed when the earthquake struck, and the steel rolling shutters were down. The RC columns adjoining these rolling shutters in the ground story sustained severe damage (Figs. 4.49 and 4.50).

Figure 4.49: RC columns sustained severe damage. Shutters were down at the time of the earthquake

Figure 4.50: Many open ground story buildings did not collapse, but sustained severe damage to the columns in the ground story

Shear Failure

During Bhuj earthquake, few buildings failed due to shear failure in columns. Detailing practice for transverse ties in columns in the affected area offers very light confinement to the core concrete against the large compressive stress generated by the extreme lateral deformation demands during strong seismic shaking. Ground floor with such reinforcement failed in brittle shear mode leading to catastrophe failure of many open ground story buildings; in some buildings, only a few ground story columns sustained significant shear and flexural cracking (Fig. 4.51). The 90° hooks of transverse reinforcement (at 200 mm centers) opened-up and resulted in buckling of longitudinal bars and consequent dilatation of core concrete (Fig. 4.47).

Figure 4.51: Shear failures in columns due to inadequate confinement reinforcement

Figure 4.52: Intermediate story collapse in the 6 story building in Bhuj

Pancaking

Pancaking occurs when the earthquake shaking causes a soft story to collapse, leading to total failure of the overlying floors. These floors crush and compress together such that the final collapsed condition of the building consist of one floor stacked on top of another, much like a stack of pancakes. During Bhuj earthquake, many full pancake collapse occurred owing to weak-column strong-beam design, nonductile detailing and plan asymmetry. In multistory building design, reduction of column sizes and curtailment of reinforced at an intermediate story is often employed. At least two instances of failure in the upper stories and the consequent partial pancake type collapse of the buildings are recorded (Fig. 4.52). Pancaking of reinforced concrete multistory buildings was common throughout the earthquake-stricken region of Turkey due to the 1999 Izmit earthquake (Fig. 4.53).

Figure 4.53: Pancaking of a building during the 1999 Izmit earthquake in Turkey

Figure 4.54 Damage to a shear wall at the west Anchorage High School caused by the 1964 Alaskan EQ

Shear Walls

Many different types of structural systems can be used to resist the inertia forces in a building that are induced by the earthquake ground motion. For example, the structural engineer could use braced frames, moment-resisting frames, and shear walls to resist the lateral earthquake-induced forces. Shear walls are designed to hold adjacent columns or vertical support members in place and then transfer the lateral forces to the foundation. The forces resisted by shear walls are predominately shear forces, although a slender wall could also be subjected to significant bending. Fig. 4.54 shows the failure of a shear wall at the West Anchorage High School caused by the Price William Sound earthquake in Alaska on March 27, 1964. Common problems with shear walls are that they have inadequate strength to resist the lateral forces and that they are inadequately attached to the foundation. For example, having inadequate shear walls on a particular building level can create a soft story. A soft story can also be created if there is a discontinuity in the shear walls from one floor to the other.

Lessons Learned from the Damages to RC Structures during Bhuj Earthquake

The types of failures of reinforced concrete frame buildings sustained during the 2001 Bhuj earthquake were also observed in many past earthquakes around the world. Vulnerability of open ground story buildings was also seen in the 1997 Jabalpur, India earthquake and the 1999 Izmit earthquake of Turkey. This is, however, the first time that damage of such a large-scale has been observed in India. The failure of many buildings with open ground stories appears to be due to the lack of design and detailing for seismic forces, and poor construction practices that have led to poor lateral strength and ductility capacities in the lateral load resisting elements at the open story. In contrast, large demands are placed on these elements during strong ground motion.

Important lessons from this earthquake include:

1. Buildings need to be designed for earthquake forces. Seismic strength and deformability of current construction is extremely inadequate.
2. Ductile detailing is essential to be able to resist strong seismic effects.
3. The popular open ground story building system is vulnerable under strong seismic shaking.
4. The beneficial effects of stiffness and strength of brick masonry infills should be exploited.
5. Irregular structural configurations (e.g. plan asymmetry, vertical offsets, short column effects) result in poor seismic performance.

4.3.3 Earthquake-Induced Settlement (Foundation Failure)

Those buildings founded on solid rock are least likely to experience earthquake-induced differential settlement. However, buildings on soil could be subjected to differential settlement causing foundation to fail. As discussed in section 4.2, a structure could settle or be subjected to differential movement from surface rupture, liquefaction, slope movement or tsunami generated due to an earthquake. Two additional conditions can cause settlement of a structure:

(i) Volumetric compression, also known as cyclic soil densification.
(ii) Settlement due to dynamic loads caused by rocking.

4.3.4 Torsion (Asymmetry structures)

Torsional problems develop when the center of mass of the structure is not located at the center of its lateral resistance, which is also known as the center of rigidity (see section 6.3.6 also). A common example is a tall building that has a first floor area consisting of a space that is open and supports the upper floors by the use of isolated columns, while the remainder of the first-floor area contains solid load-bearing walls that are interconnected. The open area having isolated columns will typically have much less lateral resistance than the part of the floor containing the interconnected load bearing walls. Thus center of rigidity is offset from the mid-point of the first floor (location of center of mass of building) and during the earthquake, the center of mass will twist about the center of rigidity, causing torsional forces to be induced into the building frame. An example of damage due to torsion is shown in Fig. 4.55.

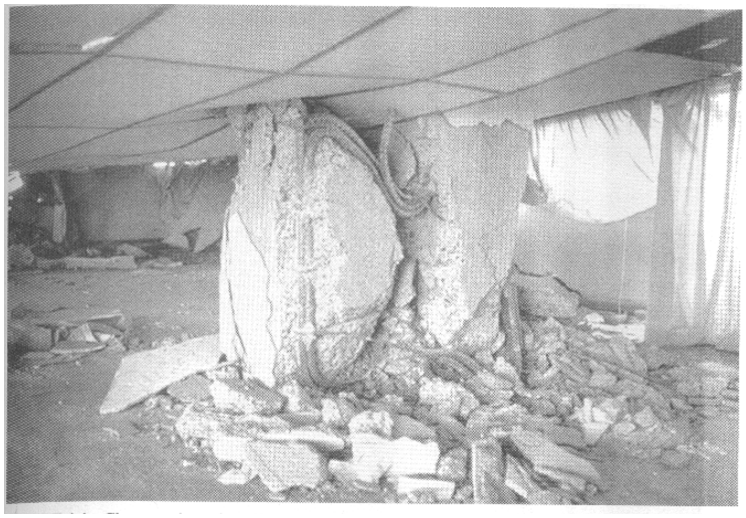


Figure 4.55: Close-up view of a collapsed second-story column at the Hotel Terminal. Note that the upper floor has displaced to the right and dropped, and the top and bottom sections of the column are now side by side. The torsional failure occurred during the 1976 Guatemala earthquake.

4.3.5 Pounding Damage

Pounding damage can occur when two buildings are constructed close to each other and, as they rock back-and-forth during the earthquake, they collide into each other. In the common situation for pounding damage, a much taller building, which has a higher period and larger amplitude of vibration, is constructed against a squat and short building that has a lower period and smaller amplitude of vibration. Thus during the earthquake, the buildings will vibrate at different frequencies and amplitude, and they can collide with each other. The effects of pounding can be especially severe if the floors of one building impact the other building at different elevations, for example, the floor of one building hits a supporting column of an adjacent building.

Figure 4.56: An example of pounding damage and eventual collapse caused by the Izmit earthquake.

An example of pounding damage is shown in Fig. 4.56. The buildings were damaged during the Izmit earthquake in Turkey. As shown in figure, the pounding damage was accompanied by the collapse of the buildings into each other. It is very difficult to model the pounding effects of two structures and hence design structures to resist such damage. As a practical matter, the best design approach to prevent pounding damage is to provide sufficient space between the structures to avoid the problem. If two buildings must be constructed adjacent to each other, then one design feature should be to have the floors of the buildings at same elevations, so that the floor of one building does not hit a supporting column of an adjacent building. Similar to pounding damage, the collapse of a building can affect adjacent structures.

4.3.6 Resonance of the Structure

Resonance is defined as a condition in which the period of vibration of the earthquake-induced ground shaking is equal to the natural period of the building (see section 3.5 and 3.6).

Soft Ground Effects: If the site is underlain by soft ground, such as a soft and saturated clay deposit, then there could be an increased peak ground acceleration (PGA), and a longer period of vibration of the ground. In Michoacan Earthquake in Mexico on Sept. 19, 1985, there was extensive damage to Mexico City occurred to those buildings underlain by 39 to 50 m of soft clays, which are within the part of the city known as the Lake zone. Because the epicenter was so far from Mexico City, the PGA recorded in foothills of Mexico City (rock site) was about 0.04g. However, at the Lake Zone, the PGA were up to 5 times greater than at the rock site. In addition, the characteristic site periods were estimated to be 1.9 to 2.8 s. This longer period of vibration of the ground tended to coincide with the natural period of vibration of the taller buildings in the 5 to 20 story range. The increased PGA and the effect of resonance caused either collapse or severe damage of these taller buildings such as shown in Fig 4.57.

Figure 4.57: Building collapse in Mexico City caused by the 1985 Michoacan earthquake.

4.3.7 Damage to Masonry Structures during Bhuj Earthquake

The most elementary of masonry construction is random rubble stone (granite) masonry in mud mortar. The wood used in the roofing is not formally cut and shaped. Earthquake-resistant features are not built into this system. There are no connections between the walls, and the roof. Housing of this type, found primarily in economically weaker sections of the society, performed extremely poor during the 2001 Bhuj earthquake. While these low-strength masonry units were high on fulfilling functional needs, they were structurally unsuitable to resist lateral seismic loads because of the building materials used. In this type of construction, little attention is paid to the details that make the roof and walls act together as a single entity. The unusually large size (up to 600 mm) pink sandstone masonry units and mud mortar (up to 75 mm thickness) used in making two-story residential buildings resulted in brittle performance (Fig. 4.58). Pink sandstone is lighter than granite, readily available, and hence very popular in the Kachchh region. Owing to the coarse shapes of the stones, the thickness of the mud mortar required for leveling is sometimes as large as 8 cm (Fig. 4.59). Such large masonry blocks with unusually large mortar thickness of a basically weak mortar material (mud) resulted in very poor performance of a large number of such structures in Bhuj. Heavy purlins carrying the weight of the roof cause stress concentration on the walls at the support points. The stone-mud walls sustained severe cracking at these locations (Fig. 4.60). Traces of traditional wisdom were seen in some structures that survived the shaking with little damage, where lintel and post system provided lateral resistance (Fig. 4.61). This practice may have come from the construction of monumental/heritage construction in the area that used wood frame in a significant way to counter seismic forces.

Figure 4.58: Pink sandstone up to 600 mm in size was used for 2-story houses

Figure 4.59: Mud mortar is sometimes as thick as 8 cm

Figure 4.60: Walls built with large stones and no through-stones separated impairing the vertical load carrying capacity.

Figure 4.61: Older construction used a large amount of wood in the post and lintel system, thereby providing some lateral resistance to inherently weak stone masonry mud mortar.

4.4 PERFORMANCE OF LIFELINES DURING EARTHQUAKES

Lifelines systems are distributed systems that provide service to maintain societal functions. Lifelines are critical in providing emergency response recovery, and in restoring society to normal. Lifeline systems include transportation systems (highways, railroads, marine ports and harbors, and airports), electric power, telecommunications, water, sewer, gas and liquid fuels – natural and liquid petroleum gas (LPG). Lifelines are sometimes defined to include health care systems e.g. hospitals. Performance of lifelines during Bhuj earthquake of 2001 is examined in following sections:

4.4.1 Electrical Power

The Gujarat Electric Board (GEB) provides electric power service to the region, with the exception of the city of Ahmedabad. The electrical power system serving the region was heavily damaged, with damage estimated to be $65 million. Nineteen GEB employees were killed in the earthquake, and forty lost family members, which adversely affected response time. The residential quarters at the Anjar substation collapsed, causing several casualties (Fig. 4.62). Power was completely out in the Kutchh district for two days. The GEB assigned 220 engineers and skilled staff to the region for assistance. By February 5, ten days after the earthquake, service was 80% restored to the 255 feeders, and fully restored by February 19.

Figure 4.62: Collapsed residential quarters at Anjar substation

Figure 4.63: Damaged wall inside Anjar substation control building with temporary support column

Primary power supply in the region is 3-70 MW lignite burning plants located in Panandhro, about 180 km northwest of the epicenter. The three plants experienced only minor cracking. Supply is supplemented by a coal-burning plant in Ahmedabad (270 km east of the epicenter) which was undamaged. The power supply is also supplemented by the Kakrapar nuclear plants Nos. 1 and 2, which are owned by the Nuclear Power Corporation of India, and located in Surat about 400 km from the epicenter. These 1993 PHWR plants were undamaged.

Transmission System

Power is transmitted from Panandhro, through 220 kV and 132 kV systems, southeast through the region. Voltage is dropped to 66 kV and ultimately 11 kV for local distribution. There was minimal damage to transmission towers. However, there were a few towers affected by liquefaction on small landslide. The transmission lines crossing the little Rann of Kachchh were undamaged, though the soils did liquefy in some areas.

Electrical Substations

Over a dozen substation control buildings of unreinforced stone masonry collapsed, and a total of 45 were damaged. Control building damage had the greatest impact on the overall system failure. Fig. 4.63 shows Anjar substation control building damage. Damage to the stone masonry control buildings was a problem throughout the system. Most of the control buildings were constructed of unreinforced stone-mortared walls, with reinforced concrete beams forming window lintels and roof supports. At the 132 kV Bhuj substation, the control room had major cracks, but the building remained standing.

Distribution Feeder Equipment

The low-voltage distribution system equipment in self-standing electrical panels located in substation control buildings was generally not damaged, except when the building was damaged. It was confirmed that some panels were anchored to their foundations, although other reconnaissance teams have reported seeing nonanchored equipment. It was observed and confirmed by discussion that no individual electrical panels toppled in the earthquake.

4.4.2 Telecommunications

Telephone service was disrupted in the region due to: collapse and/or severe damage to many telecom buildings, damage to fiber-optic lines, and system overload. Telephone exchanges were overloaded due to heavy traffic at a rate of 1,200,000 calls per hour. The Army Corp of Signals, using VSAT (very small aperture terminals), established communications outside the region, within minutes after the earthquake. They also worked to supplement emergency communications by airlifting mobile satellite communication terminals into the region, 13 by January 28, and a total of 30 by January 30. All affected exchanges were restored by February 11.

Telecom Building Damage

Extensive Telecom building damage impacted system operation and restoration. Initially, the exchanges at Bhachau, Khavda, Nakhtrana, and Raphar were set up in tents. Nakhtrana and Raphar were moved back into their original buildings after the main exchange was restored. Two exchanges were set up in tents, 10 exchanges set up in other locations, and eight exchanges set up in nearby buildings because of severe damage to their original buildings. The main Telecom Building in Bhuj was heavily damaged. It was constructed out of reinforced concrete frames with stone infill walls. Unreinforced infill walls collapsed, causing damage to interior panels and exterior facilities. The reinforced concrete frames appeared to be undamaged. Emergency power provided adequate power to the exchanges until grid power was restored. The Telephone Exchange building in Bhuj was heavily damaged. Some of the exterior masonry collapsed during the earthquake, causing a life safety hazard to nearby pedestrians and cars. The structure itself remained standing. It was reported that seven people died at this facility during the earthquake.

4.4.3 Water Supply

There are over 800 villages and cities in the Kachchh region. Over 90 percent are supplied by one of approximately 140 Regional Water Supply Systems constructed and operated by the Gujarat Water Supply and Sewerage Board (GWSSB). Most of the regional systems are supplied by groundwater. Two of the regional systems, one being Gandhidham, are partially served by surface water. The Kachchh region has been undergoing a drought for several years. Prior to the earthquake, 150 of the villages were being at least partially supplied with water transported by tank trucks; the number was expected to increase to 200–250 during the summer months. Normal precipitation is about 10–15 inches per year. Water demand is about 70 liters per person per day. The water supply systems in the region were heavily damaged as described below. As of February 19, many of the areas still depended on water delivery by tankers. In the first five days following the earthquake, the GWSSB delivered 8–10 liters per day per person (Fig. 4.64). As of February 8, that had increased to 50–100 lpd.

Figure 4.64: Women approaching water distribution location at Rambaug

4.4.4 Sewage

Five cities in the Kachchh district have complete or partial sewage collection and treatment systems, including Ganhidham, Adipur, Bhuj, and others. Bhuj has an oxidation pond for sewage treatment. The Gujarat Water Supply and Sewerage Board (GWSSB) headquarters in Bhuj collapsed. In Bhuj, there were no pipe collapses identified as of February 7 on main lines, but the system had not been fully tested. A representative of GWSSB in Bhuj expressed concern about long-term weakening of the pipe due to hydrogen sulfide corrosion. The Bhuj system's sewer pipe is 90 percent reinforced concrete and 10 percent stoneware (used for connection to houses). They were manually chlorinating their effluent using sodium hypochlorite. Prior to the earthquake, the Central Rural Sanitation Program was established to promote the use of sanitation in rural villages. The CRSP provided subsidies up to 70-80%, paid by the central Indian government, to encourage use of sanitation facilities in rural villages. A second program was in place prior to the earthquake that offered 50-75 percent subsidies (income based) to get latrines in every household. These programs were not entirely effective before the earthquake. As of late February 2001, emergency relief workers observed that these sanitation programs could not be effectively implemented in the post earthquake restoration process, at least at that time.

4.4.5 Natural Gas and Liquid Fuel

There is no natural gas system in Gujarat, India. There are 16 LPG distributors in the Kachchh district that were only briefly interrupted. Supply of liquid fuel was initiated after the earthquake on the 26th through the 31st of January by truck, rail tank car, and ultimately pipeline. Liquid fuels are produced at two refineries. The Indian Oil Company's (IOC) Koyali refinery was not affected by the earthquake, and maintained operation. The Reliance refinery in Jamnagar, south of the Gulf of Kachchh, shut down on the January 26. No damage was reported. It had not restored operation as of February 7. The crude oil pipeline from Salaya, feeding Koyali/Mathura/Panipat refineries, resumed operations on January 27th after it was checked for damage. The IOC reported that the Koyali-Ahmedabad pipeline was operational. The 800 mm product pipeline from Vadinar to Kandla, and from Kandla to Bhatinda was initially shutdown but resumed operation after it was determined that it was not damaged (Fig. 4.65).

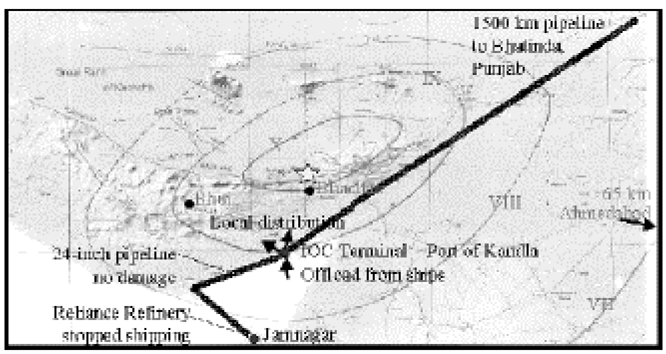


Figure 4.65: 800 mm pipeline running from the Reliance Refinery at Jamnagar to Kandla, and northeast to Bhatinda. The 24-inch pipeline was not damaged. The IOC facility at the Port of Kandla receives liquid fuels via pipeline and ship, and provides local distribution.

4.4.6 Bridges, Rail Roads and Airports

Immediately following the earthquake, there was no communication or access to the Kachchh region from the south. The Surajbadi Bridge crossing the Little Rann of Kachchh was damaged, requiring two days to repair. On reopening, the Surejbari Bridge was only open to traffic in one direction. Otherwise, bridge performance had little impact on the region.

The rail system in India, Gujarat State, and the Kachchh district is well developed. In the Kachchh District, the rail lines are composed of both meter-gauge and broad-gauge tracks. The meter-gauge system represents an older installation, and is still in service today. There was pervasive damage observed to the rail system, although no trains derailed as a result of the earthquake. The engineer on one train in the epicentral area felt the shaking, and brought the train to a safe stop. Railroad bridges were damaged, bridge approaches settled, and lateral spreading caused track misalignment causing the need for new ballast. Bridge girders shifted laterally and longitudinally 10 to 50 mm, some requiring realignment. There were cracks in masonry bridge structures. Parapets fell off many masonry arch bridges.

The earthquake impacted the civilian/Indian Air Force airport in Bhuj. The National Airport in Ahmedabad was minimally disrupted. There is a helicopter facility at the Kandla airport that was engaged in recovery activities. The airport in Bhuj is large enough to handle Airbus 320s and IL 76 military aircraft. The main airport in the Kachchh district is at Bhuj. Damage included collapse of one of the control towers and damage to many structures on the base. A tent served as the temporary control tower as personnel thought is was dangerous to work inside the second, but still standing, control tower. Radar and navigational beacons were completely destroyed, there were no navigational aids until February 5, and satellite based communications were restored on January 27th, the day following the earthquake. Two airport personnel died in the terminal building, and many employees/family members died in their homes.

The main handicap immediately after the earthquake was lack of communication. The Bhuj airport could not communicate with the airports in Bombay (Mumbai) or Ahmedabad, but could handle aircraft landing. The runway was safe for landing, there was radio communication with planes, but no communication outside the immediate landing area. In fact, within an hour of the earthquake, the first helicopter landed. Within a few hours of the earthquake, aircraft could land. On the civilian side, the passenger facility failed. Temporary offsite passenger airline facilities (Jet Air and Indian Airlines) had been put in service by mid-February. On the first day of the earthquake, there were four flights to the airport. The next day, January 27th, there were 18 flights. Large cargo planes began to move equipment and personnel from bases as far away as Chandigarh, and fly critically injured victims to Pune and other centers. Over 230 tons of relief materials including medicines, water, tents, blankets and food were carried in over 170 military sorties. In the first five days following the January 26th, 2001 earthquake, the Bhuj airport handled over 800 landings and departures. Airport charges were waived for relief flights. The main bottleneck was parking space for the aircraft. The airport used one taxiway for parking the airplanes, but this quickly became crowded, and the control tower sometimes had to tell Bombay or Ahmedabad airports to hold further flights because of the parking situation. As a result, some flights may have been delayed by several hours. The National Airport in Ahmedabad suffered essentially no damage from the earthquake. Offsite power was lost to this airport almost immediately, but the onsite emergency generators worked, and so there was no effect on airport operations.

4.5 BEHAVIOR OF NON STRUCTURAL ELEMENTS DURING EARTHQUAKES

Non-structural elements have an important role in the reliability or predictability of seismic response of any given type of construction. In considering the form of a structure, it is important to be aware that some elements, which are normally non-structural become structurally very responsive in earthquakes. This means anything, which will interfere with the free deformations of the structure during an earthquake. In buildings the principal elements concerned are cladding, perimeter infill walls, and internal partitions. Where these elements are made of very flexible materials, they will not affect the structure significantly. However, very often it will be desirable for non-structural reasons to construct them of stiff materials such as precast concrete, or concrete blocks, or bricks. Such elements can have a significant effect on the behavior and safety of the structure. Although these elements may be carrying little vertical load, they can act as shear walls in an earthquake with the following important negative or positive effects.

They may;

(1) Reduce the natural period of vibration of the structure, hence changing the intake of seismic energy and changing the seismic stresses of the 'official' structure.
(2) Redistribute the lateral stiffness of the structure, hence changing the stress distribution, sometimes creating large asymmetries.
(3) Cause premature failure of the structure usually in shear or by pounding.
(4) Suffer excessive damage themselves, due to shear forces or pounding.
(5) Prevent failure of otherwise inadequate moment-resisting frames.

First, let us consider the negative effects of infill construction. The more flexible the basic structure is, the worse the effects can be; and they will be particularly dangerous when the distribution of such 'non-structural' elements is asymmetric or not the same on successive floors.

In attempting to deal with the above problems, either of two opposite approaches may be adopted. The first is knowingly to include those extra shear elements into the official structure as analysed, and to detail accordingly. This method is appropriate if the building is essentially stiff anyway, or if a stiff structure is desirable for low seismic response on the site concerned. It means that the shear elements themselves will probably require aseismic reinforcement. Thus, 'non-structure' is made into real structure. The second approach is to prevent the non-structural elements from contributing their shear stiffness to the structure. This method is appropriate particularly when a flexible structure is required for low seismic response. It can be affected by making a gap against the structure, up the sides and along the top of the element. The non-structural element will need restraint at the top (with dowels, say) against overturning by out-of-plane forces. If the gap has to be filled, a really flexible material must be used.

Unfortunately, neither of the above solutions is very satisfactory, as the fixing of the necessary ties, reinforcement, dowels, or gap treatments is time-consuming, expensive, and hard to supervise properly. Also, flexible gap fillers will not be good for sound insulation. It can be seen from the above discussion that in regions of moderate to high seismic hazard, solid infill walls should not be added or subtracted from existing buildings without checking the earthquake resistance consequences. Finally, the positive side of infill walls should not be neglected. Many buildings would have had their performance improved by infill in past earthquakes. For example, low-rise pre-code reinforced concrete buildings in the intensity MM10 zone of the 1931 Hawke's Bay, New Zealand, earthquake were evidently saved from much more serious damage by brick infill. Many examples of this behavior were also observed in the 2001 Bhuj, India, earthquake, and in New Zealand earthquakes.

While the positive stress redistribution of infill panels is readily explained, all the pros and cons are not easily predicted. However, it appears that simple unreinforced infill, disposed symmetrically or u-shaped in plan, and built full wall height is more likely to be beneficial than not, if it does not fallout prematurely.

Damage to Nonstructural Elements: In general these elements with the three main areas of concern are:

1. The contents of the building: which may include incoming gas lines, chemical laboratories, storage shelves, lockers and vending machines, explosive and radioactive materials, expensive equipment, glass tubes etc.
2. Architectural Items: It may include ceiling and lights, partitions etc.
3. Mechanical and Electrical Building Components: it includes loss of facilities, electrical components, mechanical equipment.

4.6 THE SOCIAL AND ECONOMIC CONSEQUENCES OF EARTHQUAKES

Before discussing economic or financials consequences of earthquakes, their impacts on society and level of acceptability are discussed.

4.6.1 Earthquake Consequences and their acceptability

The primary consequence of concern in earthquakes is human casualties, i.e. deaths and injuries. The number of casualties in any given event varies enormously, depending on the magnitude, location and area of the earthquake. Number of casualty counts is caused by the collapse of buildings made of heavy, weak material such as unreinforced masonry or earth.

The physical consequence of earthquakes for human beings are generally viewed under two headings:

(A) Damage and injury to human beings
(B) Damage to the built and natural environments.

These physical effects in turn are considered as to their social and economic consequences:

1. Number of casualties
2. Trauma and bereavement
3. Loss of employment
4. Loss of employee/skills
5. Loss of heritage
6. Material damage cost
7. Business interruption
8. Consumption of materials and energy (sustaining resources)
9. Macro-economic impacts

The above physical and socio-economic consequences should be taken into account when the acceptable consequences are being decided, i.e. the acceptable earthquake risk.

Both financially and technically, it is possible to reduce these consequences for strong earthquake shaking. The basic planning aims are to minimize the use of land subject to the worst shaking or ground damage effects, such as fault rupture, landslides or liquefaction. The basic design aims are therefore confined (a) to the reduction of loss of life in any earthquake either through collapse or through secondary damage such as falling debris or earthquake-induced fire, and (b) to the reduction of damage and loss of use of the built environment.

Naturally some facilities demand greater earthquake resistance than others, because of their greater social and/or financial significance. It is important to determine in the design brief not only the more obvious intrinsic value of the structure, its contents, and function or any special parts thereof, but also the survival value placed upon it by the owner. Some of the most vital facilities to remain functional after destructive earthquakes are hospitals, fire and police stations, government offices, dams, bridges, radio and telephone services, schools, energy sources or in short, anything vitally concerned with preventing major loss of life in the first instance and with the operation of emergency services afterwards. The consequences of damage to structures housing intrinsically dangerous goods or processes is another category of consideration, and concerns the potential hazards of fire, explosion, toxicity, or pollution represented by installations such as liquid petroleum gas storage facilities or nuclear power or nuclear weapon plants. These types of consequences often become difficult to consider logically, as strong emotions are provoked by the thought of them.

Since the 1970s, with the growing awareness of the high seismic risk associated with certain classes of older buildings, programmes for strengthening or replacement of such property have been introduced in various parts of the world, notably for pre-earthquake code buildings of lightly reinforced or unreinforced masonry construction. While the substantial economic consequences of the loss of many such buildings in earthquakes are, of course, apparent, the main motivating force behind these risk-reduction programmes has been social, i.e. the general attempt to reduce loss of life and injuries to people, plus the desire to save buildings or monuments of historical and cultural importance.

Not only individual owners are concerned with the consequences of damage to their property but also the overall effects of a given earthquake are also receiving attention. Government departments, emergency services and insurance firms all have critical interests in the physical and financial overall effects of large earthquakes on specific areas. For example, disruption of lifelines, such as transport, water and power system obviously greatly hampers rescue and rehabilitation programmes.

4.6.2 Economic Consequences of Earthquakes

Figure 4.66 plots the costs of earthquake material damage worldwide per decade in the 20th century. The first half of century is incomplete; only the material damage costs for the 1906 San Francisco and the 1923 Kanto earthquakes being readily found. As with the 20th century death sequence the costs sequence is seen to be random. However, there is no correlation between the deaths and costs sequences. If the costs were normalized to a constant population and if the 1995 Kobe earthquake was not included, there would be no trend to increase with time. However, the global seriousness of earthquake damage losses is undisputed. The economic consequences of earthquakes occur both before and after the event. Those arising before the event include protection provisions such as earthquake resistance of new and existing facilities, insurance premiums and provision of earthquake emergency services.

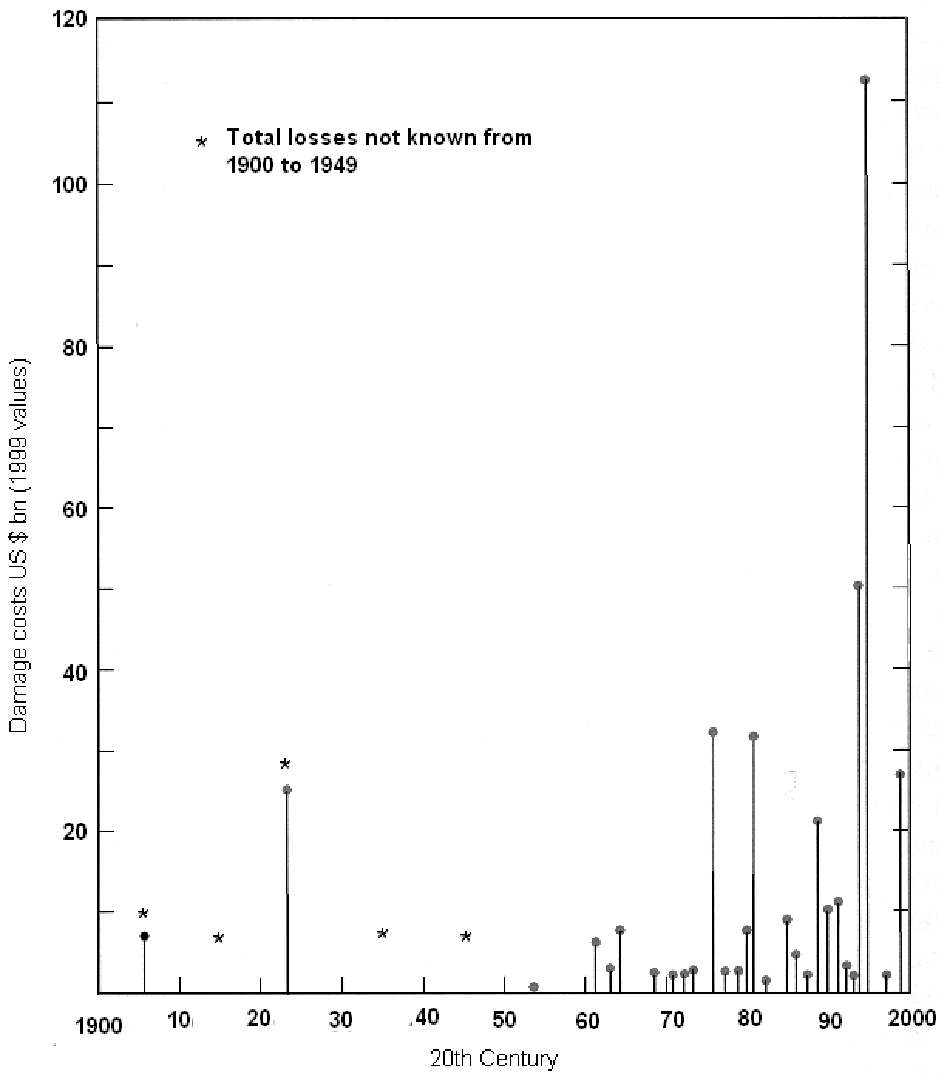


Figure 4.66: Total cost of earthquake material damage worldwide for each decade of the 20th century.

Post-earthquake economic consequences include:

1. Cost of death and injury
2. Cost of damage
3. Losses of production and markets
4. Insurance claims

The direct cost of damage depends upon the nature of the building or other type of facility, its individual vulnerability and the strength of shaking or other seismic hazard to which it is subjected. During the budgeting stages of a design, the cost of providing earthquake resistance will have to be considered, at least implicitly or sometimes explicitly such as for upgrading of older structures. The cost will depend upon such things as the type of project, site conditions, the form of the structure, the seismic activity of the region and statutory design requirements. Unfortunately it is not possible to give simple guides on costs, although it would not be misleading to say the most engineering projects designed for a seismically active area, would spend a maximum of 10% of the total cost on earthquake provisions with 5% as an average figure.

The cost of seismic upgrading of older buildings varies from as little as about 10% to more than 100% of the replacement cost, depending on the nature of the building, the level of earthquake loading used and the amount of non-structural upgrading that is done at the same time as the strengthening. Where the client simply wants the minimum total cost satisfying local regulations, the usual cost-effectiveness studies comparing different forms and material will apply. For this knowledge of good earthquake-resistant forms will of course, hasten the determination of an economical design whatever the material chosen.

In some cases, however, a broader economic study of the cost involved in prevention and cure of earthquake damage may be fruitful. These costs may be estimated on a probabilistic basis and a cost-effective analysis can be made to find the relationship between capital expenditure on earthquake resistance on the one hand and the cost of repairs and loss of income together with insurance premiums on the other. It is most important that at an early stage the owner should be advised of the relationship between strength and risk so that he can agree to what he is buying. Where stringent earthquake regulations must be followed the question of insurance versus earthquake resistance may not be a design consideration, but it can still be important, for example for designing non-structural partitions to be expandable. However, in some cases insurance may be more expensive or unavailable for facilities of high seismic vulnerability.

4.6.3 Social and Economic Impact of Bhuj Earthquake

The earthquake had a vast geographical spread, affecting 21 out of 25 districts of the state. Though Kachh and seven other districts—Surendranagar, Patan, Banaskantha, Jamnagar, Rajkot, Ahmedabad and Surat—were the most severely affected, the entire State of Gujarat reeled under the effects of the earthquake. There were about 13,800 deaths (about 20,000 deaths were initially estimated) in the earthquake, a very high level of mortality in a natural disaster. About 166,000 people were injured, out of which more than 20,000 had serious injuries. More than a million houses were destroyed or damaged. It was a collective trauma for the people of Gujarat. Table 4.1, compiled by the Government of Gujarat, provides details of the statewide impact of the earthquake. The initial estimate of deaths in the district Kachchh was close to 18,500. It seems a large number of missing people were also included in the category of dead. When the government checked these details later at the household level for the purpose of providing assistance to families of the dead, the number of deaths was revised downwards to 12,221.

Table 4.1: Details of affected districts in Bhuj earthquake

District	No. of Affected Talukas	No. of Affected Villages	Total Population	Affected Population	Human Deaths	No. of Injured
Kachchh	10	949	1,262,507	1,262,507	12,221	136,048
Ahmedabad	11	392	4,687,491	3,894,000	752	4,040
Rajkot	14	686	2,514,122	1,594,000	429	11,951
Jamnagar	11	685	1,563,558	1,563,000	119	4,930
Surat	8	94	3,397,900	397,989	46	190
Surendranagar	10	661	1,208,872	1,154,000	110	2,909
Banaskantha	8	452	2,013,519	719,000	32	2,770
Kheda	10	350	1,793,138	35,121	0	28
Bharuch	8	248	1,148,052	460,000	9	44
Gandhinagar	4	210	1,026,728	35,000	8	241
Patan	8	349	935,203	664,000	38	1,695
Junagarh	14	554	2,018,446	597,787	8	89
Navsari	5	331	1,085,692	87,783	17	52
Porbandar	3	157	376,113	376,113	10	90
Vadodara	6	85	3,039,127	186,092	1	270
Bhavnagar	11	535	2,060,315	445,226	4	45
Anand	8	124	1,647,759	4,687	1	20
Mehsana	9	611	1,648,251	1,648,251	0	1339
Sabarkantha	8	68	1,761,086	128,000	0	56
Amreli	11	273	1,484,300	599,000	0	5
Valsad	5	108	1,087,680	5,985	0	0
Total	182	7,904	37,759,859	15,857,541	13,805	166,812

In a disaster where over 13,000 people died, and almost an equal number were seriously injured, the economic and social impact on the affected families would obviously be very severe. Many of these impacts will be felt gradually, as families grapple with their sense of grief, loss of income and livelihood, and psychological trauma. According to a United Nations report, 94 unaccompanied children have been identified in all the affected Talukas to date. All of them are under the care of their close relatives and none is kept in any institution. Many of these details of losses are imprecise, as the household-based data are not available yet. However, it is true that a large number of children have been killed in the earthquake.

The disruption of social services too will have a long-term impact on the quality of life. Education and health facilities have suffered large-scale destruction in the area. Over 11,600 schools have been either damaged or destroyed. Destruction of hospitals, health care facilities, and child care centers will have a serious impact on provision of both curative and preventive services in the area, both much in demand in the aftermath of the earthquake. More than 20,000 people were seriously injured. The nature of injuries ranged from orthopedic and head injuries, to tissue losses, abdominal and thoracic trauma, and amputations. Widespread damages to surface and groundwater-based water supply schemes in the villages and cities will only exacerbate water scarcity for villagers. The state has experienced a series of natural disasters in quick succession. The loss of assets and livelihood resulting from the earthquake will have a serious impact on the level of consumption and welfare in the state.

4.7 DYNAMIC TESTING OF MODELS

Conventional earthquake-resistant design involves ensuring that a structure under design can carry a set of static lateral loads. The magnitude and distribution of lateral loads are specified by codes (e.g. Uniform Building Code) and, insofar as possible, simulate dynamic forces that the structure would experience in medium-sized earthquakes. The increasing availability of high-speed digital computers has initiated a trend whereby earthquake loads are represented by dynamic loads. Such methods involve dynamic analysis requiring, first, the idealization of the structure so that a mathematical model can be formulated and, second, the determination of the response of the mathematical model to suitable ground motion. Here it will be assumed that suitable ground motion is available and focus will be on the idealization of structures and the formulation of mathematical models.

The computation involved in performing the dynamic analysis associated with even a simple mathematical model is extensive. Hence, it is important that the mathematical model should be as simple as possible without omitting any features of the prototype that affect its dynamic behavior appreciably. Therefore, a considerable number of dynamic tests of real structures have been conducted in order to determine dynamic properties and establish mathematical models that can represent the dynamic behavior of the prototype structures.

A second purpose of these dynamic tests has been to accumulate a body of experimental results on the damping capacity of structures. Unlike the stiffness and mass properties of a structure, damping capacity cannot be calculated. Therefore, in formulating a mathematical model it is important that some experimental results on the damping capacity of structures similar to the one being modeled are available.

Since tests on real structures of necessity are conducted at low amplitudes, little information is found from such tests on the nonlinear behavior of structures. Thus dynamic tests have been conducted on small model structures vibrating at large amplitudes to study nonlinear behavior and energy absorption characteristics. Data from such tests then can be extended to an analytical evaluation of the nonlinear behavior of full-scale models. Thus some of the problems of formulating a mathematical model in a particular case are revealed. Subsequently, the mathematical model is subjected to ground motion to predict the prototype behavior under strong-motion earthquakes.

The quantities normally determined by a dynamic test of a structure are resonant frequencies, mode shapes, and damping capacities.

4.8 CONCLUSIONS

As we know, earthquakes have occurred from millions of years without warning and will continue in future. There is a need to recall lessons repeatedly taught by past earthquakes. Ground damage observed in field surveys immediately after several earthquakes is well documented. Damage in the form of ground rupture, liquefaction, landslides and tsunamis and many others effect in a wide geographical region. This kind of damage study is a timely reminder of what can happen in the seismically active areas of the country.

It is essential to mitigate the effects of strong earthquake shaking to reduce loss of life and damage. Structural damage is the leading cause of death and economic loss in many earthquakes getting influenced by strength of shaking, duration of shaking, type of subsurface conditions and type of building. Major causes of damage to buildings during earthquakes include plan asymmetry, soft story, settlement, torsion, pounding damage and resonance. Study of behavior of various building structures during earthquakes based on the past experiences gives insight for preparedness in future.

Earthquakes also disrupt the lifelines which are critical in providing emergency response recovery, and in restoring society to normal. Lifeline systems include transportation systems, electric power, telecommunications, water, sewer, gas and liquid fuels – natural and liquid petroleum gas (LPG). All behave differently and exhibit varying vulnerability towards earthquakes.

An earthquake always have social and economic consequences on the settlements including cost of death and injury, cost of damage, losses of production and markets and the insurance claims.

These days our structures can be tested for their susceptibility to earthquakes through dynamic testing models. These dynamic tests are conducted on small model structures vibrating at large amplitudes to study nonlinear behavior and energy absorption characteristics. Data from such tests can be extended to an analytical evaluation of the nonlinear behavior of full-scale models. In this manner we can take necessary actions to reduce the vulnerabilities of our structures in order to minimize the disastrous impact of earthquakes.

CHAPTER 5

SITE PLANNING, BUILDING FORMS AND ARCHITECTURAL DESIGN CONCEPTS FOR EARTHQUAKE RESISTANCE

CHAPTER 5
SITE PLANNING, BUILDING FORMS AND ARCHITECTURAL DESIGN CONCEPTS FOR EARTHQUAKE RESISTANCE

5.1 INTRODUCTION

Site Planning and building form are very important from the point of view of seismic performance of the structure. Also it is very important to have proper compatibility of elements resisting seismic forces. Site planning and selection of building forms is the first step in designing of any structure and the decisions taken are very crucial for the behavior of those structures during any earthquake. Based on examples taken from various past earthquakes and theories this chapter gives guidelines for site planning, selection of building form and formulation of architectural design concepts for earthquake resistant structures in the earthquake prone areas.

5.2 HISTORICAL EXPERIENCES

Compliance to site regulations is very important and has a historical basis. Even Napoleon had imposed regulations for compliance as for loss of serviceability in a construction project within 10 years of construction; say foundation failure or poor workmanship; the contractor and the architect were to be sent to prison. Whether such and other rules from ancient times stopped all failures is not known, but they certainly were a deterrent to shoddy construction practices and eliminated the possibility of repetitive malpractices. Present day law and order demands accountability for professional performance. The lessons learnt regarding various architectural design concepts, from past earthquakes are discussed in chapter 4 of this book.

5.3 SITE SELECTION

The selection of suitable site is a crucial step in the design of a building or planning a settlement in an earthquake prone area. There are a number of earthquake related hazards which should always be considered when choosing a site, together with the influence of the ground conditions at the site on the ground motion which the building may experience in a future earthquake. An assessment of the extent of the earthquake hazard should always form a part of the overall site assessment and of the specification for the design of any structures to be built there. No site can be expected to be ideal in all respects, so the choice of site will often involve a judgment about relative risks and the costs of designing to protect from them. But there can be some sites which could be so hazardous that they should be avoided if at all possible, since the cost of building is likely to be prohibitive. A few important considerations for site selection are given below:

5.3.1 Steep Slopes

Buildings should be sufficiently away from steep slopes. Sites located on or very close to steep slopes are always prone to landslides, especially in the earthquake prone regions. (Fig. 5.1) Even if the building has good earthquake resistant construction, they are prone to damages or total destructions on such sites. Frequent landslides in Uttarkashi region after the October 1991 earthquake caused massive devastations in the region. The periodic landslides are triggered by other aspects like excess rains, seepage etc. The Himalayan regions are particularly prone to landslides. Such landslides often prove to be more disastrous than the actual earthquake event.

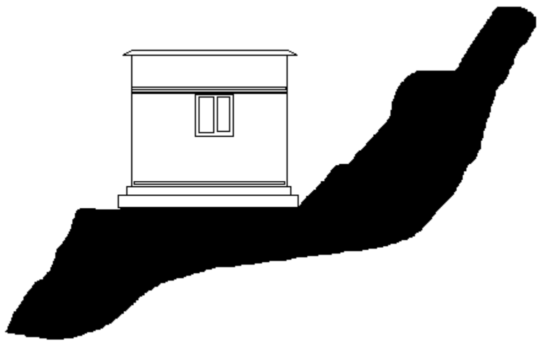


Figure 5.1: Buildings located near steep slopes

5.3.2 Filled Up Soil

Foundation should rest only on firm soil and not on filled up soil. Such constructions on filled up soils have witnessed extensive damages in the January 2001 Gujarat earthquake. (Fig 5.2a)

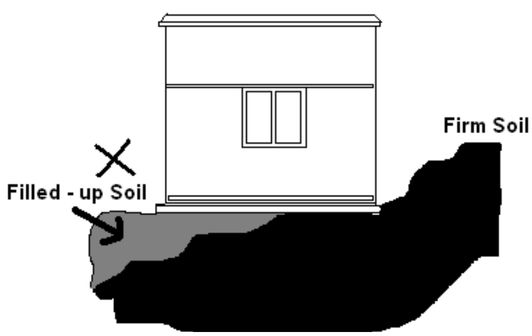


Figure 5.2 a: Buildings located on filled – up soil should be avoided

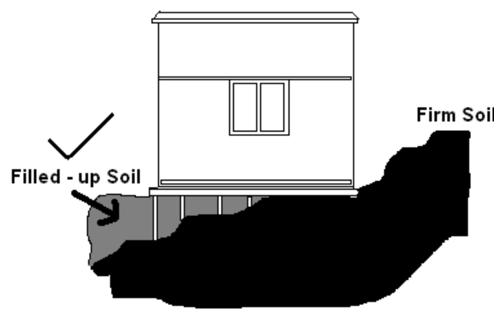


Figure 5.2 b: Buildings located on filled – up soil should be provided with raft on pile foundations

5.3.3 Raft on Pile Foundations

Many times it is unavaoidable to construct the structure on filled up soil, as in most cases choice of site is not the option we have. In such situations raft on pile foundations have to be provided as shown in Fig 5.2 b.

5.4 BUILDING FORMS

Building form has to be decided initially in the design process. Different aspects of building forms viz. scale, height, horizontal size, proportion and symmetry are discussed below:

5.4.1 Scale

A large masonry building is always in contrast with a small wood frame building which can be made a seismically safe structure with the inclusion of relatively inexpensive and unobtrusive provisions. This is because a small wood structure is lightweight and the internal forces will be low. In addition the spans are relatively small and there would be large number of walls to distribute the loads. On the other hand, for a larger building, the violation of basic layout and proportion principles result in an increasingly high cost as the forces become greater and the good performance becomes difficult as compared to an equivalent building.

As the absolute size of a building increase, the number of options for its structure design decreases. A bridge span of 100 meters may be designed as a beam, arch, truss or suspension system, but if this span increases to 1000 meters the structural discipline becomes more rigorous and the design options become limited. The architectural solutions that are perfectly acceptable at the size of a simple structural system like house become physically impossible at the scale of large spans like suspension bridge.

<u>It is not possible to alter the size of a structure and its components and still retain the same structural behavior.</u>

Every building has to be considered differently with increase in size, as the size of the building influences its seismic performance.

5.4.2 Height

Increasing the height of a building may be similar to increasing the span of a cantilever beam. As the building grows taller there is a change in the level of response to the seismic forces. The effect of the building period therefore, must be considered in relation with the period of ground motion and its amplification. The effect of an increase in height may be quiet disproportionate to the increase in seismic forces itself. Thus the doubling of the building height from 5 to 10 storeys may, if amplification occurs, result in four or five fold increase in seismic forces. These concepts are already explained in section 3.5 and 3.6.

5.4.3 Horizontal size

It is easy to visualize the overturning forces associated with height as a seismic problem, but large plan areas can also be detrimental. When the plan becomes extremely large, even if it is symmetrical, simple shape, the building can have trouble responding as one unit to earth vibrations.

Increase in length of a building increases the stresses in a floor working as a horizontal distribution diaphragm in a transverse direction. The rigidity of the floor may not be sufficient to redistribute the horizontal load during an earthquake from weaker or damaged supporting elements of the building to stronger elements or those with minor damage.

Unless there are numerous interior lateral force resisting elements, large plan buildings impose unusually severe requirements on their diaphragms, which have large lateral spans and can build up large forces to be resisted by shear walls or frames. The solution is to add walls or frames that will reduce the span of the diaphragm, though it will reduce flexibility in the use of the building. (Fig 5.3)

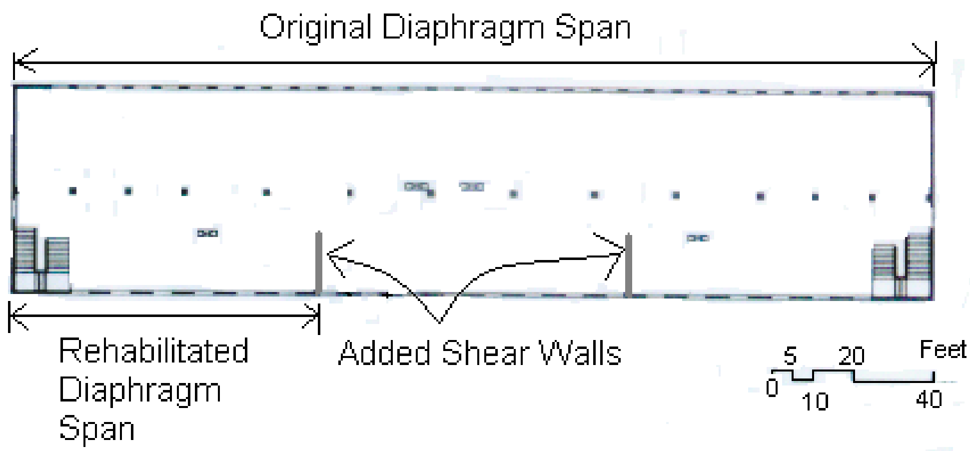


Figure 5.3: Addition of shear walls to decrease span of diaphragm

5.4.4 Proportion

In seismic design, the proportions of a building may be more important than its absolute size. For tall buildings the slenderness ratio of a building is one of the important considerations than just the height alone. The more slender the building is worse are the overturning effects of an earthquake and greater are the earthquake stresses in the outer columns, particularly the overturning compressive forces, which can be very difficult to deal with. Some experts suggest limiting the height / depth ratio to 3 or 4, to safeguard the building against overturning.

As the urban land is becoming more scarce and expensive, there is a trend to design slender buildings, which although not necessarily very high, may have a large height / depth ratio. (Fig 5.4) This trend is clearly apparent in downtown Tokyo, where multistoried buildings were built on sites that are only 5 to 6 meters wide. At the same time the economic forces often dictate the distance between the two buildings which were very close and at times they tend to respond as one unit rather than as individual freestanding buildings.

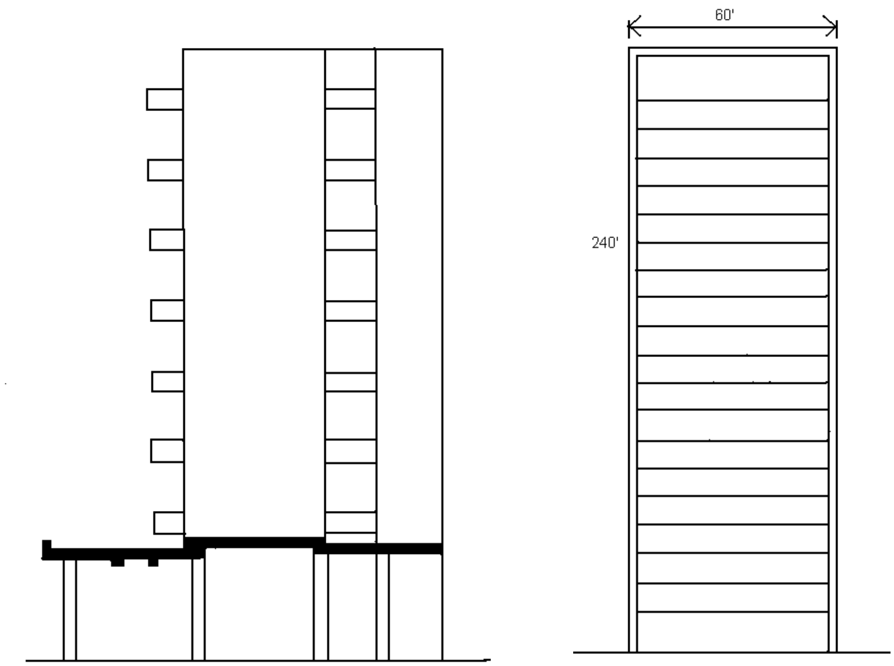


Figure 5.4: Unusually High Building

5.4.5 Symmetry

The plan shape of a building should be as simple as possible. A theoretical optimum shape is a round tower, where as long buildings, L-shaped or zigzag shape or buildings with attached wings are undesirable in the high risk areas and therefore should be avoided. (Fig 5.5)

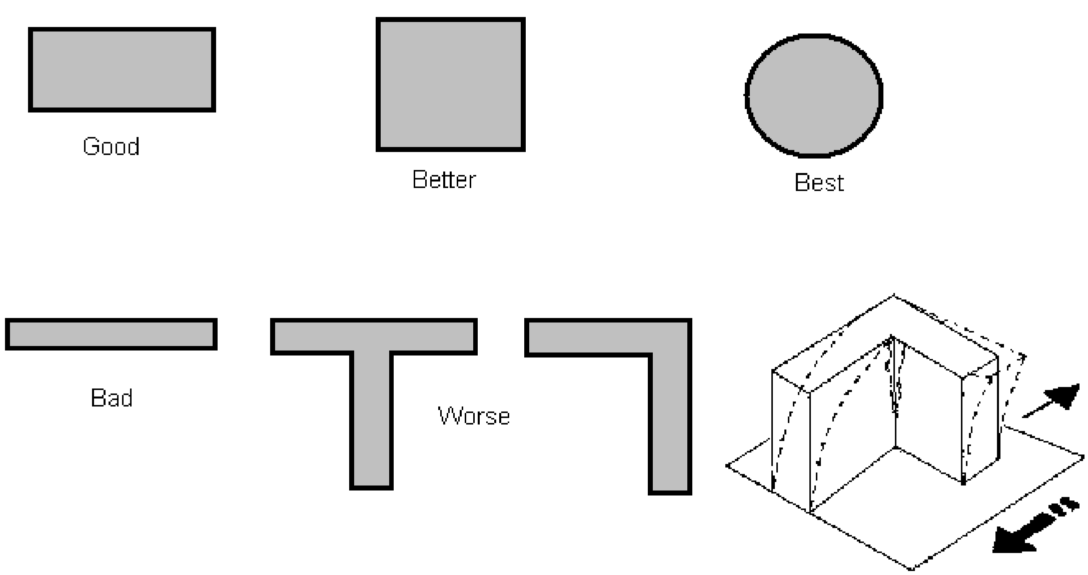


Figure 5.5: Shape of Buildings

The term symmetry denotes a geometrical property of the plan configuration, whereas the structural symmetry means that the center of mass and the center of resistance are located at the same point. In asymmetrical configuration / structural system the eccentricity between the center of mass and resistance will produce torsion and stress concentration and therefore the symmetrical forms are preferred to the asymmetrical ones. (Fig 5.6)

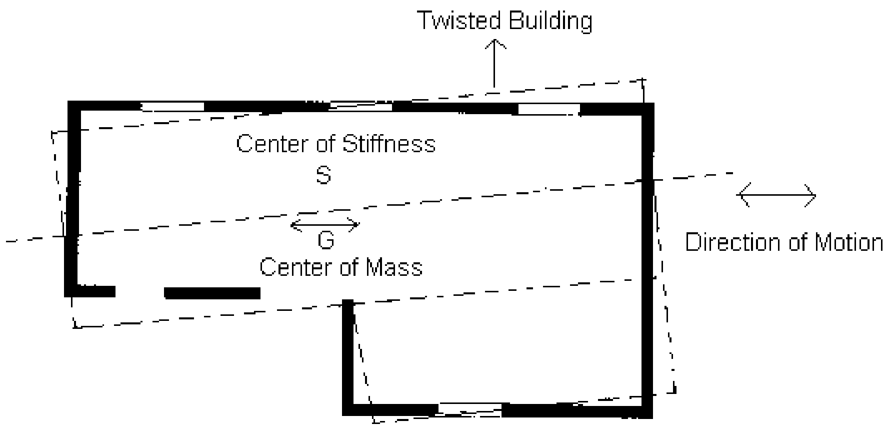


Figure 5.6: Torsion of Unsymmetrical Building Plan

Thus it is amply clear that as the building becomes more symmetrical its tendency to suffer torsion and the stress concentration will reduce and performance under seismic loads tends to considerably improve. This suggests that when good seismic performance has to be achieved along with maximum economy of design and construction, the simple, regular and symmetrical shapes are much preferred. However these tendencies must not be mistaken for an axiom that the symmetrical building does not suffer torsion.

5.5 SEISMIC EFFECTS RELATED TO BUILDING CONFIGURATION

Building configuration refers to the size, shape and proportions of the building form. From seismic point of view configuration may also include the location, shape and approximate size of structural elements as these elements are often determined based on the architectural design decisions. This extended definition of configuration is necessary because of the intricate relationship of seismic performance between these elements. In general the architectural configuration depends on:

1. Architectural design
2. Functional requirements
3. Urban design parameters
4. Planning considerations
5. Aesthetic appearance
6. Identity (distinctiveness)

The earthquake forces depend on mass and stiffness distribution, the material size and shape of the building establish it's mass. Stiffness is directly related with the type of configuration. For the same overall size and shape of the building various configuration can provide a solution.

The codal provisions for earthquake resistant buildings are based on simple, symmetrical and uniform building configurations. Their application to unusual / irregular building configurations, therefore may lead to unrealistic evaluation. It is important to understand about regular and irregular configurations, before taking the architectural design decisions.

Any configuration, whether regular or irregular, will have some resistant system, to take the lateral forces, which acts in horizontal and vertical planes. In vertical plane there are shear walls, braced frames and moment resisting frames whereas in horizontal plane the lateral forces are resisted by diaphragms formed by floor and roof slabs of the building. The presence of these resistant systems is the result of schematic architectural design.

5.5.1 Regular Configuration

Regular configuration shown in Fig 5.7 is seismically ideal. These configurations have low heights to base ratio, symmetrical plane, uniform section and elevation and thus have balanced resistance. These configurations would have maximum torsional resistance due to location of shear walls and bracings. Uniform floor heights, short spans and direct load path play a significant role in seismic resistance, of the building.

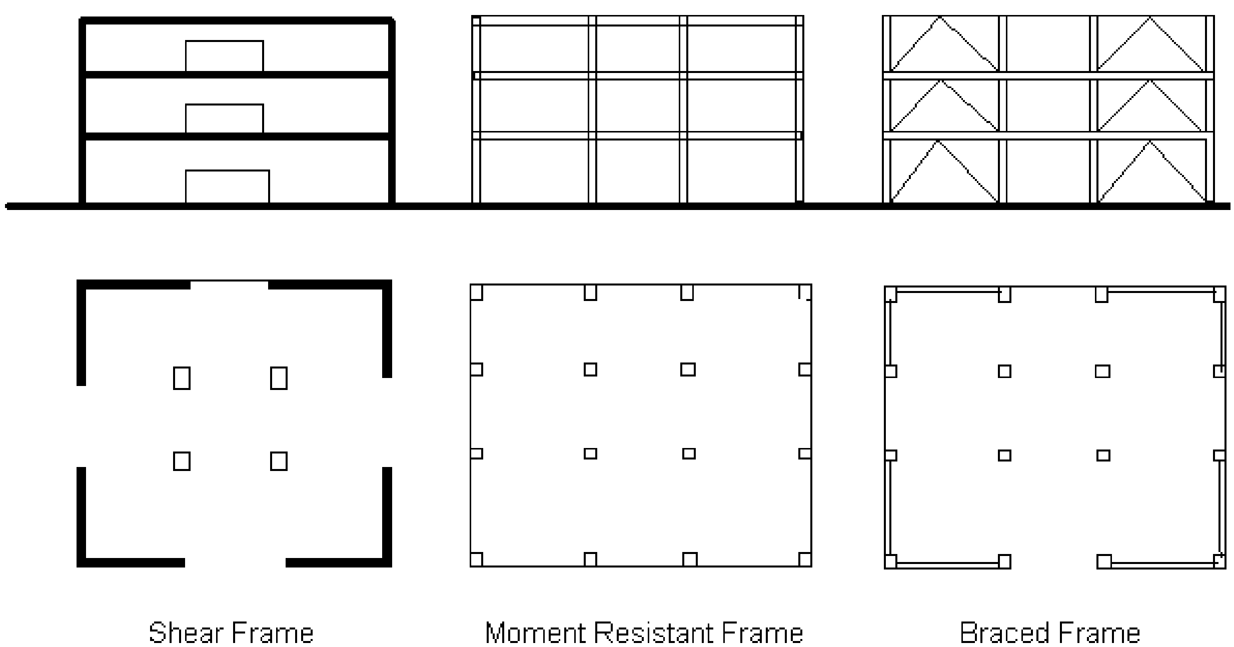


Figure 5.7: The optimal (regular) seismic configuration

5.5.2 Irregular Configuration

Some of the common irregular building configurations as defined by some American Codes are as shown in Fig 5.8. These codes permit the use of equivalent static force method in these irregular buildings with some zonal constraints as the height and stiffness changes between two stories. Fig 5.9 a and 5.9 b is the graphical representation of irregular structures or framings systems.

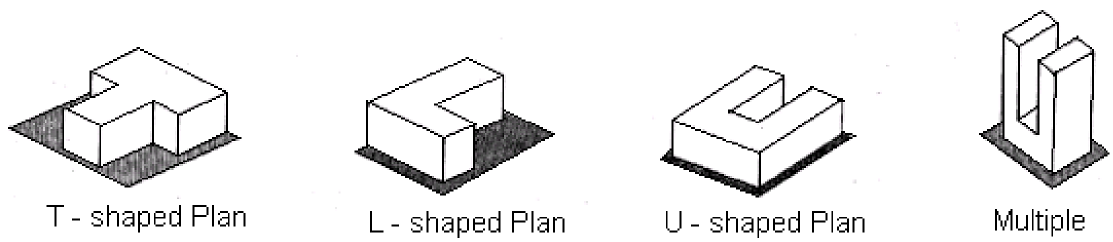


Figure 5.8: Buildings with irregular configuration

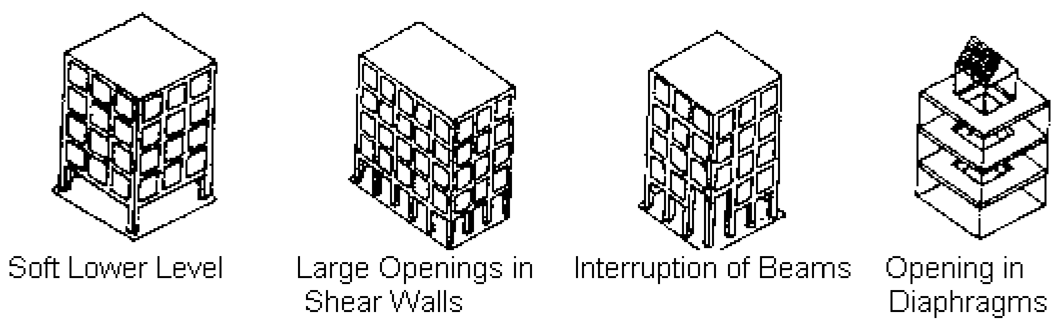


Figure 5.9 a: Buildings with abrupt changes in lateral resistance

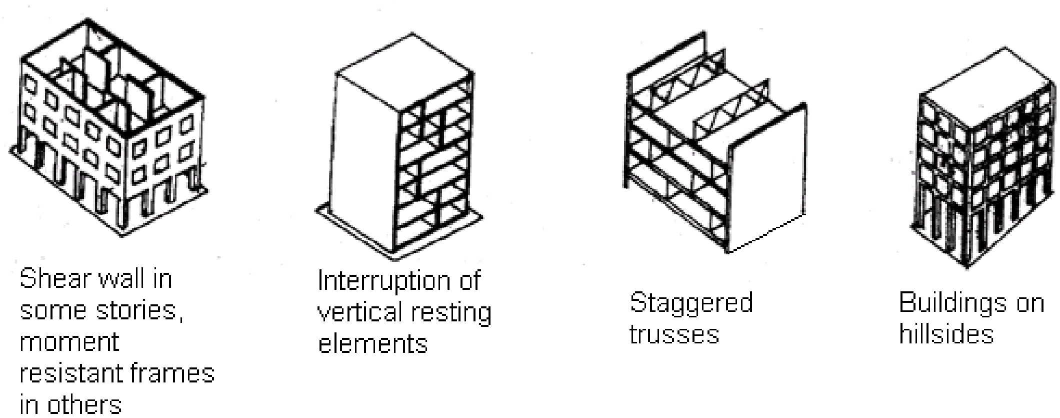


Figure 5.9 b: Buildings with abrupt changes in lateral stiffness

5.5.3 Influence of Configuration on Seismic Performance

To understand the influence of configuration on seismic performance one should understand the ways in which the building responds to the dynamic forces due to motion of the ground. Static vertical loads are directly transferred down to the round through foundation. The earthquake exerts fluctuation dynamic loads, it is difficult to determine the seismic forces without knowing and understanding the dynamic characteristic of the building along with the sequence of events and the behavior of different elements of the building structure under dynamic loads.

The forces that are exerted on the building elements and the exact nature of their resultant behaviour are quiet complex and should be taken into account while taking the decision regarding the building configuration. These forces are shown in Fig 5.10. below.

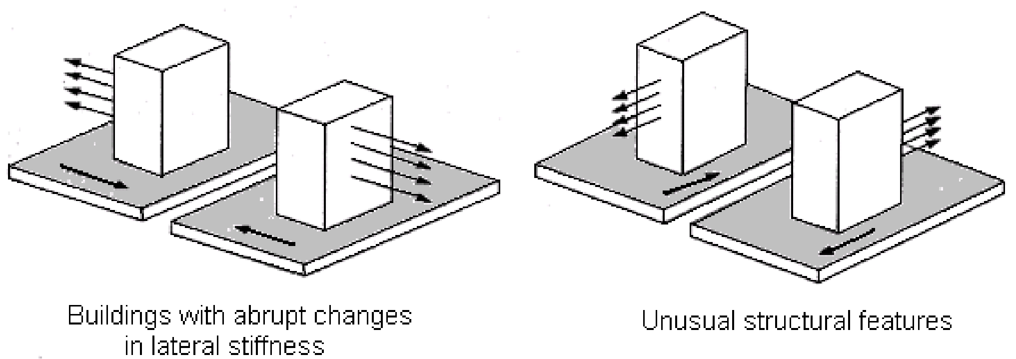


Figure 5.10: Typical analytical diagram of earthquake forces

The above diagram originates in the form of typical seismic design analysis in which earthquake forces are separately applied to each of the main axis for a rectangular shape and for a circle there would be more axis which are similar, (more stable) however for irregular shape which is complicated we may have to look at along several axis as shown in Fig 5.11 below.

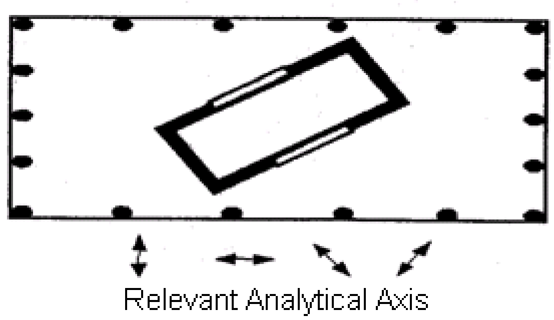


Figure 5.11: For complicated configuration more than two axes may be used for analysis

In fact the earthquake forces may come from any direction, however, the forces perpendicular to the major axes of walls or frames usually simulates the worst direction. If the ground motion and its resulting forces occur diagonally then the walls or frames along both X axes and Y axes can participate in negating the resistance and the forces in each of the wall or frame will be considerably reduced. (Fig. 5.12)

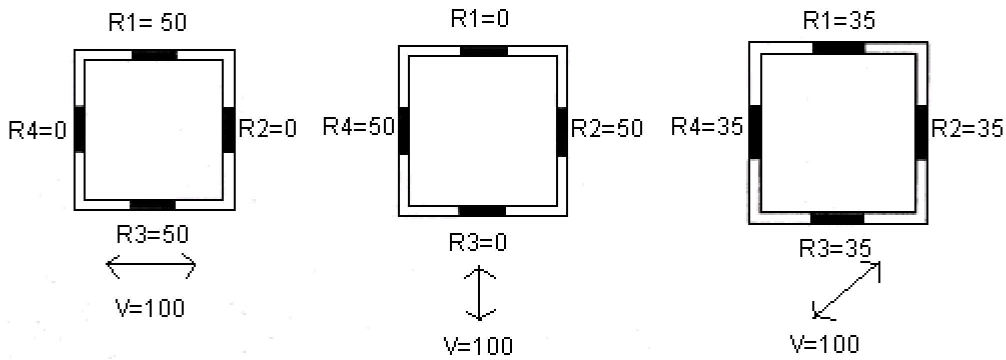


Figure 5.12: Earthquake forces on buildings

It is important to note that in reality the earthquake forces are much more complex then our diagrams would indicate. This is because the ground motion is random and the main direction of emphasis will only be axial by chance. In any event, the total ground motion will always include non axial components also. Thus a better diagram for visualizing building configuration related to reaction to the ground motion will be as shown in Fig 5.13.

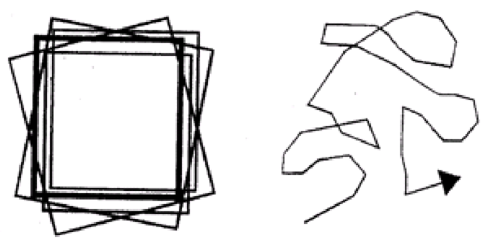


Figure 5.13: Earthquake forces: The reality

A building is not a homogeneous block but an assembly of parts, and each part is subjected to earthquake forces horizontally and vertically and from adjoining parts through joints. In a large building the ground motion affects different parts of the building differently. These forces induce torsion or incompatible movement, even in a geometrically symmetrical building as shown in Fig 5.14

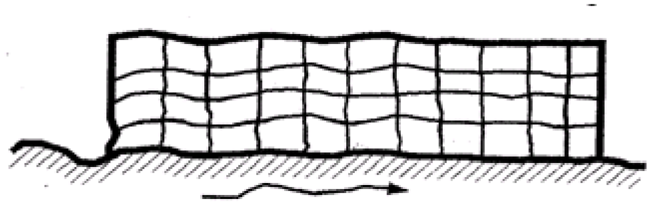


Figure 5.14: Earthquake forces acting on a large building

The building being made of parts which are joined together by means of different connections will have different localized strengths and stiffness, some calculated and some inadvertently caused by interaction of non structural elements or configuration influence. This further differentiates its behavior from that of a homogeneous building block. (Fig.5.15)

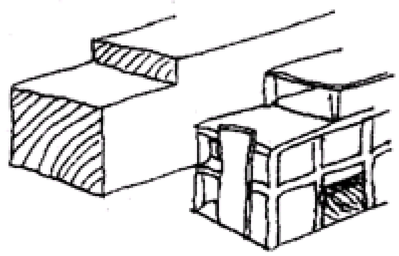


Figure 5.15: Localized strength and stiffness

5.6 PLAN AND VERTICAL IRREGULARITIES, REDUNDANCY AND SETBACKS

Many times it has been seen that geometrically building may appear to be regular and symmetrical, but it may have irregularity due to distribution of mass and stiffness. It is always better to distribute the lateral load resisting elements near the perimeter of the building rather than concentrate these, near centre of the building (Fig 5.16). As a general rule, buildings with irregular configuration perform poorly in earthquakes even when good engineering has been carried out.

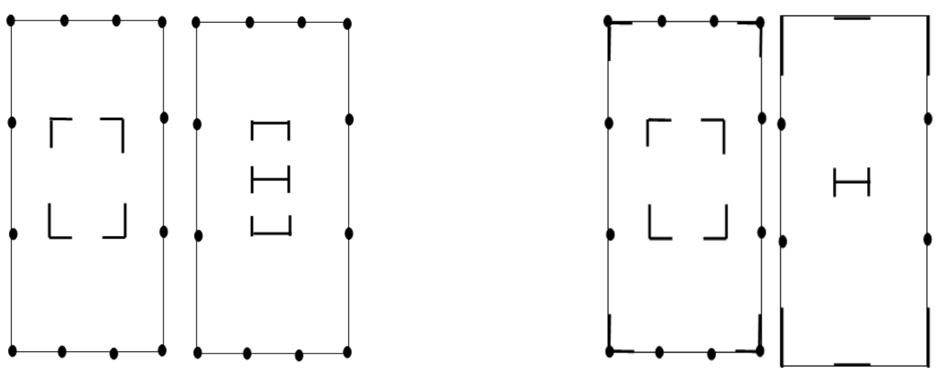


Fig 5.16 a

Fig 5.16 b

Note that the heavy lines indicate shear walls and/or braced frames

Figure 5.16 a: Arrangement of shear walls and braced frames- not recommended.

Figure 5.16 b: Arrangement of shear walls and braced frames recommended.

5.6.1 Plan Irregularity

Plan irregularities are already discussed in section 5.6.2 of this chapter. An example of discontinuity in diaphragm stiffness leading to plan irregularity is shown in Fig 5.17 and Fig 5.18.

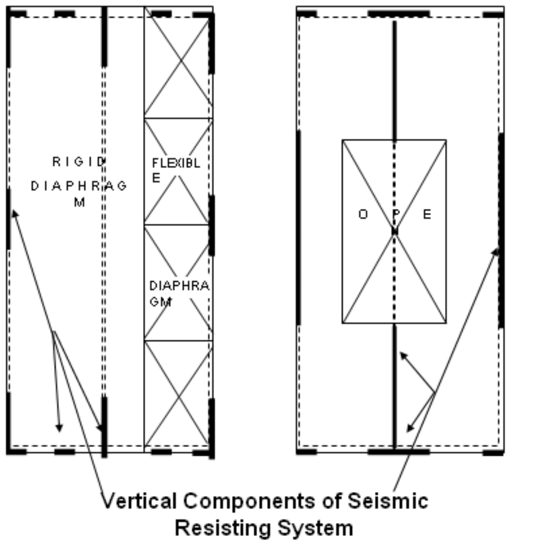


Figure 5.17: Discontinuity in Diaphragm Stiffness

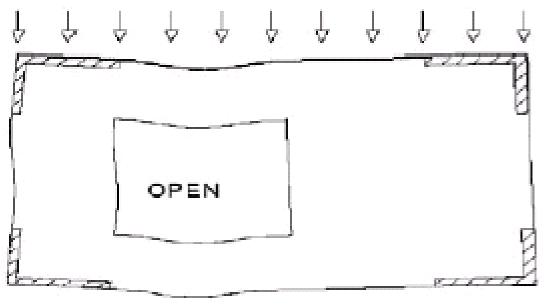


Figure 5.18: Diaphragm openings

5.6.2 Vertical Geometric Irregularity

All buildings with vertical offsets fall in this category. Also, a building may have no apparent offset, but its lateral load carrying elements may have irregularity. For instance, shear wall length may be suddenly reduced. When building is such that larger dimension is above the smaller dimension, it acts as an inverted pyramid and is undesirable.

Dynamic analysis is required in buildings with vertical irregularity where load distribution with building height is different. In buildings with plan irregularity, load distribution to different vertical elements becomes complex. In such cases floor diaphragm plays an important role and needs to be modelled carefully. For such buildings a good 3-D analysis is needed.

In irregular building, there may be concentration of ductility demand in a few locations. In such buildings just dynamic analysis may not solve the problem and special care is needed in detailing.

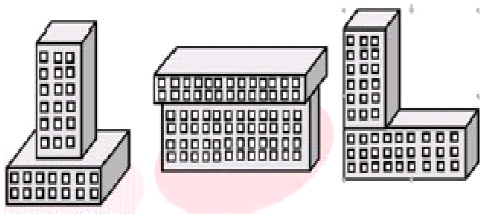
<u>Dynamic analysis is not always sufficient for irregular buildings and dynamic analysis is not always needed for irregularities.</u>

5.6.3 Projections

All projections (vertical and horizontal) are most vulnerable to damage during earthquakes. As they are cantilevers, there is no redundancy, and hardly any ductility. Design of such projections has to be five times the seismic coefficient. This is same as in the international practice11.

Out of Plane Offsets

This is a very serious irregularity wherein there is an out-of-plane offset of the vertical element that carries the lateral loads. Such an offset imposes vertical and lateral load effects on horizontal elements, which are difficult to design for adequately. Shear walls are not obvious. (Fig. 5.19 and 5.20)



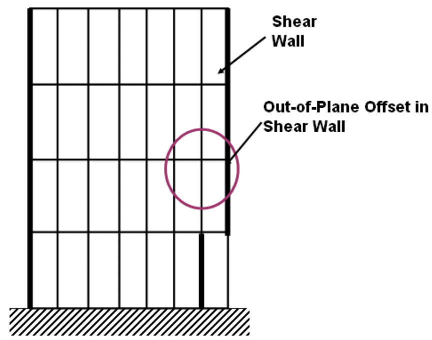


Figure 5.19 : Buildings with offsets

Figure 5.20: Damage to buildings with offsets

5.6.2 Re-entrant Corner

When an otherwise regular building has a large re-entrant corner, wings of the building tend to vibrate in a manner different from that of the entire building. Hence, building is treated as irregular when offset dimensions exceed (Fig 5.21).

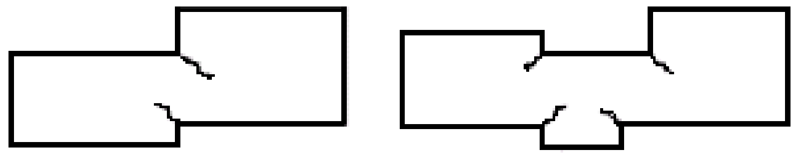


Figure: 5.21: Re-entrant Corner

5.6.3 Mass Irregularity

Mass irregularity is induced by the presence of a heavy mass on a floor, say a swimming pool. In IS1893 the mass irregularity has been defined as a situation when weight of a floor exceeds twice the weight of the adjacent floor. NEHRP defines it when the weight exceeds 150% of that of the adjacent floor. Fig 5.22

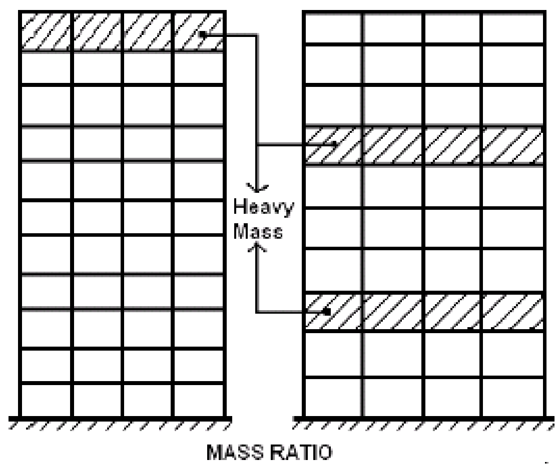


Figure 5.22: Irregularity in masses

5.7 CONCLUSIONS

The effects of building form and configuration refers not only to the overall building shape but to its design and constructional details also. Study of building performance in the past earthquakes indicates that the performance is quiet sensitive to even a small variation in the overall form. This is particularly true in relation to shear wall design and the location of service core, which acts as a major lateral resistance elements. Many buildings that were symmetrical and simple in overall plan suffered because of unsymmetrical location of service cores and escape staircases. Moreover, as soon as the structure begin to suffer damage (cracking in shear wall or columns) the distribution of its resistant elements change and thus most symmetrical structure becomes dynamically asymmetrical and is subjected to torsional forces.

Finally it must be recognized that the architectural requirements will often make asymmetrical design difficult or sometimes impossible. In these circumstances it is necessary, depending upon the size of the building and the type of asymmetry, to subdivide the major masses of the building to improve the seismic performance.

CHAPTER 6

SEISMIC DESIGN PRINCIPLES

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SEISMIC DESIGN PRINCIPLES

6.1. INTRODUCTION

Experiences from past earthquake have taught us that "Earthquakes don't kill people but buildings do". Some buildings perform satisfactorily under shaking, whereas others could not. Bhuj 2001 earthquake has shown that so-called engineered buildings may collapse like house of cards, if not designed according to safe practice norms. Seismic design principles are derived and updated as the knowledge and understanding of building behavior under shaking is incremental. A study of structural performance of buildings during the past earthquakes indicates that commonly employed constructions are not earthquake resistant and therefore require improvement in design and construction techniques.

Earthquake resistant design is possible by:

i) Proper planning;
ii) Design, and
iii) Construction details to make them collapse proof and withstand damage within acceptable limit. Elastic design is not justified because of prohibitive cost.

6.2. OBJECTIVES OF EARTHQUAKE RESISTANT DESIGN

Objectives of earthquake resistant design are discussed below:

(a) The IS 1893 (Part1): 2002 says that the Design Basis Earthquake (DBE) can be reasonably expected to occur at least once in the design life of the structure. And buildings are expected to withstand DBE without major damages. Maximum Considered Earthquake (MCE) is the most severe earthquake effects considered by this Code to happen to any building, and no collapse should happen to it due to MCE.

(b) Under minor but frequent shaking (serviceable earthquake, up to intensity VI) :
- The main members of the building that carry vertical & horizontal forces should not be damaged.
- However building parts that do not carry load may sustain repairable damage.
- In case of buildings of post-earthquake importance even non-structural elements should not undergo any displacement or distortion, otherwise loss of critical function would be incurred.

(c) Under moderate but occasional shaking (design earthquake, from intensity VII to VIII):
- The main members may sustain repairable damage.
- The other non-load bearing parts of the building may be damaged such that they may even have to be replaced after the earthquake.
- In case of buildings of post-earthquake importance, non-structural elements should undergo only predicted damage or distortion, otherwise loss of critical function would be coupled with unlimited closure period thus resulting in higher economic loss.

(d) Under strong but rare shaking (maximum limit earthquake, from intensity IX to XII)

- the main members may sustain severe and even irreparable damage
- but the building should not collapse
- and people can be safely evacuated.

Implication of objectives of earthquake resistant design is that the maximum expected earthquake load is much larger than Design Earthquake Load.

6.3. BASIC TERMINOLOGY

6.3.1 Strength

Ground vibration is random in magnitude and direction and has two horizontal components and one vertical component. Vertical vibration initiates vertical inertia force in the structure, which gets added or subtracted to gravity force. In general, factor of safety adopted for gravity load design is high enough to take care of additive vertical component for a safe structure. Horizontal vibration components introduce horizontal inertia force. To transfer this load to ground safely, a complete load transfer path is required. The elements in that load path should have adequate strength to combat duly generated stress. Strength is a material property; so selection of material inherently governs the stress limit the element can be subjected to.

6.3.2 Stiffness

Deflection under loading is a measure to understand stiffness of any element. This is a property of an element, its material, cross section, unsupported length or height; Stiffness prevents the structure or its parts from moving out of alignment more than permissible limit. This is also referred as horizontal drift or storey-to-storey drift. In a structural system, relative stiffness or rigidity amongst different elements are of serious concern in seismic analysis; though it is not of same concern in case of gravity load. Conventional assumption is that if a structure is subjected to certain force/s, and it is a combination of two or more elements, then load sharing would occur in proportion to their relative stiffness.

Low lateral stiffness leads to large deformation and more damage in inelastic response, significant P-Δ effect, damage to non-structural elements due to large deformation etc.

6.3.3 Period

Concept of period has been discussed in Chapter 3 in detail. One important thing to remember is that, the overall stiffness of a building is measured by its period. Flexible, tall buildings have extended time period. (see section 3.5 and section 7.5.2)

As per IS 1893 (Part I) - 2002, the approximate natural period of vibration (T_a), in seconds, of a moment-resisting frame building without brick infill panels may be estimated by the empirical expression:

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

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

Where, h= Height of building, in m.

(This excludes the basement stories, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement stories, when they are not so connected.)

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

$T_a = 0.09/\sqrt{d}$, where,

h= Height of building, in m. (as discussed above)., and

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

6.3.4 Centre of mass

IS 1893(Part1):2002 defines Centre of Mass as the Point through which the resultant of the masses of a system acts. This point corresponds to the Centre of gravity of masses of system.

6.3.5 Centre of rigidity or stiffness

Centre of rigidity is the geometric centre of the relative rigidities of all elements bracing the structure in both directions. It is also referred as the shear centre or centre of rotation, meaning that during a seismic event the structure would rotate about its centre of rigidity. IS 1893 (Part!): 2002 defines centre of rigidity or stiffness as the point through which the resultant of the restoring forces of a system acts (Fig 6.1).

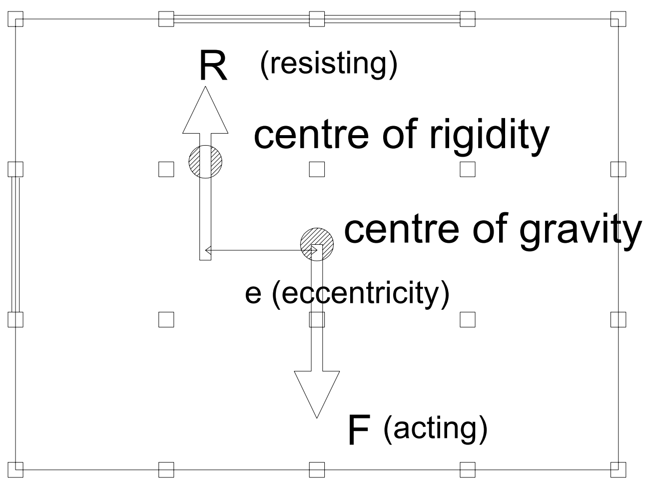

Figure 6.1: Eccentric centre of gravity and centre of stiffness

6.3.6 Torsion

Centre of gravity or mass of any structure pass through a point where it would be balanced against any rotation. Any load, when uniformly distributed, then the point would coincide with geometric centre of the structure. If this centre of mass coincides with centre of rigidity, then there would be no rotation effect. The earthquake force assumed to act at the centre of mass of the structure, and the resistance of the building would pass through the centre of rigidity. Asymmetry as far as mass and rigidity/ stiffness distribution is concerned, would experience a rotational problem. This is referred as Torsion. And as a result twisting would occur to it, which is undesirable. Not only in plan form, asymmetry has to be checked in 3 dimensional configuration of a structure to arrest the rotation phenomenon. (see section 4.3.4)

6.3.7 Design eccentricity

The offset between the centre of mass of any structure and the centre of rigidity/stiffness of the safe is referred as eccentricity. IS 1893(Part1): 2002 refers Design Eccentricity (e_{di}) as the value of eccentricity to be used at floor i in the torsion calculations for design.

6.3.8 Damping

Damping is an inherent quality of structure, and it controls or dampens response of a building during earthquake shaking. This can be visualized by comparing it with a swinging door without and with damper. In the first case it would continue to swing unless the door is being stopped forcefully. This is an example of un-damped or under-damped situation (gradually swinging becomes lesser and then stopped). In case of door with damper, the door would be stopped slowly without going for to- and fro- movement. It can be said that critical damping has been introduced in the system. Critical damping is defined as the smallest value of damping at which the element or the structure experience no cyclic motion and gradually return to neutral position (Fig 6.2). (see section 3.7)

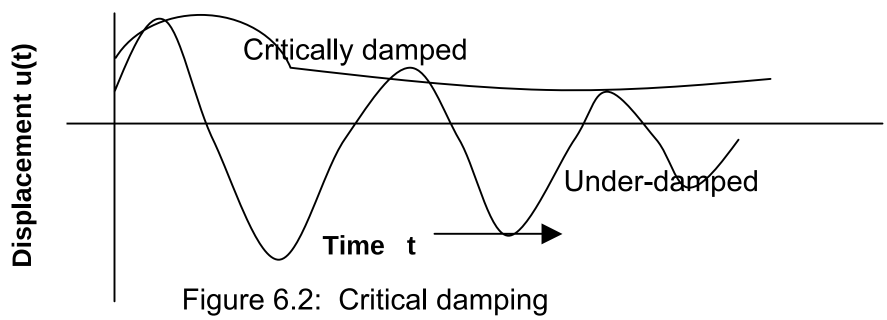

Figure 6.2: Critical damping

6.3.9 Ductility

A structure or its components whose deformation vanishes rapidly with the disappearance of the loads is said to behave elastically. All structural systems are elastic and that too in a linear fashion to a certain extent. Elements presenting permanent deformations after the disappearance of the loads are said to behave plastically. Although large permanent deformations are to be avoided, it must not be thought that plastic behaviour above the elastic range makes an element or a structure unsuitable for structural purposes; in fact the opposite is true. Above the yield point (where the structure starts behaving in a clearly plastic fashion) deformations increase more rapidly than the loads and eventually keep increasing even if the loads are not increased. This flow or yield is thus the clearest sign, and a healthy warning, that failure is imminent. And this quality if a structural material or a system exhibits it is referred as ductility.

Ductility is one of the strongest tools to design earthquake resistant structure. It offers the element to deform a large extent absorbing energy and thus can resist earthquake force better deferring collapse mechanism. Ductility can be expressed as:

$$\text{Ductility, } \mu = \frac{\text{Maximum Deformation}}{\text{Yield Deformation}} = \frac{\Delta_{max}}{\Delta_y}$$

Deformation may be measured in terms of deflection, rotation or curvature.

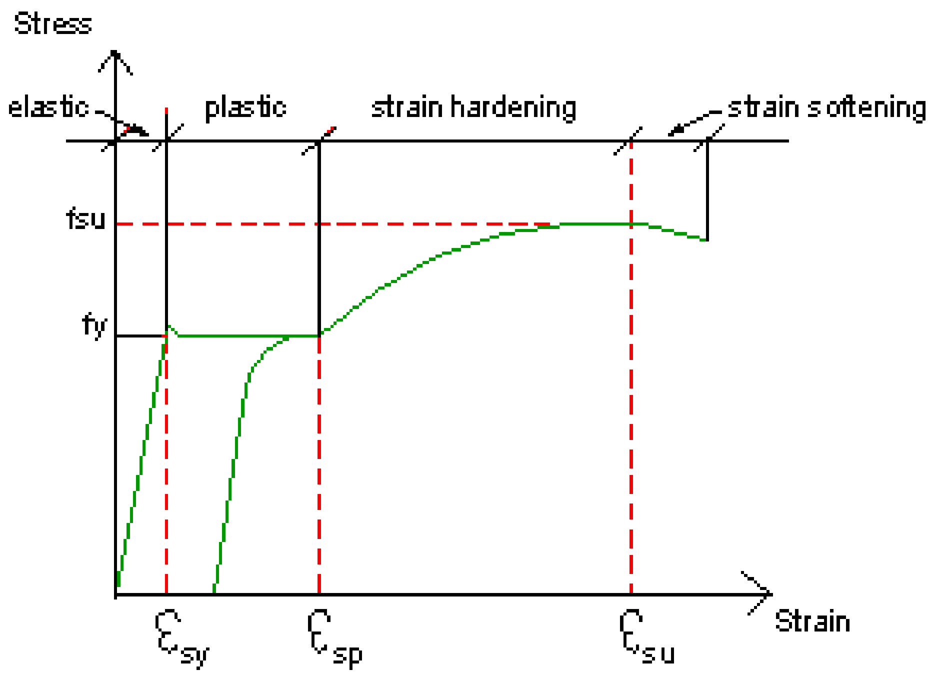

Figure 6.3: Elastic and Plastic behavior of mild steel

Increasing redundancy in the structure would result in improved ductility. Designers can predetermine for some elements to undergo large inelastic actions. At different level ductility would mean different displacement ratio: Section ductility (moment vs. bending or buckling curvature), Member ductility (transverse force vs. displacement / rotation), Storey ductility (storey shear vs. storey drift) and overall structural ductility (Base shear vs. roof displacement).

6.3.10 Response Spectrum

The concept of Response Spectrum has been introduced in section 3.11. The Response spectra is a plot of maximum response versus T (for fixed ξ) for a given ground motion. This is response in terms of force, displacement, acceleration, shear force etc. Design spectrum smoothens out the crest and trough in response spectra. This is specified concurrently with damping to be used, natural period calculation, permissible stress/ strain, load factors etc. (Fig 6.4)

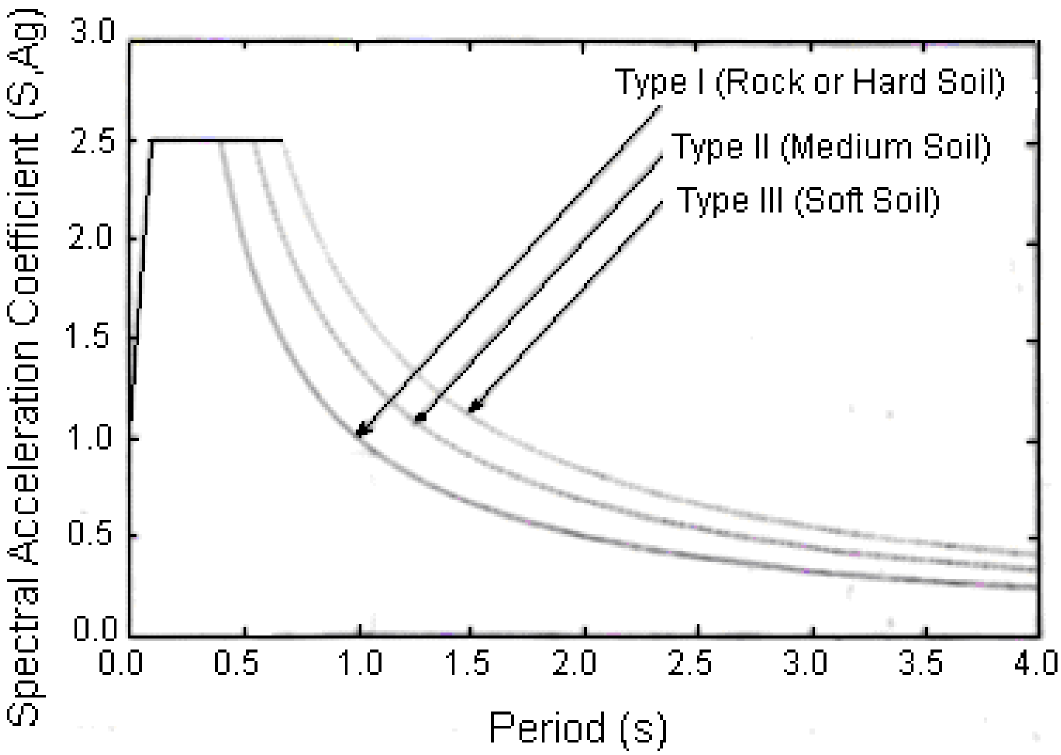

Figure 6.4: Response Spectra

6.4 DUCTILITY BASED DESIGN

Earthquake resistant design philosophy strongly supports that, member elements as well as the entire structure should be strong enough to carry the gravity load safely.

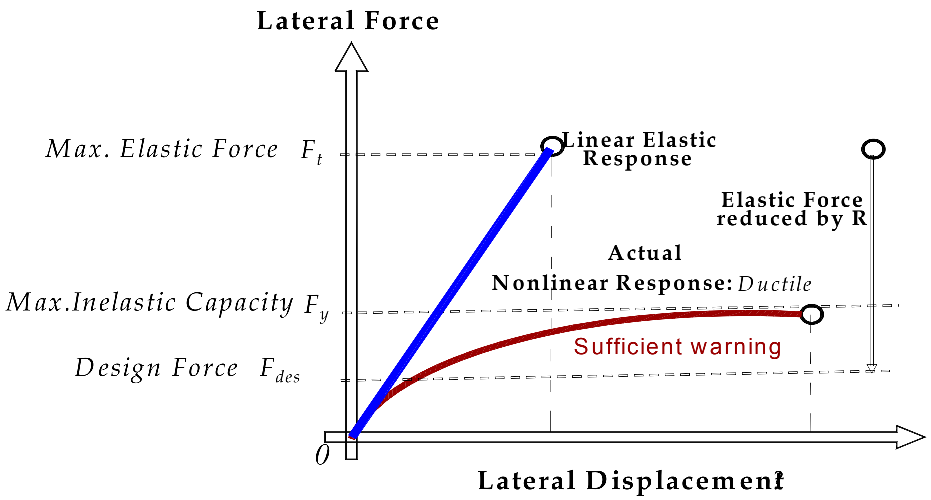

Figure 6.5: Strength and Ductility

But to resist lateral load, structures would rely on ductility, as it provides similar scope for energy absorption, yet at the same time it would fail after sending sufficient warning through deformation. No sudden collapse would occur to the structure. Reliance on ductility in comparison to strength of the structure builds basis of ductility-based design. As ductility has been relied on, ductility reduction factor and over strength factor would influence the design force through introduction of Response reduction factor, R. (Fig 6.5)

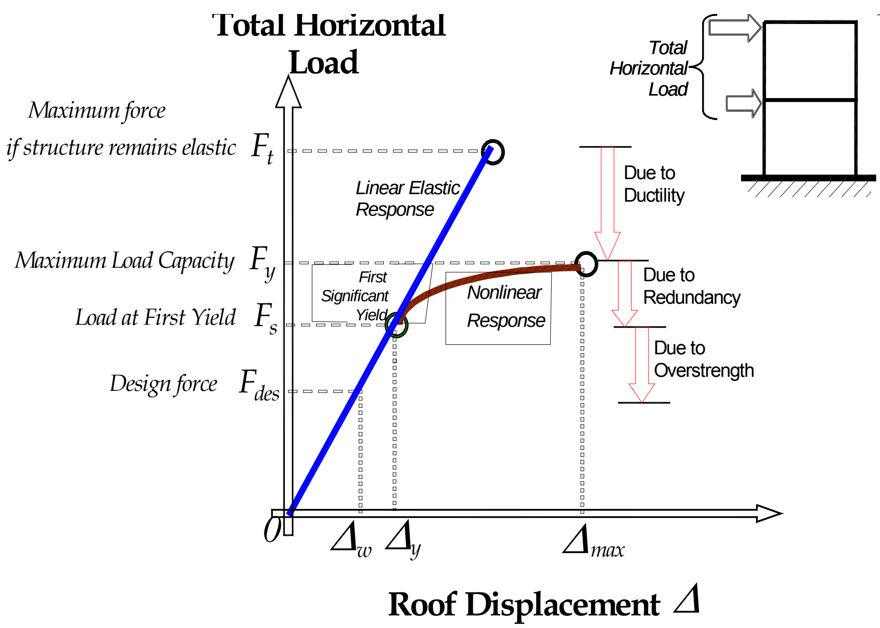

Figure 6.6: Ductility, Redundancy & Over-strength reduction

Ductility reduction factor, R_μ is the ratio of F_{max} (maximum force when the structure remains linearly elastic) and maximum inelastic force, F_y in an elasto-plastic system. If ductility can be ensured, a structure can be designed to yield at a reduced force of F_{max} / R. The R_μ is a function of ductility, μ and natural period of a structure, T. Stiffer structures with shorter T (T < ~ 0.4 s) would behave in a more ductile manner than any flexible with large T structures.

Over-strength factor, Ω is the ratio of Yield force or maximum inelastic force, F_y in an elasto-plastic system and Design Force, F_d. Following factors contribute to the over-strength factor, Ω: Load factor on seismic and gravity load, Material factors, Member sizes/ reinforcement more than required, Special ductile detailing, Redundancy, Strain hardening in materials, Higher material strength under cyclic loads, Strength contribution of non-structural members, etc.
Response Reduction Factor, R is calculated as Ductility Reduction Factor, R_μ times Over strength Factor, Ω.

i.e. $R = R_\mu \times \Omega$ and, Design Force, F_d = Maximum Elastic Force, F_{max} / R.

The IS 1893(Part 1)-2002 says, in case of ordinary RC shear wall, R is 6.0 and in Ductile RC shear wall R is 8.0.

6.5. THE CAPACITY DESIGN CONCEPT

For Earthquake Resistant Design maximum elastic seismic forces is calculated first (e.g. with help of response spectra method) and then reduce to account for ductility and over-strength. Structure should have adequate strength and good ductility.

In capacity design method planning for failure sequence is very important. The hierarchy of failure of elements in a structure is decided on basis of consequence of their failure. Local impact of failure would support the candidature of slab to be failed first. Beam would come next as it would influence two adjacent bays in that story. Then expected sequence would be Column, Foundation and lastly soil. Another assumption has to be made on type of damage. Preferable damage pattern is gradual and ductile one over sudden and brittle one; e.g. beam should fail in flexure yielding rather than local buckling. That emphasize ductile elements should yield prior to failure of brittle elements. Ductility of the structure shows the ductility of its weakest element. So, strong column with weaker beam would be an appropriate strategy for ductility based capacity design.

In a Capacity Design method, followings are the steps:

- Assess required strength of structure from seismic code
- Apply suitable safety factors on this load and material properties, and design and detail out ductile elements.
- Identify a desirable collapse mechanism
- Assess upper-bound strength (upper bound loads on the structure corresponding to yielding of ductile elements) of the ductile element
- Design brittle elements corresponding to upper-bound load calculated above; and thus ensure that brittle elements are elastic when the ductile elements yield.

To improve performance of the structure under earthquake load,

- Increase redundancy is helpful to make the predetermined elements to undergo large inelastic actions.
- Prevent premature local or member buckling of design elements.
- Ensure that predetermined locations of inelastic actions can sustain expected large plastic rotations through providing good moment connections.

6.6 CONCLUSIONS

Human settlements in highly seismic regions of the world by observations in past earthquakes developed an adequate understanding of seismic behavior of buildings and evolved sound seismic design principles. All the earthquake resistant constructions have to be based on these principles to minimize the damage to buildings during earthquakes.

Various crucial aspects for seismic design principles like strength, stiffness, period, center of mass, center of rigidity or stiffness, torsion, design eccentricity, damping, ductility and response spectrum are discussed in the chapter. It is seen that earthquake resistant philosophy strongly supports that entire structure should be strong enough to carry the gravity load safely, and at the same time ductility of the structure is important to take the lateral loads. Ductility of a structure shows the ductility of its weakest elements. Hence understanding ductility based capacity design concept is important for earthquake resistant design.

CHAPTER 7

EARTHQUAKE RESISTANT DESIGN AND DETAILING

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EARTHQUAKE RESISTANT DESIGN AND DETAILING

7.1 INTRODUCTION

We have seen that, relative movement of tectonic plates cause an earthquake, as a result of which considerable ground energy is released which causes the ground to shake over a long distance from the point of release of this energy.

When the ground on which a building is standing shakes, the superstructure is stationary but the foundation is subjected to the ground motion. This movement of the foundation induces movement in the superstructure as well, thereby subjecting the entire building to heavy vibrations, which induce heavy stresses and strains in all its members.

If the building has sufficient strength and resilience, it will move along with the ground and vibrate. If it is weak or brittle, it will be damaged or will collapse. The aim of earthquake engineering is to ensure, to the extent possible, that the necessary strength and resilience will be present in the completed building to enable it to withstand these induced stresses and strains.

Earthquake motion is chaotic, at times violent, and it involves translation and rotation of the ground in all directions, simultaneously. This pattern of ground motion is neither unique nor uniform. It varies from one earthquake to another. Its magnitude and intensity depends on the varying soil properties starting from the focus of the earthquake, which invariably is deep inside the earth, to the epicenter, which is on the crust, and then to the point where a building is located. It also depends on the properties of soil on which the building is constructed.

To the complexity of the ground motion is added the complexity of the response of the structure. This response depends on its shape, form, structural system of the superstructure, type of foundation, type of finishes, details of finishes, cladding materials, construction materials and, finally, the quality of construction. Added to this, is the problem of limitations on accurately modeling a structure mathematically to predict its response to such chaotic ground motions in elastic as well as in inelastic range.

Even though the knowledge to predict response in elastic range of a structure has advanced considerably, but that to predict its response in inelastic range is still meager. However, in case of a very severe earthquake, a structure would definitely move into the inelastic range because a number of elements and joints will yield. It is, therefore, very important to provide sufficient strength in members and joints so that even after yielding they do not completely fail and the structure does not collapse.

7.2 EARTHQUAKE CODES

Scientists have been able to identify the range of earthquakes of different magnitudes and different intensities, which could be, expected in different parts of the country, and also to some extent the probable frequency of occurrence of the highest range of earthquake in a specific region. This knowledge forms the basis of earthquake codes, which divide the country in different zones where earthquakes of a specific magnitude and a specific intensity could be expected.

The Codes also give guidelines for design of buildings with different materials, of different shapes and forms, and with different structural systems. Considering the unpredictable nature of earthquake, in its occurrence, its magnitude, its intensity and its duration, the Codes are very guarded in undertaking complete responsibility of the safety of a structure, even if it is designed following all its provisions rigorously. This is a stand taken by the codes all over the world, not only just in India.

The philosophy of design adopted by all these codes is, that if a building is designed properly and constructed properly on the basis of the code, it should not suffer any damage under a mild earthquake, should suffer damage of only non-structural elements and finishes etc, which can easily be repaired under an earthquake of medium to high intensity, and should suffer damage of structural elements, without collapse, under very severe earthquake.

None of the codes states that if a building is designed following provisions of the code, and built properly, nothing will happen to it under any earthquake that can possibly occur in that region. Because those who frame these codes realize that if they had to make this statement, the cost of the buildings would be prohibitive. The buildings would have to be designed for earthquakes of very high intensity and magnitude, the probability of occurrence of which may be only once in fifty years, or once in hundred years.

It is interesting to trace the history of the process of making these earthquake codes. Early codes were based directly on the practical lessons learned from earthquakes, relating primarily to types of construction. In some cases they placed limitations on the height of buildings. It was recognized that timber structures performed well (even the relatively tall Japanese pagodas), whereas plain masonry buildings performed poorly.

This process started in **Italy** way back in 1783 when a severe earthquake in “Calabrian” prompted the engineers to think of Earthquake (EQ) resistant buildings. On the basis of observations, the engineers stipulated that:

- All buildings which had failed & survived be built with timber frame, in-filled with stone embedded in mortar
- The maximum height of buildings be two storeys

However, these stipulations were not rigorously followed as the years, decades and a century went by, during which period there was extensive seismic activity in the region but of small to medium intensity and magnitude. Later in 1908 a severe earthquake occurred in “Messina-Reggio”, during which 160,000 persons lost their lives in a relatively small area. Most of the collapsed buildings were in masonry and had not followed the stipulations framed in 1784.

After this earthquake, a commission of nine practicing engineers and five distinguished university professors were assigned the task of identifying methods to design buildings which were cheap, could be built easily an could also resist earthquakes. The commission gave two proposals:

1. Isolate the building from the ground and place it on a compacted layer of sand or on spherical rollers.
2. Construct the building with timber frame and in-built rubble masonry but design it to withstand horizontal force equal to 1/12 of its dead weight. This force was later changed to a horizontal design load of 1/12 of the weight above for the ground floor and 1/8 of the weight above for 2nd & 3rd floor.

Proposal (2) was generally adopted. These were intuitive recommendations based on observations and these concepts of designing a building to withstand a stipulated horizontal force or isolating it from the ground are still valid.

Japan also has a long history of earthquakes, which had intuitively led them to construct very light buildings in timber. Some of the wooden pagodas constructed before 15th century are amongst the tallest wooden structures in the world and have withstood many earthquake without any reported damage. Here again, a commission was set up which observed that buildings built in wood and steel had fared much better than those in concrete. But those in masonry had fared very badly.

It is interesting to note that three buildings designed by one Dr. Tachu Naito, Professor of Architecture at Waseda University in Tokyo, withstood the 1923 earthquake remarkably well. These three buildings were the Japan industrial Bank, 100ft high in steel frame, Jitsugyo Building in reinforced concrete frame, and the Kalenki Theatre in a combination of concrete and steel. All these buildings had been designed to withstand a lateral force equal to 1/15 of their dead weight. By 1880 a Seismological Programme had been set up and some empirical criteria for design of earthquake had been developed.

In 1923 a very severe earthquake took as many as 1,40,000 lives in Japan. By then, a number of building in steel and reinforced concrete had also been built, most of which withstood the earthquake fairly well. On account of success of these buildings to resist earthquake forces, Dr. Naito was considered an authority on the subject. He had laid down four very simple principles to be followed for earthquake resistant buildings:

1. A building should be as rigid as possible with rigid joints and generous bracings or shear walls. This will ensure short building period and prevent resonance with ground motion.
2. Use a closed plan layout, rather than an open U, L, T or H shapes.
3. Rigid walls or bracings should be placed symmetrically in plan.
4. Lateral force allocation to different frames of a building is done based on their rigidities.

The famous Imperial Hotel designed by Frank Lloyd Wright survived this 1923 earthquake in Japan without too much damage. It was a fairly rigid two storied building supported on short 8' long piles at 2'x2' grid.

In the **USA,** a very severe earthquake occurred in 1906 in San Francisco. But it did not result in setting up any commission to make recommendations. Instead, the regular building code made a provision to design all buildings for a horizontal load of 30 pounds per square foot to cater for wind and earthquake forces.

It was only at 1925 "Santa-Barbara" earthquake that work to frame a seismic design code was undertaken which resulted in a "Uniform Building Code" published in 1927. The provisions in this code were, however, casual, to cater for a specified horizontal force, and were not mandatory.

A subsequent "Long-Beach" earthquake in 1933, made the authorities serious. Laws were enacted and codes were framed. Serious provisions for designing the buildings for specified horizontal loads were made, which were modified over the years as more and more knowledge of earthquake engineering was acquired, and which became more and more stringent.

1933 - Important buildings like schools, hospitals, places of assembly for 10% (Dead + some Live load) all other buildings 2% to 5% (Dead + some Live load)

1943 - Horizontal load each floor = C x dead load above C=0.6/(N+4.5), N= no of stories above. Thus for a one storey building (N=0) C=0.133 and a 13 storeys building (N=12) C=0.0364

1947 - Horizontal load varies form 3.7 to 8% of design vertical load depending on number of storeys and soil conditions

1948 - Base shear V=CW
W=dead load +0.25 live load
C=0.015/T
T= fundamental period = 0.05H
H= height in ft.
D= plan dimension in the direction of earthquake in ft.

1956 - V same as above, but
C=0.02/T

A number of subsequent revisions took place. The latest uniform building code is of 1997 edition. The name of this code has been changed to “International Building Code of USA” in 2000, the latest edition of which is of 2003.

The first **Indian** code for design of EQ resistant buildings was framed in 1962, which was subsequently revised in 1966, 1970, 1975, 1984 and finally in 2002. With every revision, revisions were modified on the basis of the latest available knowledge. However with every such revision provisions of the code became more stringent.

Originally, the country had been divided in 7 zones. Starting with EQ of very mild intensity in Zone-0, the intensity kept on increasing in Zones-I, II, III, IV and V with the heaviest in Zone-VI. Subsequently in 1984 these seven zones were reduced to five. Zone-0 was merged into Zone-I and Zone-VI in Zone-V. Recently in 2002 the zones have further been reduced to four. Zone-I has been merged into Zone-II. The latest seismic zoning map of India showing four different zones (zone II, III, IV and V) is shown in Fig 2.11.

The magnitude of seismic force experienced by a building varies with its configuration, construction materials, height, and number of floors, structural system, and type of foundation and soil characteristics. It is directly related to the intensity of earthquake. Actual forces that a structure is subjected to during an earthquake may be far greater than those specified in this Code. However, ductility, arising from inelastic material behavior and detailing and over strength arising from the additional reserve strength in a structure, over and above the design strength, are relied upon to withstand these additional forces.

The strength requirement of a building to withstand earthquake-generated forces can be assessed based on a “Static Approach” or a “Dynamic Approach”.

In the static approach, it is assumed that earthquake vibrations subject a building to horizontal forces along its height. The magnitude and distribution of such horizontal forces is related to the construction materials, height, and number of floors, foundation system, soil characteristics and the Earthquake zone in which the building is located. The total design lateral load is termed as the design seismic base shear.

In the Dynamic Approach, vibration analysis of the building is carried out to establish the base shear and its distribution over the height of the building. The Code specifies that all buildings can be designed with the static approach, except for buildings higher than 40m in Zones IV & V and higher than 90m in zones-II and III which need to be designed with the dynamic approach. But if a building is irregular it has to be designed according to dynamic approach, if it is higher than 12m in Zones-IV and V and higher than 40m in zones- II & III.

The Code has very stringent provisions to cater for torsion effects of earthquake forces. It also has equally stringent provisions to analyze and design irregular buildings.

7.3 UNIFORM PHILOSOPHY OF DESIGN

The philosophy of design against earthquake forces in all the codes is more or less the same which has been given earlier and which states that, the buildings designed according to codes should be able to resist minor earthquakes undamaged, resist moderate earthquakes without significant damage, and resist severe earthquakes without collapse.

The codes do not guarantee that a structure would never be subjected to higher forces than stipulated in any earthquake. All the Codes have similar equations to compute the base shear, and its distribution over the height of a building. This distribution is either triangular or parabolic, with the highest value at the top and least at the bottom. Some of the codes also specify a small component of the base shear to act as a point load at the top.

Provisions for variation of base shear related to importance of the building, structural system, structural material, soil properties etc are also similar. Considering regions of different intensity of earthquake, and buildings of different height, different flexibility, and on different soil conditions, the base shear could vary from 2% to 14% of the total gravity and part live load of a building. This range is also more or less similar.

It is interesting to compare this range with that specified intuitively by Italian and Japanese engineers long ago, which varied from 7% to 12%. All Codes have fairly stringent provisions for analyzing and designing irregular buildings. All Codes ascribe to the concept of inelastic response of the Structure and Ductility.

Structural Engineers are primarily responsible for the framing of Code Provisions for the buildings. But this being a collaborative process, architects need to study and contribute further to this so that it is easier for the engineers to follow those provisions in their true spirit without compromising much in the quality of architectural as well as structural design.

7.4 CHOICE OF STRUCTURAL MATERIALS

The preferred materials for medium to high-rise buildings in earthquake regions are concrete and steel. With the help of both these materials the desired level of flexibility or rigidity can be provided. Both these materials if detailed properly retain their integrity even after a large number of stress reversals, and have high level of ductility.

A precast system or a combination of prestressed and precast system is not considered very suitable but cannot be ruled out. In Russia it has been used quite successfully in regions of high seismic activity. The joints need special consideration and very careful detailing so as to be ductile and not lose integrity under cycles of reversal of stresses.
For low-rise buildings masonry or a combination of masonry with concrete frames is most popular.

7.5 CHOICE OF STRUCTURAL SYSTEMS

7.5.1 Guide Lines for Planning Earthquake Resistant Buildings

As we have seen in chapter 5 that poor form cannot be ordered to behave satisfactorily in an earthquake. Therefore in order to predict response of a form to earthquake, it should be sound and symmetrical and honor the following design principles.

i. Be simple
ii. Be symmetrical
iii. Not be too elongated in plan or too slender in elevation
iv. Have continuous and uniform distribution of strength
v. Have horizontal members which form hinges before the vertical members
vi. Have its stiffness related to sub soil properties

7.5.2 Stiff Structures versus Flexible Structures

There is a considerable difference of opinion between different specialists of earthquake engineering regarding the final choice of a flexible or a stiff structure for a specific building.
Before discussing pros and cons of the two types of structure, it is important to understand what a Flexible Structure is and what is a Stiff Structure:

a. A framed structure in concrete or steel without any shear walls, and with partition walls isolated from the frames is a very flexible structure
b. If in the structure described in (a), the partition walls are not isolated from the structure, it is less flexible than (a).

c. If in the structure described in (b) shear walls or cross bracings are also added, it is a rigid structure

d. All structures supported on masonry, or reinforced masonry walls are rigid structures.

One very important property, which determines the base shear attracted by a structure, is its natural period of vibration. If the period of vibration of the ground becomes equal to the natural period of vibration of the building, there is resonance, which can be disastrous for the building. (See section 3.5, 6.3.2 and 6.3.3)

The natural period of a flexible building is large and that of a stiff building is short. A flexible building, has a large drift, has a capacity to absorb earthquake induced energy on account of this drift and attracts a smaller base shear, and has a higher ductility.

A stiff building on the other hand has much less drift, but attracts higher base shear, absorbs earthquake energy through moments and shears and has lower level of ductility. The system, which is gaining far greater acceptance, is, a controlled combination of the two – a framed structure with in-filled partition walls and symmetrically placed shear walls.

7.5.3 Horizontal and Vertical Members

One of the essential design principles of earthquake engineering is that horizontal members should fail before vertical members. This increases the capacity of a building to keep absorbing earthquake motions even after hinges are formed in the beams, but not in the columns. Collapse would occur only after hinges are formed in the Columns.

7.5.4 Uniform & Continuous Distribution of Strength

The behavior of a structure is far closer to its analysis, and it has far greater chances of withstanding earthquake-induced forces properly without much damage if it follows the following design principles:

a. The load bearing members are uniformly distributed in plan.
b. All columns and walls are continuous and without offsets from roof to foundation.
c. All beams are free of offsets.
d. Columns and beams are co-axial.
e. Columns and beams are of nearly the same width.
f. A principal member should not change section suddenly.
g. The structure is continuous and monolithic with rigid joints

The more these principles are followed, the less would be the cost of the structure, the detailing would be easier to plan & construct, and its behavior under an earthquake would be much better.

7.6 IS 1893:2002 CRITERIA FOR EARTHQUAKE RESISTANT DESIGN

7.6.1 Inelastic Seismic Response of Structures and Ductility

Under sustained loading a member suffers elastic deformations, which keep on increasing linearly with the load, upto the yield stress of the material. After this is a stage called “Plastic Stage” in which the member keeps deforming without any additional load, which is followed by a “Strain hardening” stage in which the member capacity slightly increases, and finally the “strain softening” stage in which the member collapses (see section 6.3.9).

7.6.2 Load Combinations

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

1. 1.5(DL+IL)
2. 1.2(DL+IL±EL)
3. 1.5(DL±EL)
4. 0.9DL±1.5EL

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

When the lateral load resisting elements are not oriented along the orthogonal horizontal directions, the structures shall be designed for the effects due to full design earthquake load in one horizontal direction plus 30% of the design earthquake load in the other direction.
For instance, the building should be designed for (±ELx ± 0.3ELy) as well as (±0.3ELx ± Ely), where x and y are two orthogonal horizontal directions.

7.6.3 Estimation of Base Shear

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

$$V_b = A_h W$$

Where, A_h = Design horizontal acceleration spectrum value as discussed in Chapter6, using natural Period Ta in the considered direction of vibration

W = Seismic weight of the building

7.6.4 Distribution of Shear in Multi-Storied Building

Vertical distributions of Base shear to different floor levels of a multistoried building have been referred in IS 1893 (Part I): 2002. The design Base Shear (V_b) computed in 7.6.4 shall be distributed along the height of the building as per the following expression:

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

Where, Q_i = Design lateral force at floor,i.

W_i = Seismic weight of the building

h_i = Height of floor I measured from base, and

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

7.6.5 Estimation of Earthquake Loading

For the purpose of determining seismic forces, the country is classified into four seismic zones as in Fig. 9. **Design Horizontal Seismic Coefficient** can be calculated as :

$$A_h = \frac{ZI}{2R}\left(\frac{Sa}{g}\right)$$

Z = Zone factor
I = Importance factor
R = Response reduction factor I/R shall not be greater than one.
Sa/g = Average response acceleration coefficient from response spectra as shown in Fig 6.4 for the fundamental period of the structure.

The buildings shall be designed for lateral force as calculated below:

$$Q_i = V_B \frac{W_i h_i^2}{\Sigma^n W_j h_j^2}$$

V_B = $A_h W$
V_B = Base shear
W = Seismic weight = Dead load + part of super /imposed load
This design base shear is distributed along the height of the building as given below:
Q_i = Design lateral force at floor i
W_i= Seismic weight of floor i
h_i = height of floor I;
n = number of stories

The total shear in any horizontal plane shall be distributed to the various vertical element of lateral force resisting system (shear walls, bracing).

7.7 TYPES OF CONSTRUCTION

7.7.1 Framed Construction

This type of construction is suitable for multistoried and industrial buildings. Vertical Load Carrying Frame Construction consists of frames with flexible (hinged) joints and bracing members. Such buildings shall be adequately strengthened against lateral forces by shear walls and / or other bracing systems in plan, elevation and sections such that EQ forces shall be resisted by them in any direction.

7.7.2 Moment Resistant Frames With Shear Walls

The frames may be of reinforced concrete or steel with semi-rigid or rigid joints. The walls are rigid capable of acting as shear walls and may be of reinforced concrete or of brickwork reinforced or unreinforced bounded by framing members through shear connectors.
The shear walls should extend from the foundation either to the top of the building or to a lesser height as required from design consideration. In design, the interaction between frame and the shear walls should be considered properly to satisfy compatibility and equilibrium conditions.

7.7.3 Box Type Construction

This type of construction consists of prefabricated or in-situ masonry, concrete or reinforced concrete wall along both the axes of the building. The walls support vertical loads and also act as shear walls for horizontal loads acting in any direction.
All traditional masonry construction falls under this category. In prefabricated construction attention shall be paid to the connections between wall panels so that transfer of shear between them is ensured.

7.8 CATEGORIES OF BUILDINGS

For the purpose of specifying the earthquake resisting features in masonry, the buildings have been categorized in five categories A to E (Table 7.1) based on the value of A_h

$$A_h = a_o \, I \, b$$

A_h = Design seismic coefficient for the building
a_o = Basic seismic coefficient for the seismic zone in which the building is located.
I = Importance factor applicable to the building
b = Soil foundation factor

Table 7.1: Building category for Earthquake Resisting Features

Building Categories	Range of α_h
A	0.04 to less than 0.05
B	0.05 to 0.06 (both inclusive)
C	More than 0.06 and less than 0.08
D	0.08 to less than 0.12
E	Equal to or more than 0.12

7.9 DUCTILE DETAILING OF R C STRUCTURES

7.9.1 Ductility

The main structural elements and their connection shall be designed to have a ductile failure. This will enable the structure to absorb energy during earthquakes to avoid sudden collapse of the structure. Providing reinforcing steel in masonry at critical sections, as provided in this standard will not only increase strength and stability but also ductility.

Ductile detailing is required to give them adequate toughness and ductility to resist severe earthquake shocks without collapse. Ductility is the ability of a structure to undergo deformations after its initial yield, without any significant reduction in ultimate strength.

Types of Ductility:

- Material ductility;
- Cross-section ductility ;
- Member ductility;
- Structure ductility and
- Energy ductility.

IS 13920 suggests ductile detailing of reinforced concrete structures subjected to seismic forces.

7.9.2 Flexure Members

Standards provided by IS Codes for design of flexure members are:

- The member should preferably have a width-to-depth ratio of more than 0.3.
- The width of member should not be less than 200 mm.
- The depth of member should preferably be not more than ¼ of the clear span.

Longitudinal Reinforcement

- At least two bars throughout the member length at top and bottom.
- Positive steel at a joint face must be at least equal to 1/2 the negative steel at that face.
- Steel provided at each of the top and bottom face of the member at any section along its length should be at least equal to 1/4 of the maximum negative moment steel provided at the face of either joint.

The longitudinal bars should be spliced, only if hoops are provided over the entire splice length, at spacing not exceeding 150 mm.
The lap length should not be less than the bar development length in tension. Lap splices should not be provided:

- Within a joint,

- Within a distance of 2d from joint face
- Within a quarter length of the member where flexural yielding may generally occur under the effect of earthquake forces

Web Reinforcement

The spacing of hoops over a length of 2d at either end of a beam should not exceed

- d/4
- 8 times the dia of the smallest longitudinal bar

However, it need not be less than 100 mm (Fig 7.1). The first hoop should be at a distance not exceeding 50 mm from the joint face. Vertical hoops at the same spacing as above should 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 should have vertical hoops at a spacing not exceeding d/2.

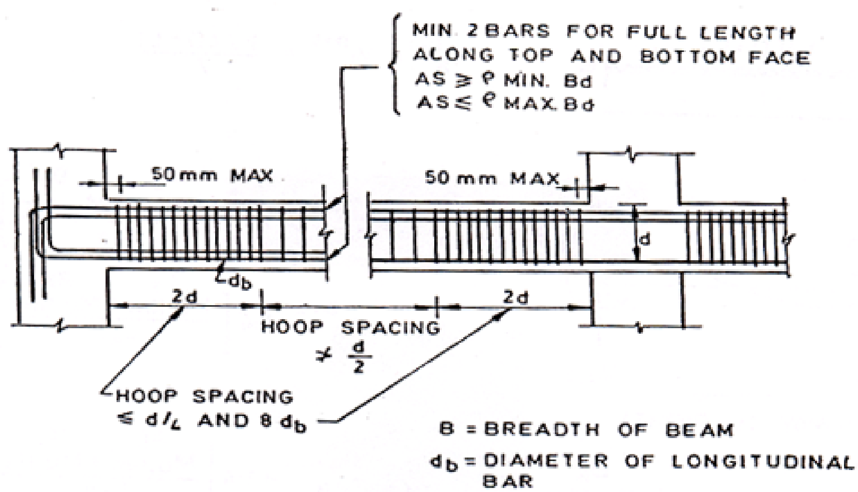


Figure 7.1: Beam Reinforcement

7.9.3 Columns and Frame Members Subjected To Bending and Axial Load

The specifications for the columns and frame members subjected to bending and axial loads are:

- The minimum dimension of the member shall not be less then 200 mm.
- Frames, which have beams with center-to-center span exceeding 5m or columns of, unsupported length exceeding 4 m; the shortest dimension of the column shall not be less than 300 mm.
- Ratio of the shortest cross sectional dimension to perpendicular dimension shall preferably not be less than 0.4

a) Transverse Reinforcement

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 (Fig 7.2).
Alternatively, a pair of overlapping hoops may be provided within the column. The hooks shall engage peripheral longitudinal bars.

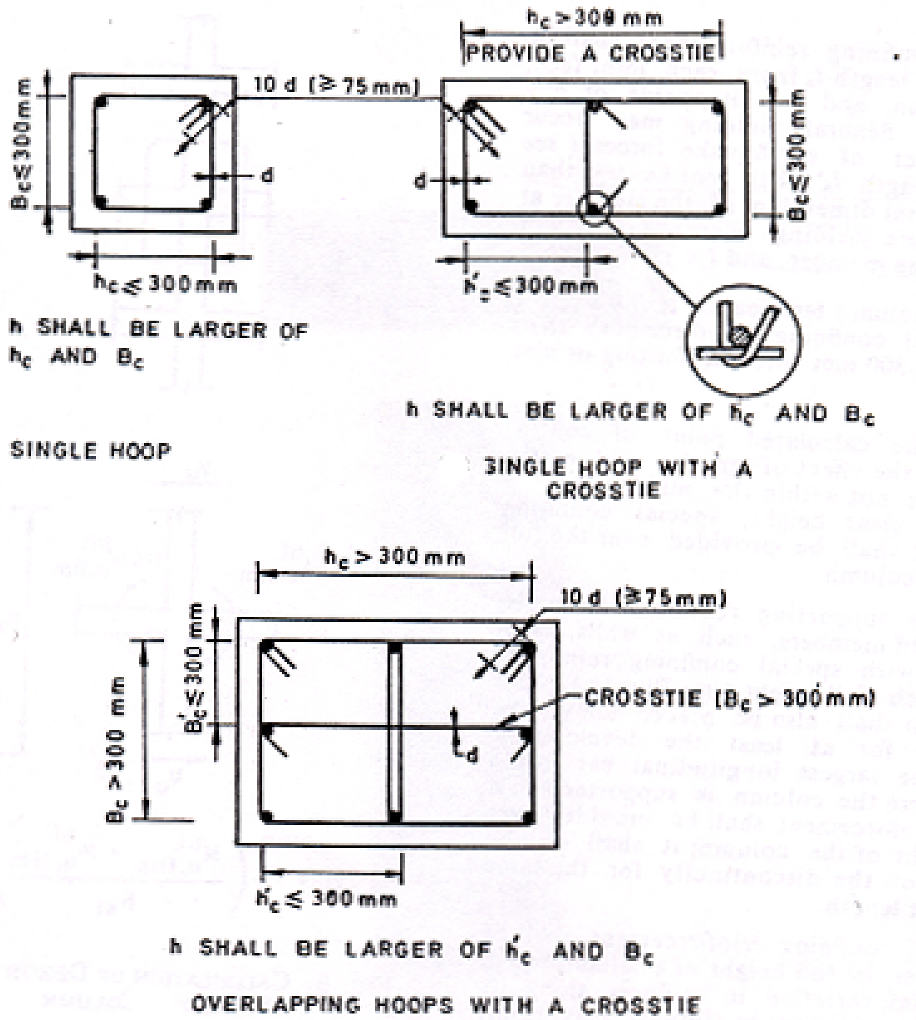


Figure 7.2: Transverse Reinforcement in Column

b) Special Confining Reinforcement

Special confining reinforcement shall be provided over a Length 'l_o' from each joint face, towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces (Fig 7.3). The length 'l_o' shall not be less than:

- larger lateral dimension of the member at the section where yielding occurs,
- 1/6 of clear span of the member, and
- 450 mm.

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.

Also, special confining reinforcement shall be provided over the full height of a column, which has significant variation in stiffness along its height. This may result in variation in stiffness.

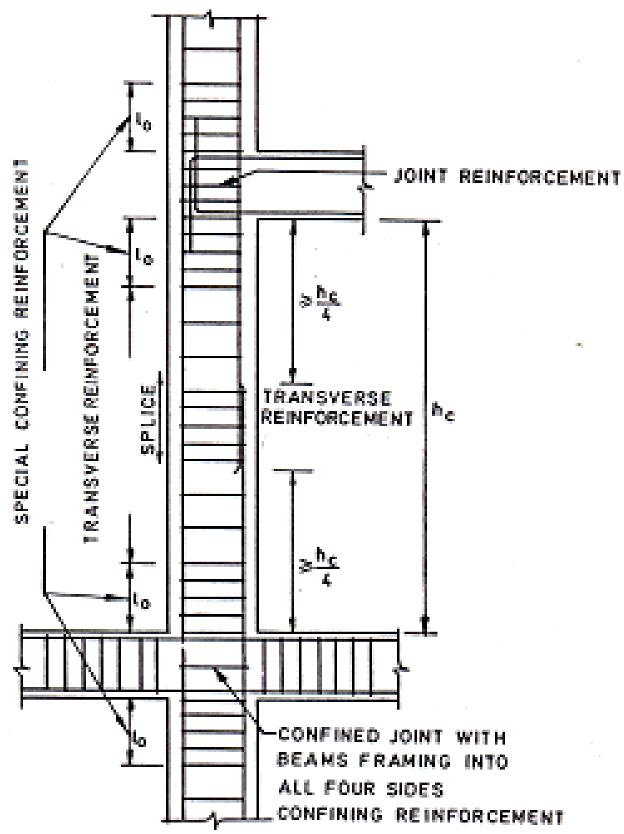


Figure 7.3: Column and Joint Detailing

7.9.4 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 (Fig 7.4).

A joint which has beams framing into all vertical faces of it and where each beam width is at least 3/4th 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.

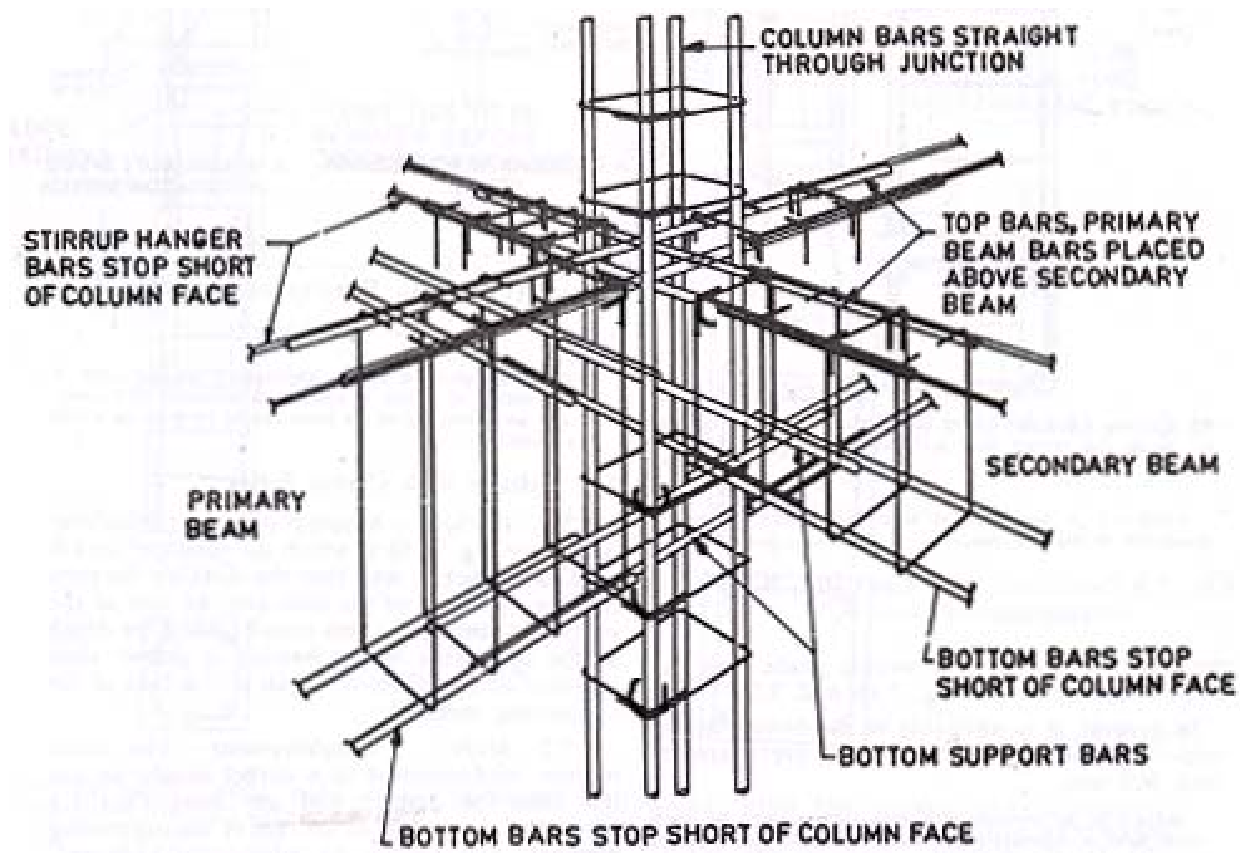


Figure 7.4: Beam Column Intersection

7.9.5 Shear Walls

a) Boundary Elements

Boundary elements are portions along the wall edges that are strengthened by longitudinal and transverse reinforcement. Though they may have the same thickness as that of the wall web it is advantageous to provide them with greater thickness.

b) Coupled shear wall

Ductile coupling beams shall connect coupled shear walls. If the earthquake induced shear stress in the coupling beam exceeds, $\frac{0.1 * ls * \mathrm{sqrt}(fck)}{D}$

Where, *ls* = clear span of the coupling beam

D = Over all depth the entire EQ induced shear and flexure shall, preferably, be resisted by diagonal reinforcement.

c) Openings in Shear Walls

The shear strength of a wall with openings should be checked along critical planes that pass through openings. Reinforcement shall be provided along the edges of openings in walls. The area of the vertical and horizontal bars should be equal to that of the respective interrupted bars. The vertical bars should extend for the full storey height. The horizontal bars should be provided with development length in tension beyond the sides of the opening.

d) Discontinuous Walls

Columns supporting discontinuous walls shall be provided with special confirming reinforcement.

e) Location of Shear Walls

Location of shear walls in plane and sections are very important for torsion phenomenon in the whole structure. (see section 4.3.4 and 6.3.6)

7.10 EARTHQUAKE RESISTANT MEASURES IN MASONRY STRUCTURES

7.10.1 Masonry Unit

Well-burnt bricks and solid concrete blocks having a crushing strength not less than 35 Mpa are masonry units. However, higher strength of masonry units may be required depending upon number of storeys and thickness of walls. Squared stone masonry, stone block masonry or hollow concrete block masonry of adequate strength, may be used.

7.10.2 Mortar

Where steel reinforcing bars are provided in masonry the bars shall be embedded with adequate cover in cement sand mortar not leaner than 1: 3 (minimum clear cover 10 mm) or in cement concrete of grade M15 (minimum clear cover 15 mm or bar diameter whichever more), so as to achieve good bond and corrosion resistance. In Table 7.2 proportion of mortar mixes have been proposed.

Table 7.2: Recommended Mortar Mixes

Category of Construction	Proportion of Cement-Lime-Sand
A	M2 (Cement-sand 1:6; or M3 (Lime-cinder: 1:3) or richer
B	M2 (Cement-lime-sand 1:2:9 or Cement-Sand 1:6) or richer
D, E	H2 (Cement-sand 1:4) or M1 (Cement-lime Sand 1:1:6) or richer

Note : Though the equivalent mortar with lime will have less strength at 28 days, their strength after one year will be comparable to that cement mortar.

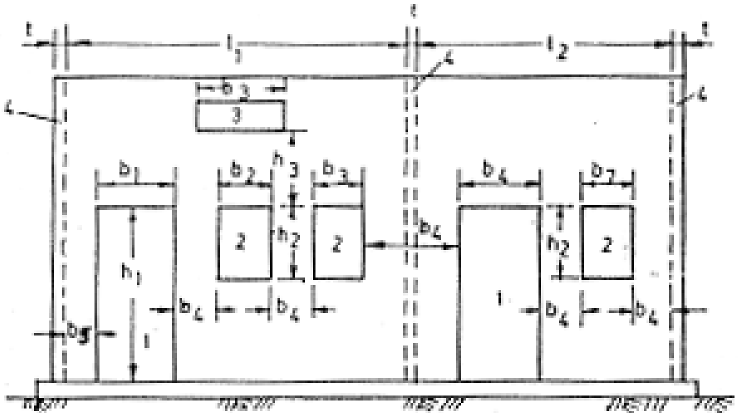
7.10.3 Masonry Bond

For achieving full strength of masonry, the usual bonds specified for masonry should be followed so that the vertical joints are broken properly from course to course.

To obtain full bond between perpendicular walls, it is necessary to make a slopping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise, the toothed joint should be made in both the walls alternatively in lifts of about 450 mm.

7.10.4 Opening in Bearing Walls

Doors and window openings in walls reduce their lateral load resistance and hence, should preferably be small and more centrally located (Table 7.3, Fig 7.5). Openings in any storey shall preferably have their top at the same level so that a continuous band could be provided over them, including the lintels throughout the building.



1. Door 2. Ventilator 3. Window 4.Cross wall

Figure 7.5: Dimensions of Opening and Piers

Table 7.3: Size and Position of Opening in Bearing Wall

Sl._No	Position of Opening	Details of Opening for Building Category		
1	Distance b_3 from the inside corner of outsider wall, Min	A and B Zero mm	C 230 mm	D and B 450 mm
2	For total length of openings, the ratio (b_1 + b_2 + b_3) l_1 or (b_4 + b_7) l_2 shall not exceed : a) one-store dyed building b) two-storied building c) 3 or 4-stroyed 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	340 mm	450 mm	560 mm
4	Vertical distance between two openings one above the other	600 mm	600 mm	600 mm

Where openings do not comply with the guidelines of Table 7.3, they should be strengthened by providing reinforced concrete or reinforcing the brickwork, with high strength deformed bars of 8 mm diameter but the quantity of steel shall be increased at the jambs, if so required.

If a window or ventilator is to be projected out, the projection shall be in reinforced masonry or concrete and well anchored. If an opening is tall from bottom to almost top of a storey, thus dividing the wall into two portions, these portions shall be reinforced with horizontal reinforcement of 6 mm diameter bars at not more than 450 mm intervals, one on inner and one on outer face, properly tied to vertical steel at jambs, corners or junction of walls, where used. The use of arches to span over the openings is a source of weakness and shall be avoided. Otherwise, steel ties should be provided.

7.10.5 Seismic Strengthening Measures

Horizontal Bands

All masonry buildings shall be strengthened by the methods, as specified for various categories of buildings, (Table 7.4, Fig 7.6 and Fig 7.7) schematically, the overall strengthening arrangements to be adopted for category D and E buildings which consist of horizontal bands of reinforcement at critical levels, vertical reinforcing bars at corners, junctions of walls and jambs of opening.

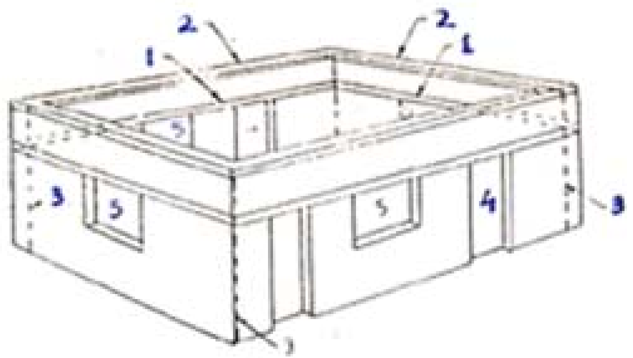


Figure 7.6: Overall Arrangement of Reinforced Masonry Buildings

Lintel band if provided in panel or partition walls also will improve their stability during severe earthquake. Gable band is a band provided at the top of gable masonry below the purlins. This band shall be made continuous with the roof band at the eaves level.

Table 7.4: Strengthening Arrangements Recommended for Masonry Buildings (Rectangular Masonry Units)

Building Category	Number of stories	Strengthening to be provided in all storeys
(1)	(2)	(3)
A	i) 1 to 3 ii) 4	a a, b, c
B	i) 1 to 3 ii) 4	a, b, c, f, g a, b, c, d, f, g
C	i) 1 and 2 ii) 3 and 4	a, b, c, f, g a to g
D	i) 1 and 2 ii) 3 and 4	a to g a to h
E	1 to 3	a to h

Where:
a – Masonry mortar
b - Lintel band
c - Roof band and gable band where necessary
d - Vertical steel at corners and junctions of walls
e - Vertical steel at jambs of openings
f – Bracing in plan at tie level of roofs
g – Plinth band where necessary
h – Dowel bars

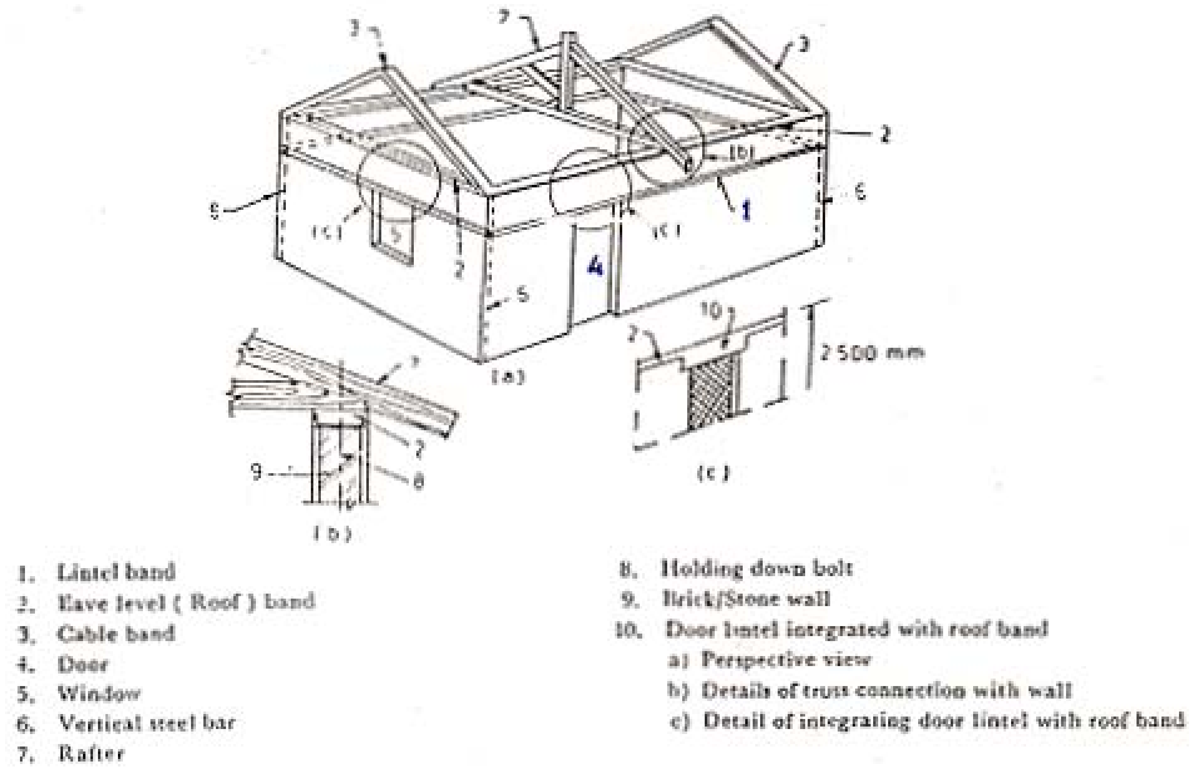


Figure 7.7: Overall Arrangement of Reinforced Masonry Buildings having pitched roof

Table 7.5: Recommended Longitudinal steel in Reinforced Concrete beams

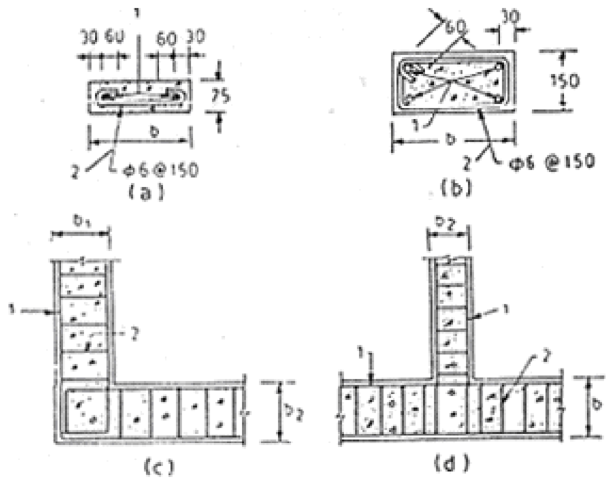
Span (m)	Building Category B		Building Category C		Building Category D		Building Category E	
	No. of Bars	Dia (mm)	No. of Bars	Dia (mm)	No. of Bars	Dia (mm)	No. of Bars	Dia (mm)
5 or less	2	8	2	8	2	8	2	10
6	2	8	2	8	2	10	2	12
7	2	8	2	10	2	12	4	10
8	2	10	2	12	4	10	4	12

Notes:

1. Span of wall will be the distance between center lines of its cross walls or buttresses. For spans greater than 8 m, 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 6mm dia spaced at 150 mm apart.

Section and Reinforcement of Band

The band shall be made of reinforced concrete of grade not leaner than M15 or reinforced brick work in cement mortar not leaner than 1:3.
The band shall be of the full width of the wall, not less than 75 mm in depth and reinforced with steel (Table 7.5).
In case of reinforced brickwork, the thickness of joints containing steel bars shall be increased so as to have a minimum mortar cover of 10 mm around the bar.
For full integrity of walls at corners and junctions of wall and effective horizontal bending resistance of bands continuity of reinforcement is essential. (Fig 7.8)



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) Section plan at T-junction of walls

Figure 7.8: Reinforcement and Bending Detail in R.C. Band

Table 7.6: 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 storyed building not permitted
	Third	10	10	12	
	Second	10	12	16	
	Bottom	12	12	20	

Notes:
1. The diameters given above are for HSD bars. For mild steel plain bars, use equivalent diameters.
2. The vertical bars will be covered with concrete M15 or mortar 1:3 grade in suitably created pockets around the bars. This will ensure their safety from corrosion and good bond with masonry.
3. In case of floors / roofs with small precast components.

Vertical Reinforcement

Vertical steel at corners and junctions of walls, which are up to 340 mm (brick) thick, should be provided as specified in Table 7.6. For walls thicker than 340 mm the area of the bars should be proportionately increased.

The vertical reinforcement should be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond. It should be passing through the lintel bands and floor slabs or floor level bands in all storeys.

Vertical reinforcement at jambs of window and door openings should be provided as per Table 7.6. It may start from foundation of floor and terminate in lintel band.

7.11 STRENGTHENING OF HOLLOW BLOCK MASONRY STRUCTURES

7.11.1 Horizontal Band

U-shaped blocks may be used for construction of horizontal band, at various levels of the storeys (Fig 7.9) where the amount of horizontal reinforcement shall be taken 25% more than that given for reinforced concrete bands and provided by using four bars and 6 mm diameter stirrups.

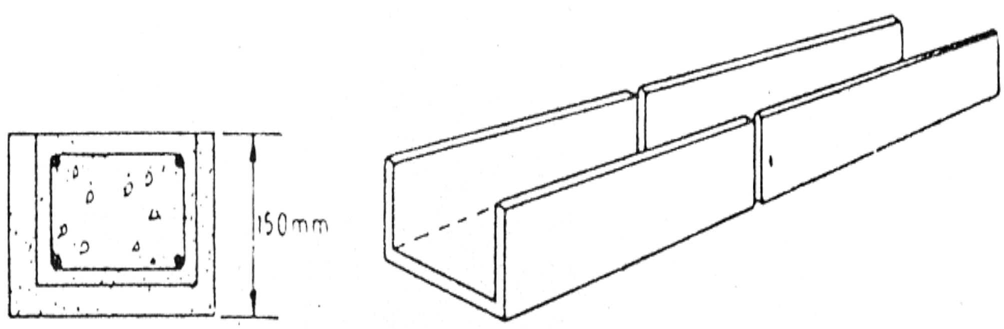


Figure 7.9: U-Blocks for Horizontal Bands

7.11.2 Vertical Reinforcement

Bars should be located inside the cavities of the hollow blocks, one bar in each cavity.

Where more than one bar is planned these can be located in two or three consecutive cavities. The cavities containing bars are to be filled by using micro-concrete 1: 2: 3 or cement-coarse sand mortar 1: 3 and properly rammed for compaction.

The vertical bars should be spliced by welding or overlapping for developing full tensile strength. For proper bonding, the overlapped bars should be tied together by winding the binding wire over the lapped length. To reduce the number of overlaps, the blocks may be made U-shaped.

7.12 IMPROVING EARTHQUAKE RESISTANCE OF LOW STRENGTH MASONRY BUILDINGS (IS 13828-1993)

7.12.1 Low Strength Masonry Construction

Brick construction using weak mortar, random rubble and half-dressed stone masonry construction using different mortars such as clay mud lime-sand and cement sand are low strength masonry. These constructions should not be permitted for important buildings with I>1.5 and should preferably be avoided for building category D (Table7.1) (see section 7.8)

Precautions should be taken to keep the rain water away from soaking into the wall so that the mortar is not softened due to wetness. An effective way is to take out roof projections beyond the walls by about 500 mm.

Use of a waterproof plaster on outside face of walls will enhance the life of the building and maintain its strength at the time of earthquake as well. Low strength masonry should not be used in Building category E as mentioned in Table 7.1.

Ignoring tensile strength, free standing walls should be checked against overturning under the action of design seismic co-efficient allowing for a factor of safety of 1.5.

7.12.2 Brickwork in Weak Mortars

The fired bricks should have a compressive strength not less than 3.5 MPa. Strength of bricks and wall thickness should be selected for the total building height. The mortar should be lime-sand (1: 3) or clay mud of good quality. Brick size depending on wall thickness should preferably be built using 1: 6 cement-sand mortar.

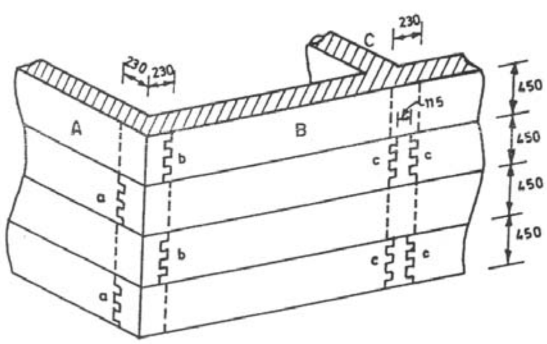
The minimum wall thickness shall be one brick in one storey construction and one brick in top storey and 1 ½ brick in bottom storeys of up to 3-storey construction. It should also not be less than l/16 of the length of wall between two consecutive perpendicular walls.

The height of the building shall be restricted to the following, where each storey height shall not exceed 3m.

For Categories A, B and C - three storeys with flat roof; and two storeys plus attic pitched roof.

For Category D - two storeys with flat roof; and one storey plus attic for pitched roof.

For achieving full bond between perpendicular walls, it is necessary to make a sloping (stepped) joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise the toothed joint should be made in both the walls, alternately in lifts of about 450 mm (Fig 7.10).



a, b, c = दीवार A, B और C में दंतुर जोड़

a, b, c = Toothed joints in wall A, B and C

सभी आयाम मिलिमीटर में

All dimensions in millimetres.

Figure 7.10: Alternating Toothed Joints in Walls at Corner and T-Junction

7.12.3 Stone Masonry (Random Rubble of Half- Dressed)

- The mortar should be cement-sand (1:6), lime-sand (1:3) or clay mud of good quality.
- The wall thickness should not be larger than 450 mm.
- Preferably it should be about 350 mm, and the stones on the inner and outer wythes should be interlocked with each other.

- The masonry should preferably be brought to courses at not more than 600 mm lift.

If walls longer than 5 m are needed, buttresses may be used at intermediate points not farther apart than 4.0 m. The size of the buttress be kept of uniform thickness. Top width should be equal to the thickness of main wall, t, and the base width equal to one sixth of wall height.

7.12.4 Opening in Bearing Walls

- Door and window openings in walls reduce their lateral load resistance and hence, should preferably be small and more centrally located. (Table 7.7)
- Openings in any storey shall preferably have their top at the same level so that a continuous band could be provided over them, including the lintels throughout the building.
- Where openings do not comply with the guidelines of Table 7.7, providing reinforced concrete lining with two numbers of high strength deformed bars of 8 mm diameter should strengthen them.

Table 7.7: Size and Position of Opening in bearing walls

	Description	Building Category	
		A, B & C	D
1	Distance b_3, from the inside corner of outside walls, Min	230 mm	600 mm
2	Total length of openings ratio: Max $(b_1 + b_2 + b_3) / l_1$ or $(b_6 + b_7) / l_2$ - one storeyed building - 2 & 3 storeyed building	 0.46 0.37	 0.42 0.33
3	Pier width between consecutive opening b_4	450 mm	600 mm
4	Vertical distance between two openings one above the other h_1, Min	600 mm	600 mm
5	Width of opening of ventilator , b_8, Max	750 mm	750 mm

7.12.5 Seismic Strengthening Arrangements

a) Reinforced band

The band should be made of reinforced concrete of grade no leaner than M15 or reinforced brickwork in cement mortar not leaner than 1:3. The bands should be of the full width of the wall, not less than 75 mm in depth and should be reinforced with 2 HYSD bars 8 mm dia and held in position by 6 mm dia bar links, installed at 150 mm apart (Fig 7.11).

b) Wooden band

As an alternative to reinforced band, the lintel band could be provided using wood beams in one or two parallel pieces with cross elements. Plinth band is a band provided at plinth level of walls on top of the foundation wall, This is to be provided where strip footings of masonry (other than reinforced concrete or reinforced masonry) are used and the soil is either soft or uneven in its properties as frequently happens in hill tracts.

c) Vertical Reinforcement

Vertical steel at corners and junctions of walls storeys may be welded or suitably lapped, which are up to 350 mm thick should be provided as specified in (Table 7.8). For walls thicker than 350 mm, the area of the bars should be proportionately increased.

The vertical reinforcement should be properly embedded in the plinth masonry of foundations and roof slab or roof band so as to develop its tensile strength in bond. It should pass through the lintel bands and floor slabs or floor level bands in all storeys. Bars in different storeys may be welded or suitably lapped.

All dimensions are in mm

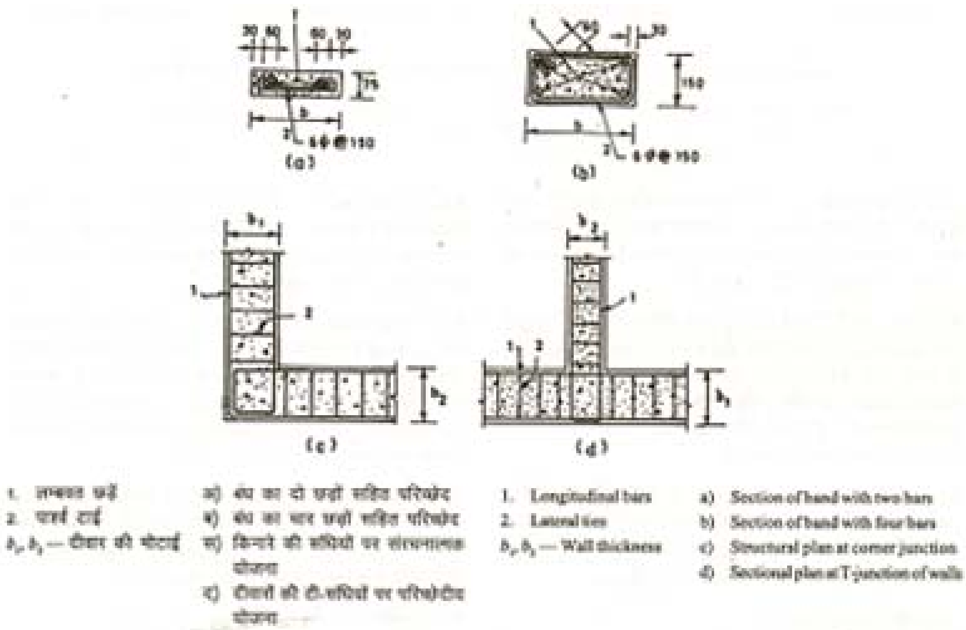


Figure 7.11: Reinforcement and Bending Detail in Reinforced Concrete Band

Table 7.8: Vertical Steel Reinforcement in Low Strength Masonry Walls

No. of storeys	Storey	Diameter of HSD Single Bar; in mm, at each critical section for			
		Category A	Category B	Category C	Category D
One	-	Nil	Nil	Nil	10
Two	Top	Nil	Nil	10	10
	Bottom	Nil	Nil	10	12
Three	Top	Nil	10	10	10
	Middle	Nil	10	10	12
	Bottom	Nil	12	12	12

Notes:

1. The diameters given above are for HSD (High Strength Deformed) bars with yield strength 415 MPa. For mild steel plain bars, use equivalent diameters.
2. The vertical bars should be covered with concrete of M15 grade or with mortar 1:3 (cement-sand) in suitably created pockets around the bars. This will ensure their safety from corrosion and good bond with masonry.

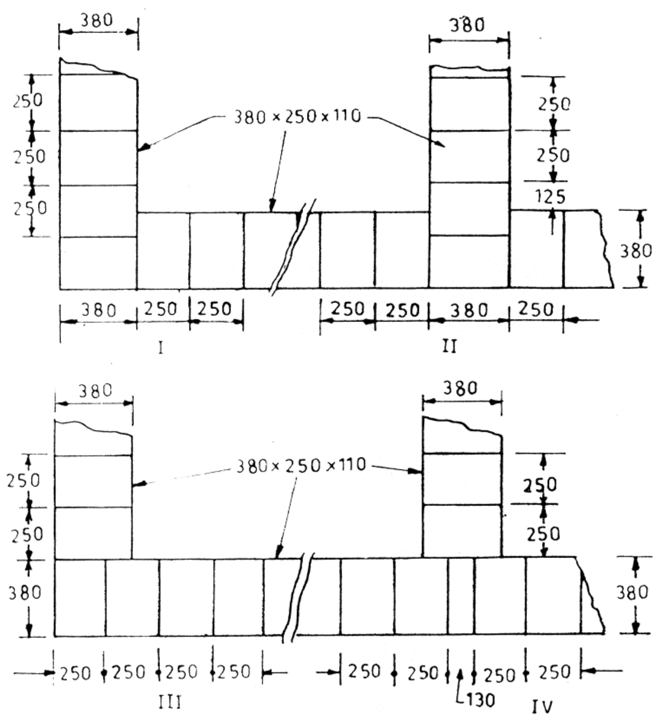
7.13 EARTHQUAKE RESISTANT DESIGN AND CONSTRUCTION OF ADOBE STRUCTURE (IS 13827-1993)

7.13.1 Box or Adobe Construction

Suitable soil should be used for making the blocks, by using uniform size of moulds, after keeping the soil-water mix for 24 hours. The blocks should be allowed to dry out of the moulds so as to allow 'free' shrinkage without developing fissures. The square type will be better for stronger construction in view of less vertical joints between units and better breaking of vertical joints.

The mud 'mortar' used to join the blocks together should be the same soil as used in making blocks. However, to make it non shrinking, straw in the ratio (1: 1, by volume, should be mixed. The wet mix should be allowed to rest for 7 days (minimum 3 days) before use.
The usual good principles of bonds in masonry to be adopted for construction of adobe walls, are:

- All courses should be laid level
- The vertical joints' should be broken between the consecutive courses by overlap of adobes and should be fully filled with mortar (Fig 7.12) and
- The perpendicular joints between walls should be made in such a way that through vertical joint is avoided (Fig 7.12).



(I), (II) First layer (III), (IV) Second layer
All dimensions in millimetres.

Figure 7.12: Typical Bond detail in Adobe Wall

7.13.2 Rammed Earth Construction

Rammed earth construction is also known as 'Pise' or 'Tapial' construction in some countries.
The soil suitable for rammed earth construction will generally have less clay than that used for making adobes. The moisture content should be kept less but close to optimum moisture content determined by Proctor Compaction Test.
The soil should be placed in layers of about 100 mm thickness and fully compacted, then water should be sprinkled on the compacted layer before placing the next layer of 100 mm. The total height of this block achieved this way may be kept 500 to 800 mm. Before starting the new block, sufficient water should be poured on the completed layer to ensure its connection with the new layer.

7.13.3 Recommendations for Different Seismic Areas

a) Walls

The height of the adobe building should be restricted to one storey plus attic only in seismic zones V and IV and to two stories in zone III. Important building (I > 1.5) should not be constructed with earthen walls in seismic zones IV and V and restricted to only one storey in seismic zone III.
The length of a wall, between two consecutive walls at right angles to it, should not be greater than 10 times the wall thickness nor greater than 64 t2/h where h is the height of wall (Fig 7.13)
A long wall should be strengthened with intermediate vertical buttresses (Fig 7.13).

b) *Openings*

Openings have been shown in Fig 7.14

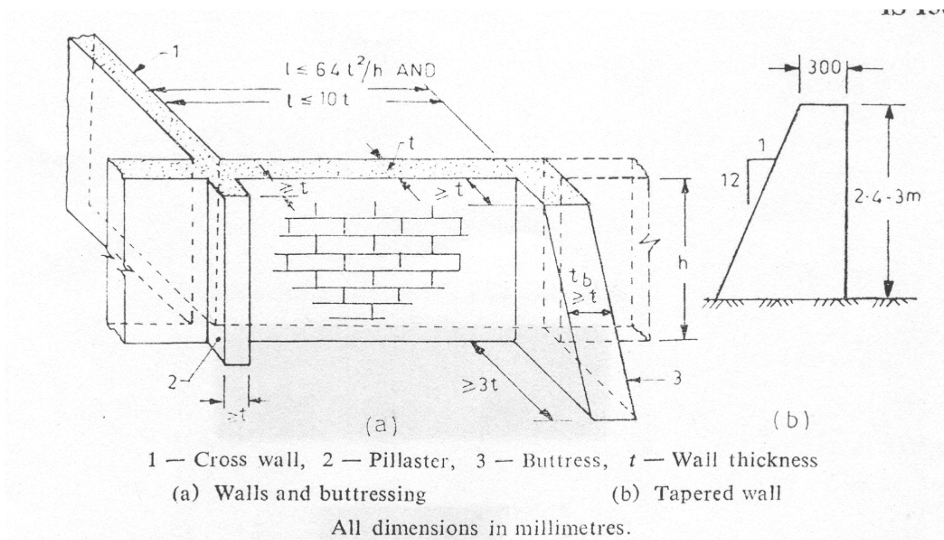


Figure 7.13: Wall Dimension

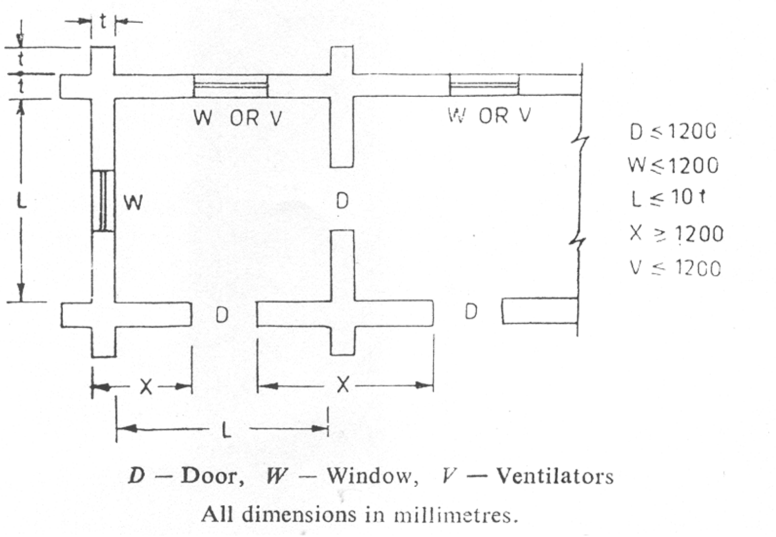


Figure 7.14: Wall dimensions, Pilasters at Corners

c) Foundation

The depth of foundation below existing ground level should be at least 400 mm. The footing should preferably be built by using stone, fired brick using cement or lime mortar. Alternatively, it may be made in lean cement concrete with plums (cement: sand: gravel: stones as 1: 4: 6: 10) or without plums as 1: 5: 10. Lime could be used in place of cement in the ratio lime: sand: gravel as 1: 4: 8.
The wall above foundation up to plinth level should preferably be constructed using stone or burnt bricks lay in cement or lime mortar. Clay mud mortar may be used only as a last resort. Height of plinth should be above flood water line or a minimum of 300 mm above ground level.

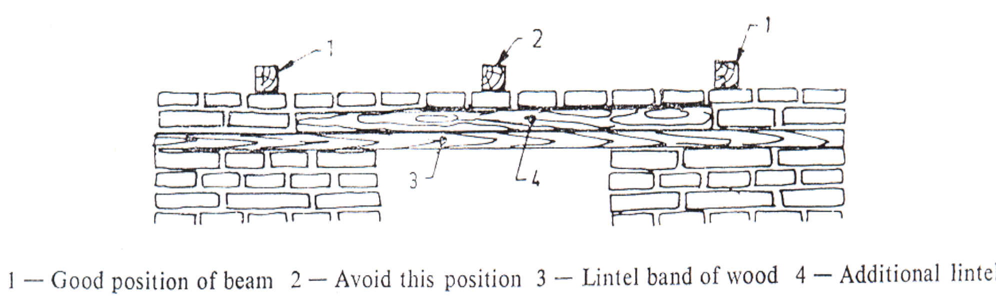


Figure 7.15: Reinforcing Lintel Under Floor Beam

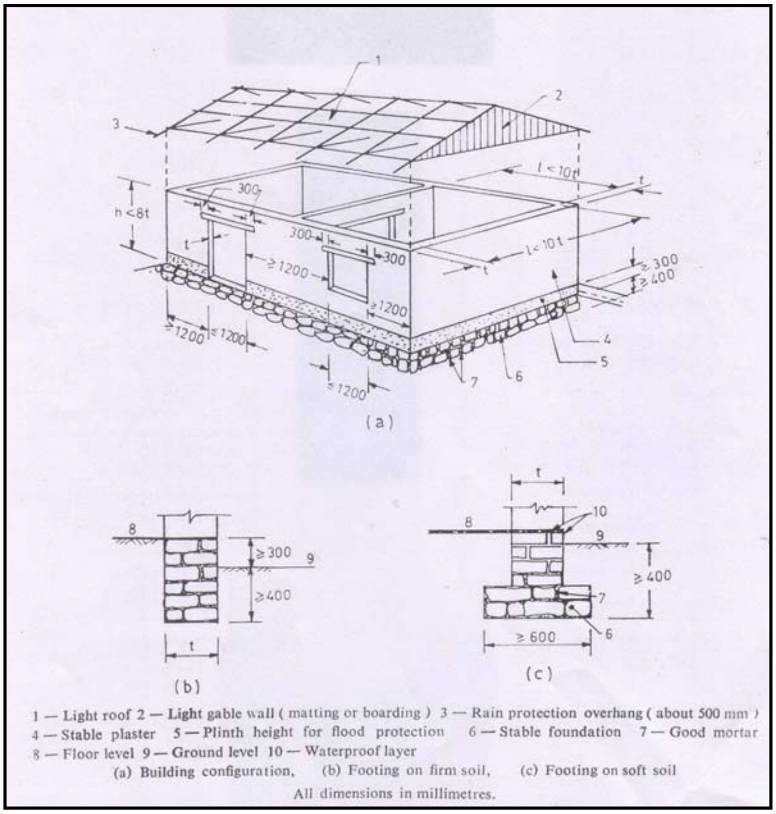


Figure 7.16: Adequate Configuration of Earthen Building

d) Roofs

The roofing structure must be light, well connected and adequately tied to the walls. Trusses are superior to sloping roofs consisting of only rafters or A frames. The roof beams, rafters or trusses should be rested on longitudinal wooden elements for distributing the load on walls.

The roofing beams or rafters should be located to avoid their position above door or window lintels. Otherwise, an additional timber should reinforce the lintel (Fig 7.15).

e) Adequate Configuration

Summarizing most of the recommendations contained in this standard, a configuration is shown in Fig.7.16, which will, in general, be adequate for seismic areas including Zones V and IV.

f) Collar Beam and Horizontal Band

Two horizontal continuous reinforcing and binding beams or bands should be placed, one coinciding with lintels of door and window opening, and the other just below the roof in all walls in seismic zones III, IV and V.

Proper connection of ties placed at right angles at the corners and junctions of walls should be ensured. Where the height of wall is not more than 2.5 m, the lintel band can be avoided, but the lintels should be connected to the roof band (Fig 7.17).

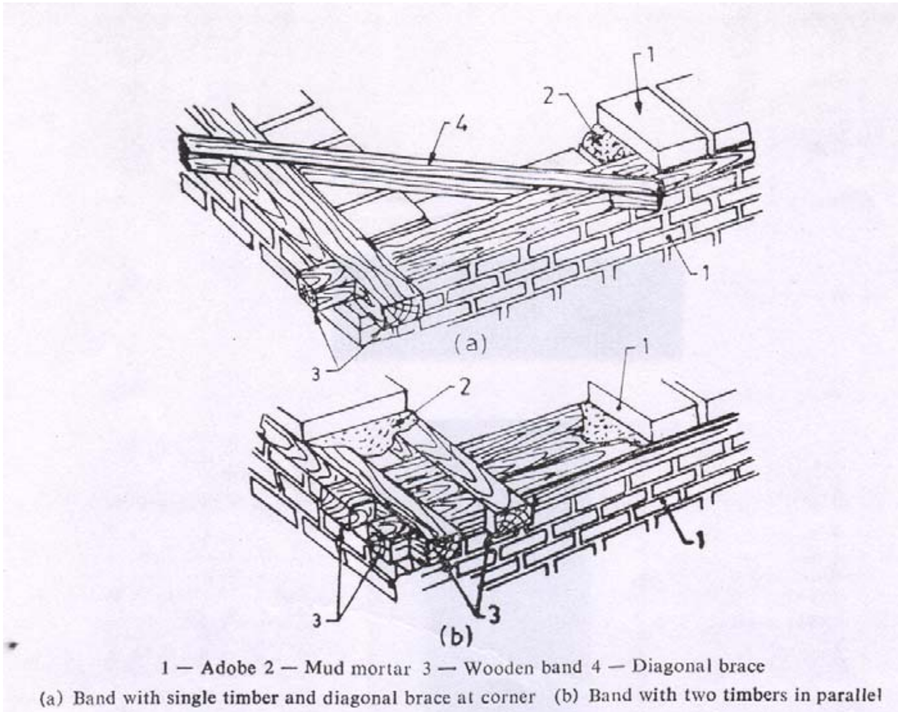


Figure 7.17: Wooden band in walls at lintel and roof levels

g) Pilasters and Buttresses

Where pilasters or buttresses are used, as recommended earlier at corner or T-junctions, the collar beam should cover the buttresses as well (Fig 7.18). Use of diagonal struts at corners will further stiffen the collar beam.

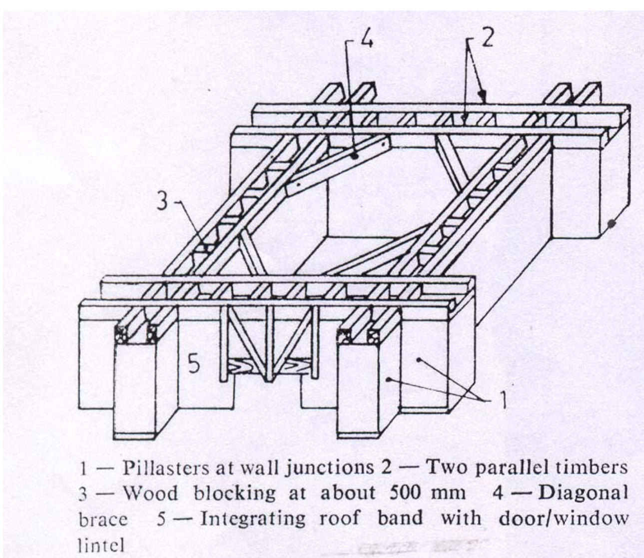


Figure 7.18: Roof Band on Pilastered Walls

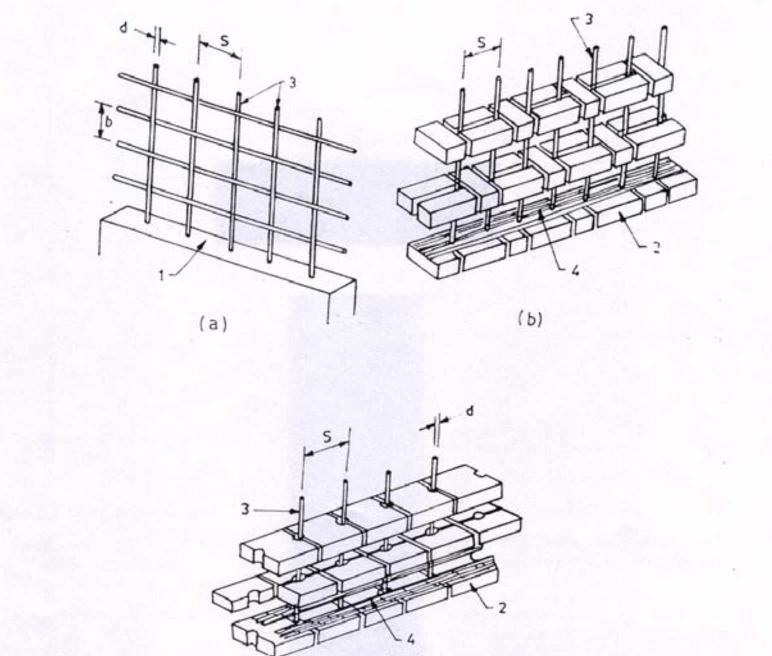


Figure 7.19: Reinforcement in Earthen walls

h) Vertical Reinforcement in walls

In seismic zone V, mesh form of reinforcing is recommended. A mesh of canes or bamboos (Fig 7.19) along with the collar beams (which may in this case be made from canes or bamboos themselves) acts as reinforcement for the wall. The vertical canes must be tied to the horizontal canes as well as the collar beam at lintel and the roof beam at eave level.

7.14 CONCLUSIONS

Earthquakes are treacherous and unpredictable and induce complex forces on buildings. A building must have the capacity to with stand these complex forces in elastic and inelastic range to retain its integrity. It requires fairly involved analysis and design effort.

If the building is simple, symmetrical, without any mass discontinuity and follows the desirable norms of earthquake resistant buildings, the level of this involved effort is reasonable and the additional cost of the structure is also reasonable and within limits.

If on the other hand the building is irregular, complicated and flaunts all these desirable norms, the level of this involved effort is very high, and the additional cost of the structure is also very high.

For the design of a building in regions of high intensity earthquake, whether the building is regular or irregular, an Architect must associate with a Structural Engineer who is conversant with earthquake resistant design principles.

Interaction between the Architect and the Structural Engineer must start at the concept stage.

The Architect must try to incorporate requirements of the structural engineer in his planning of the building and its aesthetics.

The structural engineer must give as much liberty as possible to the Architect to give expression to his creativity, without compromising structural integrity of the building.

In order to achieve this, knowledge of earthquake engineering is as essential for an Architect as for an Engineer.

CHAPTER 8

EARTHQUAKE RESISTANT CONSTRUCTION DETAILING

CHAPTER 8
EARTHQUAKE RESISTANT CONSTRUCTION DETAILING

8.1 INTRODUCTION

Considering the universally accepted philosophy of design against earthquake forces, all professionals, Architects, Engineers and Builders, who are involved with the design and construction of buildings, have a responsibility to the society to undertake the task to deliver safe shelter with utmost diligence, and follow principles, concepts and techniques which over the years have been identified to lead to safer buildings which have much better chances of survival against severe earthquakes.

8.2 FOUNDATION

8.2.1 Foundation Support or Soil Stabilization

If the expected settlement or lateral movement for a proposed structure is too large, then different foundation support or soil stabilization options must be evaluated.

One alternative is soil improvement methods. Instead of soil improvement, the foundation can be designed to resist the anticipated soil movement caused by the earthquake. For example, mat foundations or post-tensioned slabs may enable the building to remain intact, even with substantial movements.

Another option is a deep foundation system that transfers the structural loads to adequate bearing material in order to bypass a compressible or liquefiable soil layer.

A third option is to construct a floating foundation, which is a special type of deep foundation in which the weight of the structure is balanced by the removal of soil and construction of an underground basement. A floating foundation could help reduce the amount of rocking settlement caused by the earthquake.

Typical factors that govern the selection of a particular type of foundation are presented in Table 8.1.

8.2.2 Shallow Foundations

A shallow foundation is often selected when the structural load and the effects of the earth- quake will not cause excessive settlement or lateral movement of the underlying soil layers. In general, shallow foundations are more economical to construct than deep foundations. Common types of shallow foundations are described in Table 8.2 and shown in Fig 8.1. If it is anticipated that the earthquake will cause excessive settlement or lateral movement, then isolated footings are generally not desirable. This is because the foundation can be pulled apart during the earthquake, causing collapse of the structure. Instead, a mat foundation (Fig.8.2) or a post-tensioned slab is more desirable: This is because such foundations may enable the building to remain intact, even with substantial movements.

Table 8.1 Selection of Foundation Type

Topic	**Discussion**
Selection of foundation type	Based on an analysis of the factors listed below, a specific type of foundation (i.e., shallow versus deep) would be recommended by the geotechnical engineer.
Adequate depth	The foundation must have an adequate depth to prevent frost damage. For such foundations as bridge piers, the depth of the foundation must be sufficient to prevent undermining by scour.
Bearing capacity failure Settlement	The foundation must be safe against a bearing capacity failure. The foundation must not settle to such an extent that it damages the structure.
Quality	The foundation must be of adequate quality that it is not subjected to deterioration, such as from sulfate attack.
Adequate strength	The foundation must be designed with sufficient strength that it does not fracture or break apart under the applied superstructure loads. The foundation must also be properly constructed in conformance with the design specifications.
Adverse soil changes	The foundation must be able to resist long-term adverse soil changes. An example is expansive soil, which could expand or shrink, causing movement of the foundation and damage to the structure.
Required specifications	The foundation must be able to support the Structure during an earthquake without excessive settlement or lateral movement. The foundation may also have to meet special requirements or specifications required by the local building department or governing agency.

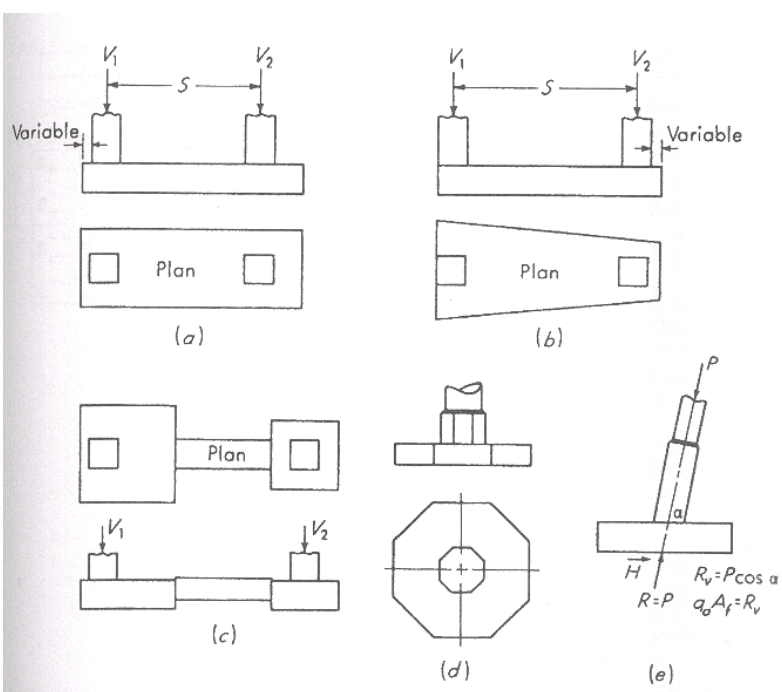


Figure 8.1: Examples of shallow foundations *(a)* Combined footing; *(b) C*ombined trapezoidal footing; *(c)* Cantilever or strap footing; (d) Octagonal footing; *(e)* Eccentric loaded footing with resultant coincident with area so soil pressure is uniform

Table 8.2 Common Types of Shallow Foundations

Topic	**Discussion**
Spread footings	Spread footings are often square in plan view, are of uniform reinforced concrete thickness, and are used to support a single load directly in the center of the footing.
Strip footings	Strip footings, also known as wall footings, are often used to support load-bearing walls. They are usually long, reinforced concrete members of uniform width and shallow depth.
Combined footings	Reinforced concrete combined footings are often rectangular or trapezoidal in plan view and carry more than one column load (see Fig. 8.1).
Other types of footings	Fig 8.1 shows other types of footings, such as the cantilever (also known as strap) footing, an octagonal footing, and an eccentric loaded footing with the resultant coincident with area so that the soil pressure is uniform.
Mat foundation	If a mat foundation is constructed at or near ground surface, then it is considered to be a shallow foundation. Fig 8.2 shows different types of mat foundations. Based on economic considerations, mat foundations are often constructed for the following reasons: 1. *Large individual footings:* A mat foundation is often constructed when the sum of individual footing areas exceeds about one-half of the total foundation area. 2. *Cavities or compressible lenses:* A mat foundation can be used when the subsurface exploration indicates that there will be unequal settlement caused by small cavities or compressible lenses below the foundation. A mat foundation would tend to span the small cavities or weak lenses and create a more uniform settlement condition. 3. *Shallow settlements:* A mat foundation can be recommended when shallow settlements predominate and the mat foundation would minimize differential settlements. 4. *Unequal distribution of loads:* For some structures, there can be a large difference in building loads acting on different areas of the foundation. A mat foundation would tend to distribute the unequal building loads and reduce the differential settlements. 5. *Hydrostatic uplift:* When the foundation will be subjected to hydrostatic uplift due to a high groundwater table, a mat foundation could be used to resist the uplift forces.
Conventional slab-on-grade	A continuous reinforced concrete foundation consists of bearing wall footings and a slab-on-grade. Concrete reinforcement often consists of steel rebar in the footings and wire mesh in the concrete slab.
Shallow foundation alternatives	If the expected settlement or lateral movement for a proposed shallow foundation is too large, then other options for foundation support or soil stabilization must be evaluated. Commonly used alternatives include deep foundations, grading options, or other site improvement techniques.

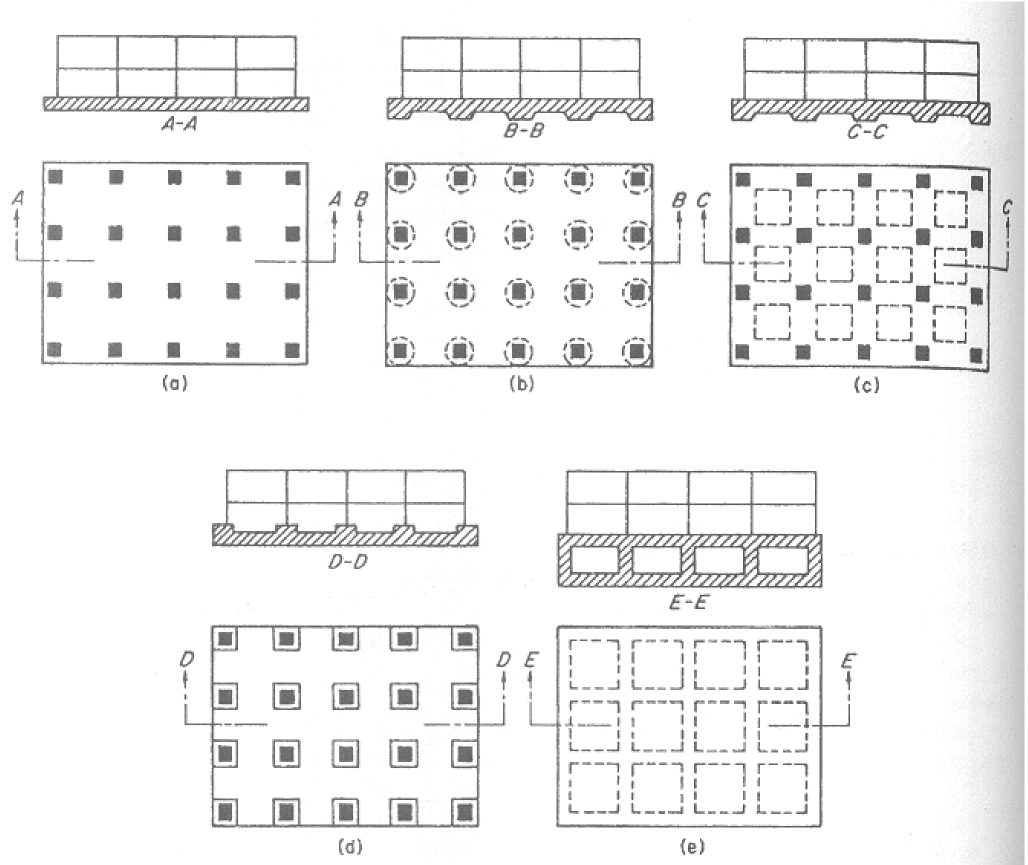


Figure 8.2: Examples of mat foundations *(a)* Flat plate; *(b)* plate thickened under columns; *(c)* beam- and-slab; *(d)* plate with pedestals; *(e)* basement walls as part of mat

8.2.3. Deep Foundations

Common types of deep foundations are described in Table 8.3 Fig 8.3 and Fig 8.4 show common types of cast-in-place concrete piles and examples of pile configurations.

Deep foundations are one of the most effective means of mitigating foundation movement during an earthquake. For example, the Niigata earthquake resulted in dramatic damage due to liquefaction of the sand deposits in the low-lying areas of Niigata City. At the time of the Niigata earthquake, there were approximately 1500 reinforced concrete buildings in Niigata City, and about 310 of these buildings were damaged, of which approximately 200 settled or tilted rigidly without appreciable damage to the superstructure. The damaged concrete buildings were built on very shallow foundations or friction piles in loose soil. Similar concrete buildings founded on piles bearing on firm strata at a depth of 20 m did not suffer damage.

Besides buildings, deep foundations can be used for almost any type of Structure. For example, concrete piles were used to support a storage tank. The soil beneath the storage tank liquefied during the Kobe earthquake, but still there was no reported damage to the storage tank. For earthquake conditions, two of the most commonly used types of deep foundations are the pier and grade beam system, and pre-stressed concrete piles.

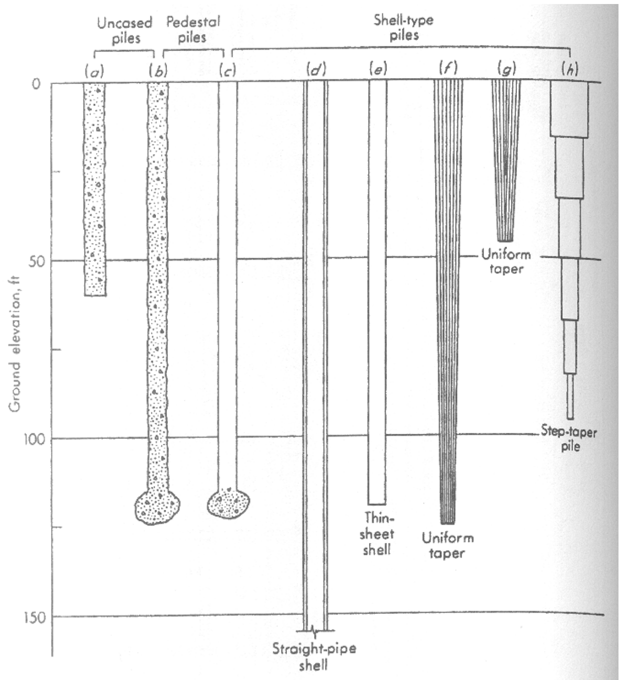


Figure 8.3: Common types of cast-in-place concrete piles:
(a) Uncased pile; *(b)* Franki uncased-pedestal pile; (c) Franki cased-pedestal pile;
(d) Welded or seamless pipe pile; *(e)* Cased pile using a thin sheet shell;
(f) mono-tube pile; *(g)* Uniform tapered pile; *(h)* step-tapered pile.

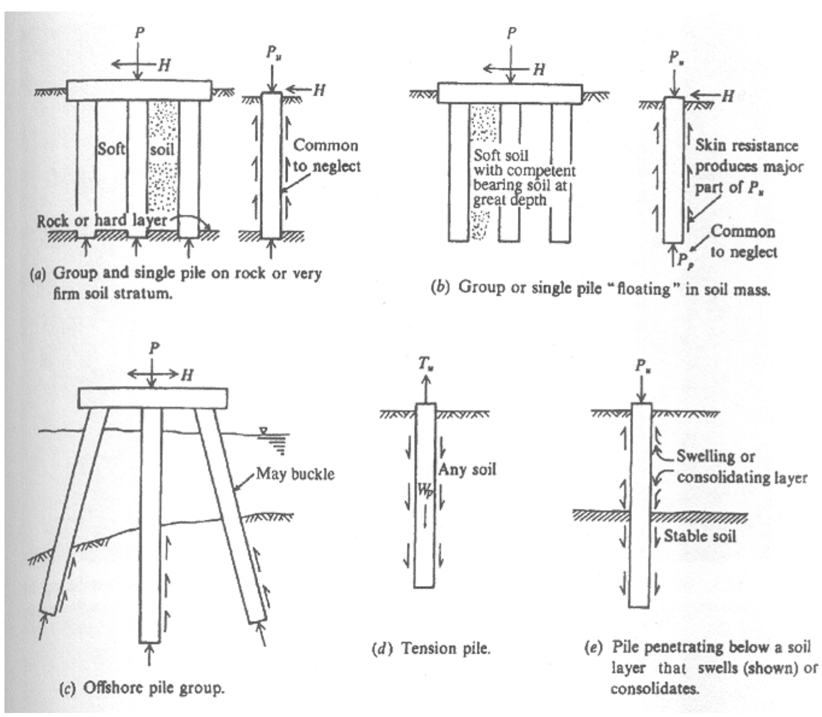


Figure 8.4: Typical pile configurations

Table 8.3: Common Types of Deep Foundations

Topic	**Discussion**
Pile foundations	Probably the most common type of deep foundation is the pile foundation. Piles can consist of wood (timber), steel H-sections, precast concrete, cast-in-place concrete, pressure-injected concrete, concrete-filled steel pipe piles, and composite-type piles. Piles are either driven into place or installed in predrilled holes. Various types of piles are as follows: *End-bearing pile:* This pile's support capacity is derived principally from the resistance of the foundation material on which the pile tip rests. End-bearing piles are often used when a soft upper layer is underlain by dense or hard strata. *Friction pile:* This pile's support capacity is derived principally from the resistance of the soil friction and/or adhesion mobilized along the side of the pile. Friction piles are often used in soft clays where the end-bearing resistance is small because of punching shear at the pile tip. A pile that resists upward loads (tension forces) would also be considered to be a friction pile. *Combined end-bearing and friction pile:* This pile derives its support capacity from combined end-bearing resistance developed at the pile tip and frictional and/or adhesion resistance on the pile perimeter. *Batter pile:* A pile driven in at an angle inclined to the vertical that provides high resistance to lateral loads. Piles are usually driven into specific arrangements and are used to support reinforced concrete pile caps or a mat foundation.
Concrete-filled steel pipe piles	In this case, the steel pipe pile is driven into place. The pipe pile can be driven with either an open or a closed end.
Pre-stressed concrete piles	Typical pre-stressed concrete piles are delivered to the job site and then driven into place.
Piers	A pier is defined as a deep foundation system, similar to a cast-in-place pile that consists of a column like reinforced concrete member. Piers are often of large diameter and also commonly referred to as drilled shafts, bored piles, or drilled caissons.
Caissons	Large piers are sometimes referred to as caissons. A caisson can also be a watertight underground structure within which work is carried on.
Mat or raft foundation	If a mat or raft foundation is constructed below ground surface or if the mat or raft is supported by piles or piers, then it should be considered to be a deep foundation system.
Floating foundation	A floating foundation is a special type of deep foundation where the weight of the structure is balanced by the removal of soil and construction of an underground basement.

8.3 SOIL STABILISATION

Soils have been modified to improve their engineering properties for hundreds of years. In the past 75 years, however, improved knowledge of soil behavior and geotechnical hazards has led to the development and verification of much innovative soil improvement techniques. Increased recognition of seismic hazards and improved understanding of the factors that control them have led these techniques to be applied to the mitigation of seismic hazards in the past 30 years.

In both seismically active and inactive areas, soil improvement techniques are commonly used at sites where the existing soil conditions are expected to lead to unsatisfactory performance. Unsatisfactory performance can take many forms, but usually involves unacceptably large soil movements. The movements may include horizontal or vertical (or both) components and may take place during and/or after earthquake shaking. In the absence of earthquake shaking, unacceptable movements usually result from insufficient soil strength and/or stiffness. Consequently, most soil improvement techniques were developed to increase the strength and stiffness of soil deposits.

During earthquakes, other factors can contribute to unacceptable performance. In particular, the buildup of excess pore water pressure can lead to very large deformations. Consequently, commonly used techniques for mitigation of seismic hazards often involve reducing the tendency of the soil to generate positive excess pore water pressure during earthquake shaking as well as increasing the strength and stiffness of the soil.

Advances in soil improvement technology have generally resulted from the initiative and imagination of contractors. Research and explanatory "theories" have followed, rather than led, implementation; for some widely used techniques, proven theories have yet to be developed. In such cases, indirect or empirical evidence must be relied upon and the study of case histories is particularly important. At present, a wide variety of soil improvement techniques are available for mitigation of seismic hazards. The costs of these methods vary widely, and the conditions under which they can be used are influenced by the nature and proximity of structures and constructed facilities. On the basis of the mechanisms by which they improve the engineering properties of the soil, the most common of these can be divided into four major categories: densification techniques, reinforcement techniques, grouting/mixing techniques, and drainage techniques.

8.4 RETAINING WALLS

Earth retaining structures, such as retaining walls, bridge abutments, quay walls, anchored bulkheads, braced excavations, and mechanically stabilized walls, are used throughout seismically active areas. They frequently represent key elements of ports and harbors, transportation systems, lifelines, and other constructed facilities. Earthquakes have caused permanent deformation of retaining structures in many historical earthquakes. In some cases, these deformations were negligibly small; in others they caused significant damage. In some cases, retaining structures have collapsed during earthquakes, with disastrous physical and economic consequences. This section discusses the behavior of retaining walls during earthquakes and presents several of the most common approaches to the seismic design of different types of retaining walls.

8.4.1 Types of Retaining Walls

The problem of retaining soil is one of the oldest in geotechnical engineering; some of the earliest and most fundamental principles of soil mechanics were developed to allow rational design of retaining walls. Many different approaches to soil retention have been developed and used successfully. In recent years, the development of metallic, polymer, and geo-textile reinforcement has led to the development of many innovative types of mechanically stabilized earth retention systems.

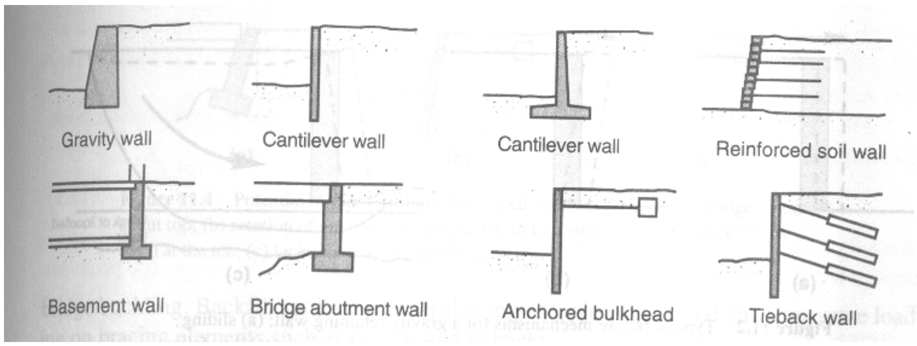


Figure 8.5: Common types of earth retaining structures

Retaining walls are often classified in terms of their relative mass, flexibility, and anchorage conditions. Gravity walls are the oldest and simplest type of retaining wall (Fig 8.5). Gravity walls are thick and stiff enough that they do not bend; their movement occurs essentially by rigid-body translation and/or rotation. Certain types of composite wall systems, such as crib walls and mechanically stabilized walls, are thick enough that they bend very little and consequently are often designed as gravity walls (with appropriate consideration of internal stability). Cantilever walls, which bend as well as translate and rotate, rely on their flexural strength to resist lateral earth pressures. The actual distribution of lateral earth pressure on a cantilever wall is influenced by the relative stiffness and deformation of both the wall and the soil. Braced walls are constrained against certain types of movement by the presence of external bracing elements. In the cases of basement walls and bridge abutment walls, lateral movements of the tops of the walls may be restrained by the structures they support. Tieback walls and anchored bulkheads are restrained against lateral movement by anchors embedded in the soil behind the walls. The provision of lateral support at different locations along a braced wall may keep bending moments so low that relatively flexible structural section can be used.

8.4.2 Seismic Design Consideration of Retaining Walls

The design of retaining walls for seismic conditions is similar, in many respects to designing for static conditions. In both cases, potential modes of failure are identified and the wall is designed to avoid initiating them. Although the response of retaining walls under seismic loading conditions is much more complex than under static conditions, conventional design procedures make use of simplifying assumptions that render the problem tractable.

8.5 FLOORS AND ROOFS

Flexible floors and roofs contribute to damage as lateral force because its own mass is not transferred to all the walls by rigid diaphragm action. The lateral load is transferred as concentrated load at few places and only on few walls. Horizontal bracings are to be provided at tie level to increase rigid diaphragm action of flexible roofs.

Heavy roof attracts large seismic force which is too high to resist for the walls supporting the roof. Triangular portion of the wall below the inclined roof is called gable end and which is extremely vulnerable to earthquakes. Hence, heavy roof made of stone or thick layer of mud is more prone to damage as compared to light tin sheet roof. Gable bands must be provided in trussed roofs.

8.6 OPENINGS

Use of too many and very large windows and doors should be avoided as these reduce the strength of wall. IS 4326 and IS 13828 recommend a minimum horizontal distance of 45 cm and a minimum vertical distance of 60 cm between any two openings. Avoid openings close to cross walls and edges. The total width of all openings must preferably be kept less than half the width of the wall for one storied houses, and less than one third for two storied houses.

However, means of access are of particular concern, during evacuation of a damaged building after an earthquake. It may be an important safety precaution and they should be clear of obstruction. The same should be available to rescue and inspection personnel entering the building. Besides the structural integrity of the building access routes should have protected ceilings, partitions and stairway enclosures.

8.7 APPENDAGES

8.7.1 Vertical projections

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

8.7.2 Horizontal projections

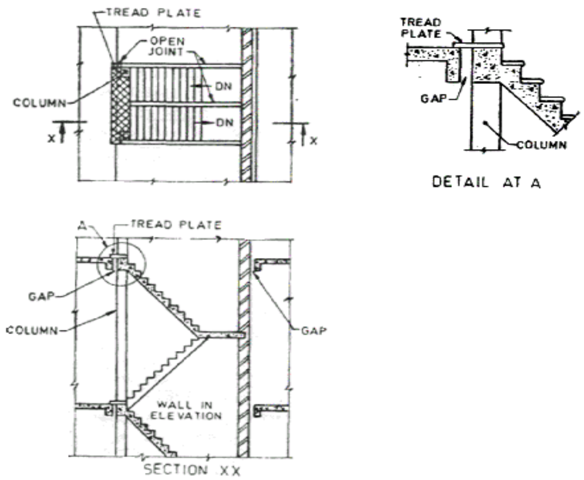
All horizontal projections like cornices and balconies shall be designed and checked for stability for five times the design vertical coefficient equal to 10/3 A_h.

8.8 BOUNDARY WALL

Compound walls shall be designed for the design horizontal coefficient A_h with importance factor $I = 1.0$.

8.9 STAIRCASES

IS 4326:1993 states that the interconnection of the stairs with the adjacent floors should be appropriately treated by providing sliding joints at the stairs to eliminate their bracing effect on the floors. Large stair halls shall preferably be separated from the rest of the buildings by means of separation or crumple section. Three types of stair construction may be adopted: separated staircase, Built-in stair case, and Staircases with sliding joints (Fig.8.6).



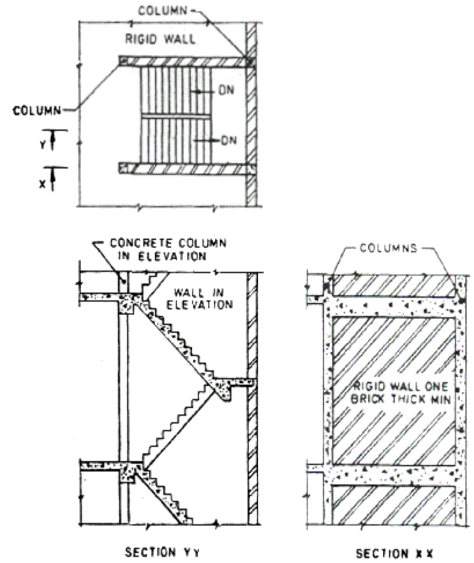


Figure 8.6 a: Separated Staircase   Figure 8.6 b: Rigidly built-in staircases

8.10 SEISMIC JOINTS

Two adjacent buildings or two adjacent units of the same building with separation joint in between shall be separated by a distance equal to the amount R times the sum of the calculated storey displacements, to avoid damaging contact when the two units deflect towards each other (Pounding Effect). Fig 8.7 shows the details of separation joints.

When floor levels of two similar adjacent units or buildings are at the same elevation levels, factor R in this requirement may be replaced by R/2.

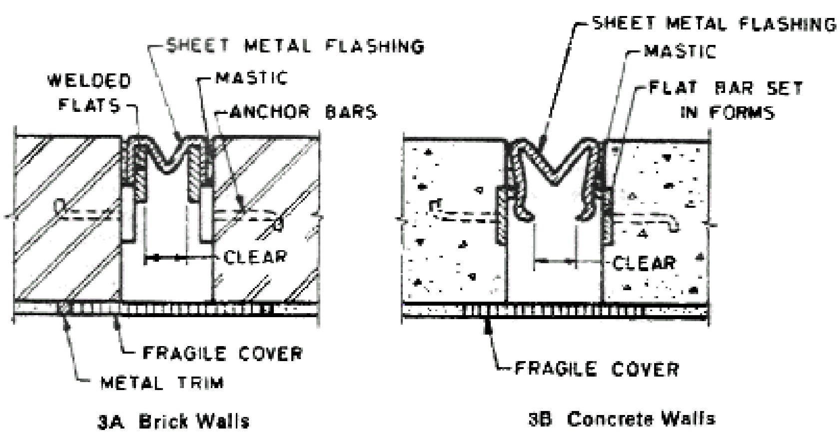


Figure 8.7: Separation Joints

IS 4326:1993 earthquake resistant design and construction of buildings says seismic joint or separation of adjoining structure or part of the same structure is required for structures having different total heights or storey heights and different dynamic characteristic.
In case of seismic joint, a complete separation of the parts shall be made except below the plinth level. For the seismic joints in long buildings, movement due to temperature changes shall be taken care. Min width of expansion joint is given in Table 8.4.

Table 8.4: Seismic joint

Sl. No.	Type of Construction	Gap Width / Storey, in mm for Design Seismic Coefficient = 0.12
1	Box system of frames with shear walls	15.0
2	Moment resistant reinforced concrete frame	20.0
3	Moment resistant steel frame	30.0
Notes: Minimum total gap shall be 25 mm. For any other value, width of the gap shall be determined proportionately.		

8.11 NON-STRUCTURAL ELEMENTS

8.11.1 Introduction

In regions of very high seismic activity, it is important to give sufficient attention to all those non-structural elements of a building, which come in contact with the structural frames and shear walls. Either through suitable separation, it has to be ensured that non structural elements are not effected by the movements and distortion of structural elements, or else the connections should be so made that contact of the non-structural elements to structural elements is with a soft material which can deform.
In case, none of these two solutions are adopted, and brittle non-structural elements are in direct contact with the structure, one should be prepared to carry out extensive repairs to the in-fill brittle elements after a strong earthquake.

Awareness about structural systems' safety is more pronounced and that gets reflected in code provisions and in practical application. Attention paid to the safety of non-structural elements is less. Reasons are many. Primary one is structural collapse is much more life threatening than scattered falling of non-structural elements. Any damage resulting in partial or full closure of built spaces due to natural hazard would cause loss. Structural retrofitting decision would be a partial solution towards safety from Earthquake hazards, unless the problem of non-structural elements is attended. Non-structural damage can modify structural response. Non-structural elements are important to safeguard, otherwise continuity of work would be disrupted resulting in

- Economic Loss
- Loss of Building Function
- Structural Response Modification

Buildings with all essential facility must be safe or usable for emergency purposes after an earthquake in order to preserve health and safety of general public. Hospitals and others medical facilities, Fire & Police stations, Municipal Government Disaster mitigation operation and Communication center etc. are very important buildings during emergency. Non-structural elements in buildings are 60 to 70 % of total cost. Damage to them causes higher aggregate loss. Therefore non-structural Elements needs to be constructed / used with seismic considerations.

8.11.2 Non-structural damage

Non-structural damage can be categorized as:

1. • Architectural components
2. • Mechanical & electrical components
3. • Building contents and equipments

1. Architectural components

- **Exterior elements:** Cladding, Veneers, Glazing, In filled walls, Canopies, Parapets, Cornices, Appendages, Ornamentation, roofing, Louvers, Doors, Detached planters, Signs etc.

- **Interior elements:** Partitions, Ceilings, Stairways, Storage racks, Shelves, Doors, Glass, Furnishings, (File cabinets, Book case, Library stacks, Display cases, Desks, Chairs, Lockers, Etc.), Ornamentation, Detached planters, Artwork, etc.

2. Mechanical / Electrical / Plumbing elements:

HAVC equipments, Elevators, Piping, Ducts, Electric panel board, Life support system, Fire protection system, Telephone, Communication system, Motors, Power control system, Emergency Generators, Tanks, Pumps, Escalators, Boilers, Chillers, Fire extinguishers, Controls, Light fixtures, etc.

3. Contents

Electronic Equipment, data processing facilities, Medical suppliers, Blood bank Inventories, High tech equipment, hazardous & Toxic materials, Antiques, Fine arts, (Museum & art galleries) Office equipments, Radios, Life support equipments, etc.

8.11.3 Hazards and causes to damage

Four types of hazards can be caused due to damage to Non-structural elements

i) **Direct hazards –** casualties due to broken glass, light fixtures, appendages.
ii) **Loss of critical function –** loss of electric power and communication to hospitals, police, fire, etc.
iii) **Release of hazardous materials –** chemical, radiation etc.
iv) **Fire caused –** due to gas leakage, electric lines disruption, etc.

The causes of non-structural damages are primarily two: acceleration and displacement.

8.12 CONCLUSIONS

Earthquakes are treacherous and unpredictable and induce complex forces in buildings. A building must have the capacity to with stand these complex forces in elastic and in elastic range to retain its integrity. To design a building to have this capacity requires fairly involved analysis and design effort. If the building is simple, symmetrical, without any mass discontinuity and follows the desirable norms of earthquake resistant buildings, the level of this involved effort is reasonable and the additional cost of the structure is also reasonable and within limits. If on the other hand the building is irregular, complicated and flaunts all these desirable norms, the level of this involved effort is very high, and the additional cost of the structure is also very high.

For the design of a building in regions of high intensity earthquake, whether the building is regular or irregular, an Architect must associate with a Structural Engineer who is conversant with earthquake resistant design principles. Interaction between the Architect and the Structural Engineer must start at the concept stage. The Architect must try to incorporate requirements of the structural engineer in his planning of the building and its aesthetics. The structural engineer must give as much liberty as possible to the Architect to give expression to his creativity, without compromising structural integrity of the building. In order to achieve this, knowledge of earthquake engineering is as essential for an Architect as for an Engineer.

CHAPTER 9

CONSTRUCTION QUALITY CONTROL

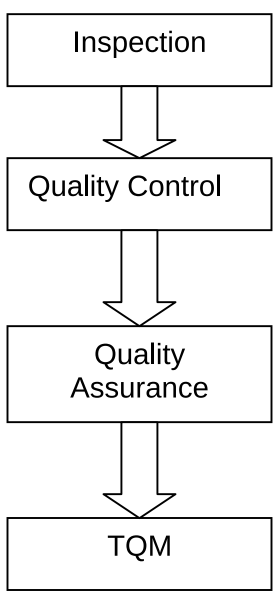
CHAPTER 9
CONSTRUCTION QUALITY CONTROL

9.1 INTRODUCTION

It has been observed all over the world that buildings suffer damages in earthquakes because of their construction quality. Buildings constructed according to the standards specified for different zones with proper quality control in materials used and construction techniques have performed much better than those built with substandard materials, inadequate bonding, insufficient curing etc. The aim of construction quality control is attaining those construction details, which will not only improve the looks of the building but also increase the strength, durability and resistance to the forces of the earthquake. It is seen that in most cases, achievement of good quality does not involve extra cost, but only extra care and better understanding.

9.2 EVOLUTION OF QUALITY MANAGEMENT

Construction quality essentially has three components viz. conformance to the requirements, fitness to the purpose and customer satisfaction. There is always a difference between the expected performance and the observed performance of any structure during earthquake. But in any case failure of structure is unacceptable. Fig 9.1 gives evolution of quality management.



Salvage, sorting, grading, blending, corrective actions, identify sources of non-conformance

Develop quality manual, process performance data, self-inspection, product testing, basic quality planning, use of basic statistics, paperwork control

Quality systems development, advanced quality planning, comprehensive quality manuals, use of quality costs, involvement of non-production operations, failure mode and effects analysis, SPC.

Policy deployment, involve supplier & customers, involve all operations, process management, performance measurement, teamwork, employee involvement.

Figure 9.1: Evolution of quality management

9.3 REASONS FOR POOR CONSTRUCTION

All poor constructions initially are insignificant and tolerable, but with passing time develop various problems as given below:

1. Seepage/Leakage
2. Separation
3. Delamination
4. Cracks
5. Subsidence, settlement
6. Collapse
7. Spalling
8. Flaking
9. Curling
10. Warping
11. Corrosion
12. Bulging
13. Sagging

The poor constructions are basically due to following reasons:

1. Improper or inadequate design;
2. Improper construction;
3. Inadequate maintenance; and
4. Durability problems/Exposure.

Architects are responsible for the first two reasons viz. improper or inadequate design and improper construction; hence these two are discussed in detail.

9.3.1 Improper or inadequate design

Improper or inadequate design is the responsibility of architect or structural engineer. Many a times wrong or poor reinforcement detailing results in to defects like improper rebar placement, leading to problems like congestion of rebar's, inadequate cover etc. The inadequate specifications can also give scope to the on site workers to decide for some crucial detailing aspects.

Architects may make mistakes in specifications because of wrong interpretation of building codes. These days various computer softwares are used for structural detailing and drafting job. This also results in crucial mistakes if there is misuse of these softwares. Also improper execution sequence which may not be specified in the design can lead to serious errors in the construction. Generally the improper or inadequate design is a result of one or more of the following:

1. No soil investigations;
2. Proto-type designs;
3. Under design:
   - Incorrect assumption of Loads,
   - Loads not considered,
   - Errors, omissions and mistakes in design calculations,
   - Poor detailing and drafting,
   - Inadequate provisions for secondary stresses like shrinkage etc.
4. Lower grades;
5. Lower sections;
6. Future expansions;
7. Lower specifications and
8. Irregular form

9.3.2 Improper construction

Many times when the architects or structural engineer's job is satisfactory, i.e. there is no inadequacy or improperness in the design given, but still the resultant construction can be poor. This can be due to various reasons viz. inferior material, inferior supervision or inferior machinery.

Inferior materials

In India use of inferior construction material is common. Architects and the owner of the building have to be careful about this problem and check the quality of their building material from time to time. The material may be second grade or cheap for saving of money. Lack of inspection in procuring, handling and storing of material should be avoided to ensure quality of materials. Inferior workmanship can also reduce the quality of material to great extents.

Inferior supervision

Incompetent construction staff would lack good supervision of construction activity and hence result in poor construction. The inferior supervision can also be due to lack of training and lack of will of the staff. These two factors would result in poor interpretation of drawings and hence wrong or poor constructions. The staff can also encourage poor constructions for saving money for themselves.

Inferior machinery

Inferior or defective machinery with poor manufacturing standards and poor maintenance, can lead to poor constructions. The improper placement of machinery by the site staff, and the absence of trained operator for the machinery can result in improper functioning of machinery leading to undesirable results.

Photographs of various construction defects found in RCC buildings are given in following figures. (Fig 9.2 to Fig 9.18)

Figure 9.2: Improper alignment of column

Figure 9.3: Congestion of reinforcement

Figure 9.4: Inferior construction

Figure 9.5: Poor road construction

Figure 9.6: Improper gaps

Figure 9.7: Wrong alignment of wall

Figure 9.8: Leaking Toilets

Figure 9.9: Stains and spalling of columns

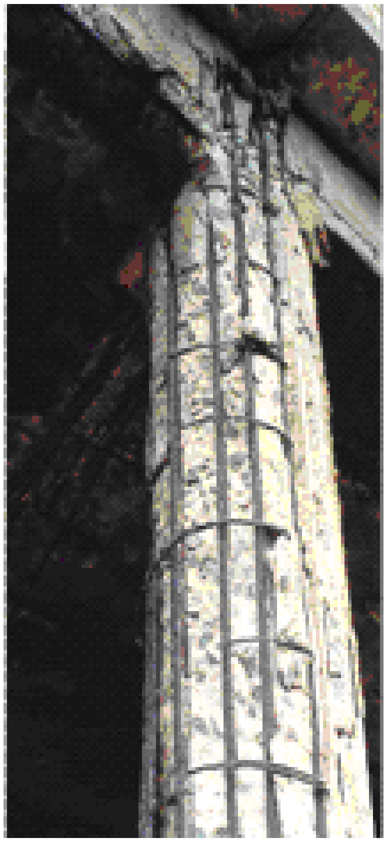


Figure 9.10: Spalling of columns

Figure 9.11: Poor workmanship of brick masonry

Figure 9.12: Disturbance of structural elements

Figure 9.13: Discontinuity of designed structural members

Figure 9.14: Wrong installation of window frame

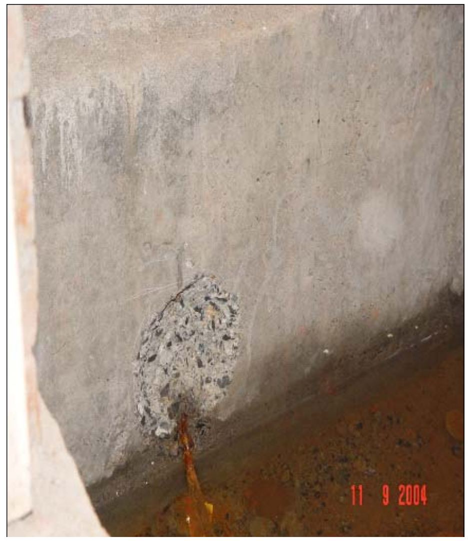


Figure 9.15: Corrosion of reinforcement

Figure 9.16: Poor workmanship

Figure 9.17: Construction defects

Figure 9.18: Wrong placement of structural members

9.4 CONSTRUCTION QUALITY CONTROL IN MASONRY STRUCTURES: QUALITY OF STONE MASONRY WALLS

Good quality construction work including repair, retrofitting and reconstruction of the building is very important to meet the following objectives:

1. To achieve the strength of the building under Normal Dead and Live Loads. Foundation, walls, column, floor and roof are the main structural elements to be taken care of.
2. To achieve adequate strength of the building for earthquake effects. The various details including plinth band, lintel band and vertical reinforcing are specified in IS : 4326 & 13828 of 1993.
3. To achieve durability of the building over long time so as to meet the seismic requirements as and when the next earthquake strikes.

Every effort should therefore be made by all concerned to achieve the specified standards. It should also be realized that the constructions are and will be keenly watched by the prospective beneficiaries. Hence quality must be ensured by all building agencies in architectural and structural designs as well as construction.

The strength of coursed rubble (RCR) masonry under vertical as well as horizontal earthquake loads depends upon the integrity of the wall cross-section and the bond between perpendicular walls. To achieve high strength of rubble masonry the following measures must be ensured:-

9.4.1 Interlocking of Stones and Breaking Vertical Joints.

To achieve the integral behavior of the stone wall, interlocking of the stones in the cross-section as well as in the length of the wall is necessary and ‘stacking bond' should not be done as shown in Fig 9.19 in which vertical joints occur continuously. The proper interlocking of stones including breaking of vertical joints is shown in Fig. 9.20 and Fig 9.21.

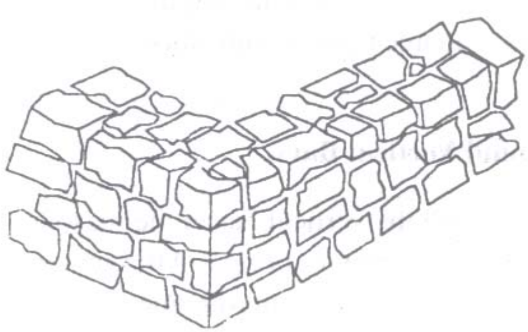


Figure 9.19 Stacked stone wall (poor construction)

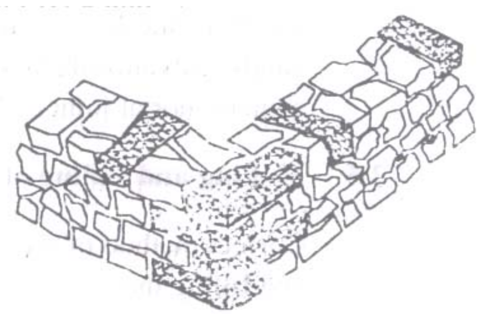


Figure 9.20: Interlocking stone wall with ‘through’ and ‘corner’ stones (very good construction)

9.4.2 Provision of 'through' stones

'Through' stones must be provided in the wall at horizontal intervals of 1.2 m and vertical interval of 0.6 m (Fig 9.20 and Fig 9.21). Where such long stones are not available, concrete blocks, cast using 1:3:6 or M1O concrete, having 150 x 150 mm cross section and length equal to the thickness of the wall may be used instead. Such bonding elements are to be provided whether the wall is 350-380 mm thick using cement mortar or 400-450 mm thick using mud mortar.

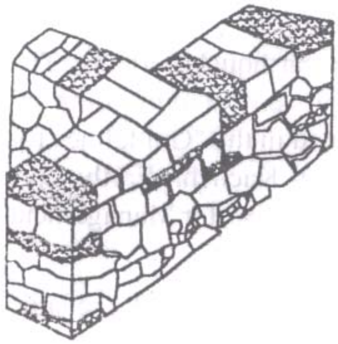


Figure 9.21:
Interlocking stone wall with "through' stones

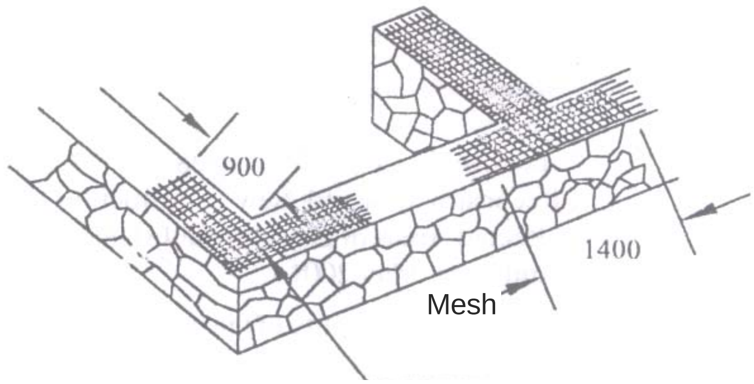


Figure 9.22: Galvanized chicken wire mesh *(double layer)* at corner and T-junction at window sill level

9.4.3 Provision of Long Stones at all Wall Junctions

For ensuring good bond between perpendicular walls, long stones are to be provided at the corners as well as T-junctions (Fig. 9.20 and Fig. 9.21). Such stones should be about 50 cm long in the case of 350-380 mm thick walls and about 60 cm long in 400-450 mm thick walls. Where such stones are not available, solid concrete blocks of 150 x 150 mm in section and 500 or 600 mm long (cast using 1:3:6 concrete) may be used instead.

9.4.4 Guarding against Vertical Separation between Perpendicular Wall

Safeguarding the walls against splitting at the vertical joint between any two perpendicular walls if at the window side levels, is recommended (that is, about mid way between the plinth and lintel bands). Galvanized chicken wire mesh in double layer should be embedded in the 1:4 cement mortal joint as shown in Fig 9.22.

9.4.5 Making and Filling of Pockets around Vertical Bars

Correct method of creating a pocket around the vertical bar in the masonry and filling the pocket with micro-concrete is shown in Fig. 9.23. After moving the 75 mm dia plastic pipe upward out of the masonry, micro-cement (coarse aggregate below 1.0 mm) is to be filled in the hollow and tamped by means of 10 mm dia rods.

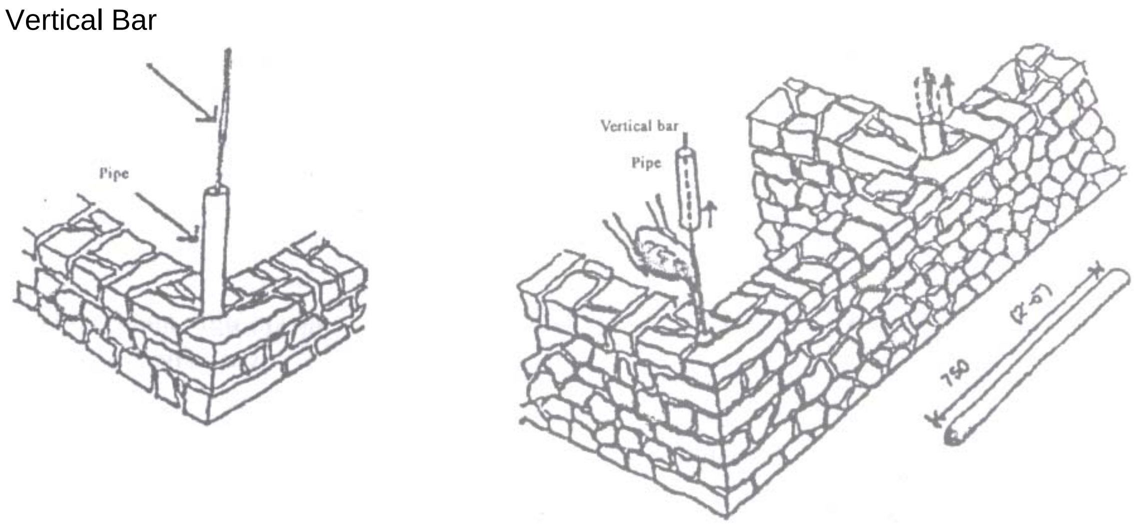
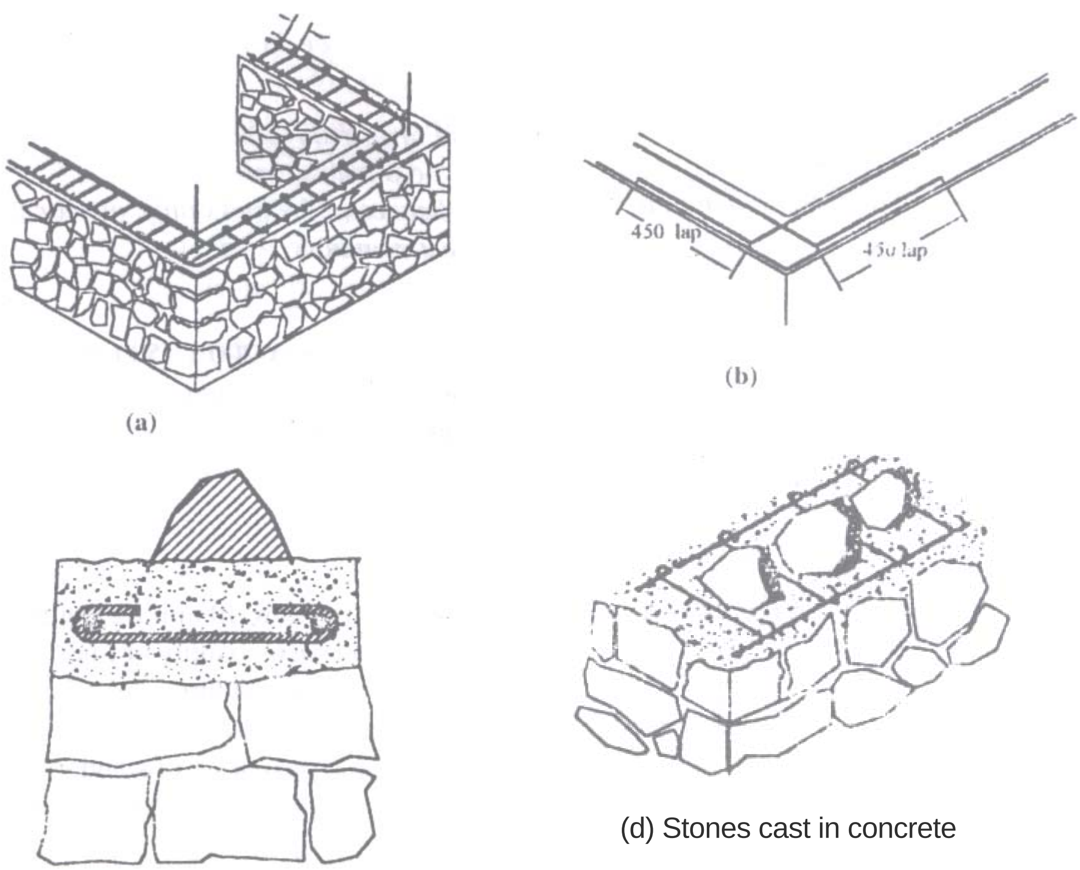


Figure 9.23: Method of filling micro-concrete around vertical bars in masonry.



(d) Stones cast in concrete

(c) Cross section

Figure 9.24: Seismic band at Plinth or Lintel level in stone walls

9.4.6 Bending of Bars and Casting of Seismic Bands

Fig 9.24 shows the bending of longitudinal bars with adequate overlaps and tying of cross links with them. This will ensure full continuity of the longitudinal reinforcement at the corners. It is also suggested that stones may be cast in the band-concrete so that 1/4 to 1/3 of stone remains projecting outside the concrete. This arrangement will save the amount of concrete as well as establish excellent bond between the band and the wall masonry.

9.4.7 Curing of Stone Wall Built using Cement Mortal

All newly constructed masonry should be covered with polythene sheets to prevent fast drying of mortar in the very dry conditions. Covering the walls with such sheets after sprinkling water for curing will help preserving the moisture over longer periods thus saving consumption of water.

9.5 QUALITY OF CONCRETE BLOCK WALL

The strength and stability of the wall will depend on the strength and quality of the blocks, the quality of the mortar and construction of the wall. The wall should be built truly vertical by frequent checking using a plumb.

The dry blocks should be wetted before laying so that they do not suck the water from the mortar. Cement mortar should he freshly mixed and must be fully consumed within 60 minutes of cement mixing with water to avoid setting of cement before laying.

After the wall portions are constructed, they shall be cured for a minimum period of 7 days by frequent watering. Uncured mortar does not set and its proper strength is not achieved. (In Manjil earthquake of 1990 in Iran, the main cause of destruction of brickwork constructed using cement mortar was that the bricks were not soaked in water before laying and no curing was done after construction). Needless to say that proper bond should be maintained to break the vertical joints and the vertical joints between the blocks must be fully filled with mortar.

Tests conducted in the Tehran laboratory on brick walls under lateral pressure showed that unsoaked and uncured brick walls failed at a lateral pressure of only 1/770 of that required for breaking the fully cured walls built by soaking the bricks.
To ensure filling of vertical joints between the blocks fully, mortar grout may be used before starting the next coarse.

9.5.1 Quality of Concrete Blocks

The main wall material is concrete blocks of 300 x 200 x 150 mm nominal (290 x 200 x 140 mm actual size) laid in 1:6 cement-sand mortar. To obtain good quality concrete blocks of adequate strength, 5.0 MPa (50 kg/sqcm at 28 days) and imperviousness as well as strength of wall, the following points are to be implemented.

Mix Design

Concrete mix should be well graded, with a few 40 mm size aggregate, coarse sand of high fineness modulus and such quantity of fines so as to fill the voids. The proportions should be arrived at through a mix-design method or by trials, by using the locally available materials. For this purpose each block making center should have a set of standard sizes and a weighing machine for determining the Fineness Modulus values of the materials.

Overall Mix

Cement 1 part to 15 parts of the fine + coarse aggregates, measured by volume.
The aggregates consist of the following:

Coarse sand (FM 3.17)
Stone dust below 6 mm (FM 3.67)
Crushed grit 6 to 10 mm (FM 4.11)
Crushed aggregates 12 to less than 40 mm
Hand broken aggregates 40 to less than 50 mm.

Testing of sand

The coarse sand should not contain more than 8 % of the silt by weight. Silt content may be tested by using graduated cylinder method, held stationary for minimum 12 hours. Record of silt content should be kept at block making/construction site.

If silt content is found to be more than permissible, the cement should be increased by trial to achieve the desired strength.

Bulking of moist sand has to be tested and the quantity of sand is to be adjusted to achieve design mix. A record of bulking test may also be maintained.

Making of blocks

After ensuring the design mix by proper control of measurement of the materials, proper mixing in the mixer and compaction by vibrating machines are the key factors. The large stone pieces have a tendency of either coming to the top or settling below the mix in the mixer. Uniform mixing should be ensured by adjusting the angle of the revolving mixer. Regarding compaction, it should be noted that 10 percent less compaction may reduce strength by 40 to 50 percent, hence adequate compaction must be ensured

In many rural and town areas, blocks are being made by hand moulding with uncontrolled or without compaction. In such cases even a concrete mix of 1:4:8 (i.e. I part of cement to 12 parts of fine + coarse aggregates) may not give the desired minimum strength of 50 kg/sqcm. Hence thorough compaction by vibrators or by adequate welding using 16mm dia rods of 400 mm length is absolutely necessary for strength as well as economy in the use of cement

In order to increase the horizontal shear strength of cement block walls, the blocks may be made with a 'frog' as in bricks on its face. For a 290 x 200 x 140 mm block, the 'frog' may be made 150 x 100 x 6 mm deep.

Curing and transporting

The blocks should be cured for 7 days and dried for few hours before transporting to avoid excessive breakage.

9.5.2 Control on strength of blocks

Each block-making Center of appreciable size should have a power-operated compression-testing machine. A small Center may have a hand-operated machine. The machine should be of good quality make, such as AIMIL.

It may be specified that from the daily output of each labor gang, 3 blocks should be selected at random which should be tested after 7 days curing under compression testing machine. For standardization purpose, once a week, 3 additional blocks should be cast for testing after 28 days of curing. The record of testing should be maintained in bound registers at each site and each entry to be signed by the engineering staff of executing and supervising agencies.

9.5.3 Recording test results

The rest results obtained in accordance with the above guidelines are to be recorded in appropriate tables. The test results and acceptability should be written with proper signatures. For example, the block test results may be recorded in tables with the headings shown in Table 9.1.

Table 9.1. - Test Results of Concrete Blocks (or Concrete Cubes)

SN	Name of Gang making Blocks	Date of casting	Date of Testing	Comp. Strength, kN			Average strength kN	Strength N/mn^2 or kg/cm^2	Remarks (Signatures
				Block 1	Block 2	Block 3			

For the average of 3 blocks comprehensive strength should be expressed in N/mm^2 or in kg/cm^2 or Mpa, signature of technician of the builder and verification by construction supervisor or J. E. in-charge should be recorded against each test entry.

9.6 QUALITY OF BRICK WALL

9.6.1 Quality of bricks

The bricks should be well burnt with red color, neither under-burnt not over-burnt, having a minimum compressive strength of 5.0 N/mm^2 (50 kg/cm^2), when tested flat. During testing the frog may be filled with a mortar 1:4 and the flat surfaces smoothened either by grinding, rubbing with carborandom stone or by applying a thin layer of plaster. The bricks should give a ringing sound when struck with each other.

9.6.2 Bonding in brick work

In normal construction, English bond is used in brick work in India as shown in Fig 9.25 for one brick thick walls as normally used in one to two storeys constructed using cement mortar. The 'bond' used in brick columns of size 1 x 1 up to 2 *x 2* bricks is also shown in the Fig 9.25. This will ensure that the vertical mortar joints will be broken in every two consecutive courses.

9.6.3 Brick laying

Bricks being porous absorb water. It is therefore essential that the bricks are soaked in water fully before laying on the cement mortar layer. Unsoaked bricks will suck water from the mortar and create hindrance in the setting of cement mortar. For achieving full strength of brickwork it is necessary that all vertical joints between the bricks must be fully filled with mortar. One defect in brickwork commonly seen at the sites is that the longitudinal joint between two bricks is not filled and left open. Another defect seen is that the bricks are laid upside down, that is, the frog is on the under side. This does not allow development of proper shear key between the brick courses since the frog remains unfilled. These defects should not be allowed. To ensure complete filling of all vertical joints it may be necessary to fill the joints with mortar grout before starting the next course.

If these precautions are taken in construction and proper curing of the brickwork is carried out for a minimum period of 7 days, full strength of the brickwork under vertical as well as lateral loading due to wind or earthquake will be fully achieved. For proper curing of the walls reference may be made to section 9.4.7 above.

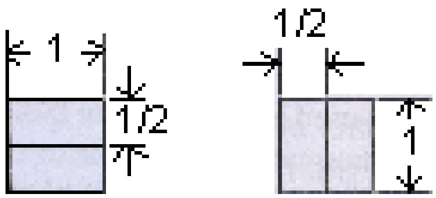


Figure 9.25 a: 1 X 1 Brick column

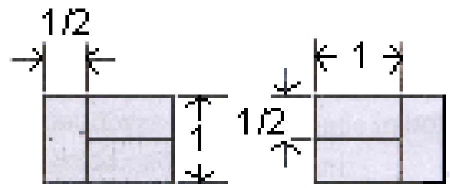


Figure 9.25 b: 1 X 1 ½ Brick column

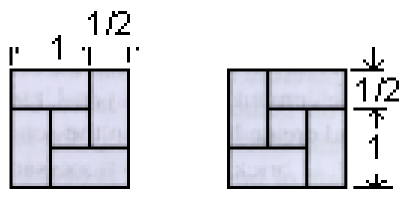


Figure 9.25 c: 1 ½ X 1 ½ Brick column

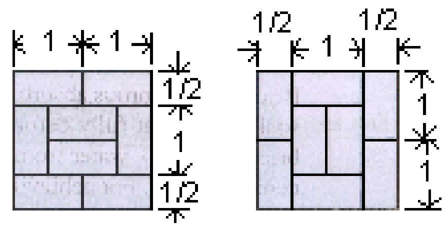


Figure 9.25 d: 2 X 2 Brick column

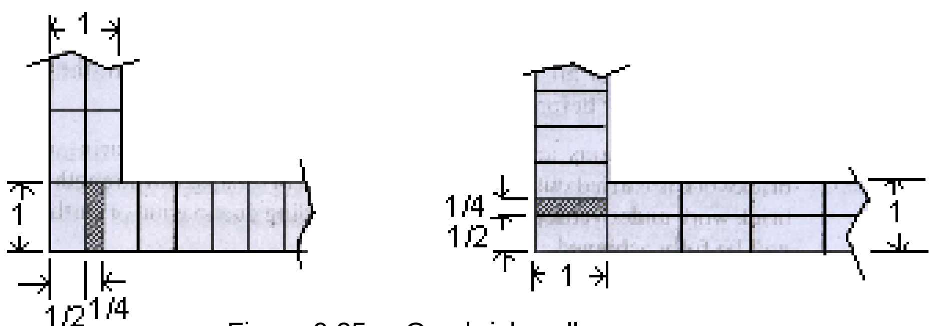


Figure 9.25 e: One brick wall

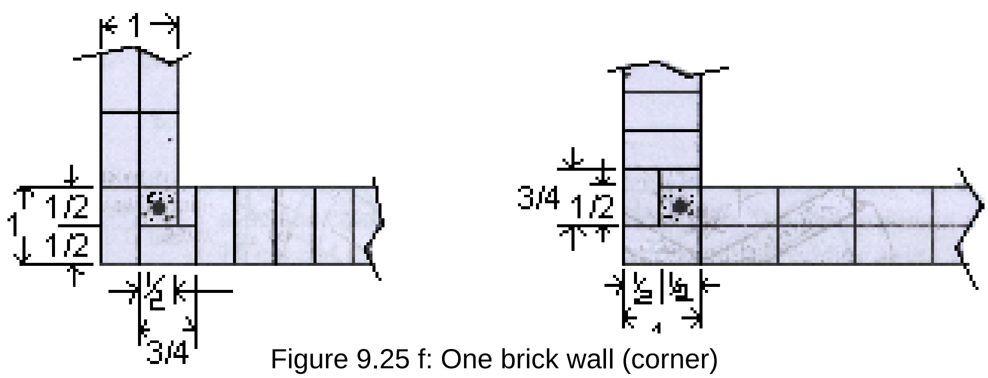


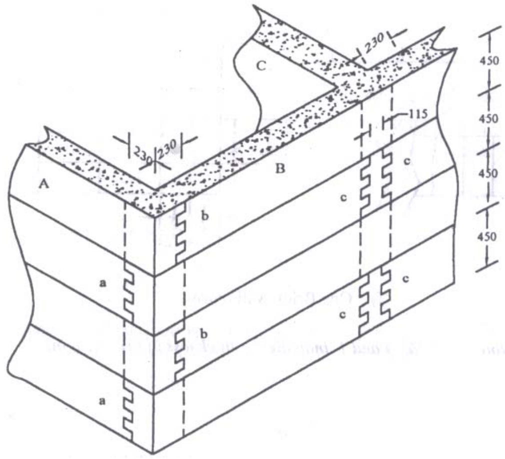
Figure 9.25 f: One brick wall (corner)

Note: ½, ¼, ¾ and 1 indicate the thickness in brick lengths

Figure 9.25: Bonding in brick walls and column forming pocket for vertical bar

9.6.4 Vertical joint between perpendicular walls

For convenience of constructions, builders prefer to make a toothed joint between perpendicular walls, which is many times left hollow and weak.



a, b. c toothed joint in walls A, B and C

Figure 9.26:
Alternating toothed joints in walls at corner and T – Junction

To obtain full bond it is necessary to make a sloping, (that is stepped joint by making the corners first to a height of 600 mm and then building the wall in between them. Otherwise, the toothed joint should be made in both the walls alternately in lifts of about 450 mm (Fig.9.26).

9.7 BENDING AND PLACING OF REINFORCEMENT

The following care must be taken for achieving high strength of the RCC and durability of the bar by avoiding/minimizing of corrosion:

a. The bars should be straight, not crooked, cut to required sizes and bent to proper shapes as per drawings.
b. The bars for the seismic bands should have a minimum cover of 25 mm below and above them. The concrete mix should be M 20 (1:11/ 2 :3 nominal) to prevent corrosion.
c. To keep the vertical reinforcing bars at the corners and joint properly vertical, an L-bend should be provided at its bottom end and each bar should be held by a tripod of bamboos or other spare reinforcing bars till such time that the concrete filled in the pocket around the bar is fully set and capable of holding the bar in vertical position.
d. A minimum overlap of 600 mm for 12-mm bars, 500 mm for 10 mm and 400 mm for 8-mm dia should be provided.
e. The cover to any bar (main or distribution) should be kept 15 mm minimum and 20 mm maximum in concrete slabs used as floor or roof. The cover in beams to the main bars should not be less than 25 mm and to the stirrups not less than 15 mm. For achieving proper cover, either cover blocks of 1:3 cement sand mortar of required thickness or PVC cover parts should be used.

9.8 REINFORCED CONCRETE LINTELS AND SLABS

9.8.1 Concrete mix

The concrete mix shall be M 20 (1:11/2:3 nominal) using cement, coarse sand and crushed grit of less than 20 mm size. The slump should not exceed 10 cm and the concrete should be compacted by rodding using 16 mm bars of about 600 mm length. Use of vibrator will, of course, be better keeping the slump of about 50 mm.

When the mix is to be designed to give the characteristic strength of 20 MPa, the target strength in the mix design should be 26.5 MPa on 150 mm cubes at the age of 28 days. For quality control on the concrete mix during construction, regular sampling and testing of concrete using 150mm cubes should be carried out and the concrete should give an average strength at 28 days of 24 MPa, the individual cube strength lying between + 15% of the mean strength obtained.

9.8.2 Slope in roofs

To prevent ponding of water on the roof and consequent leakage, the concrete roofs must be laid so as to have a minimum camber of 1/200 of span at the centre and a minimum slope of about 1 in 60. That is, for a roof width of 4m, the camber may be kept as 20 mm and the height difference between the opposite edges should be about 70 mm. It is further suggested that the roof slab be kept projecting beyond the wall with a minimum of 75 mm at the lower edge and provided with a drip course.

9.8.3 Curing of concrete

Exposed surfaces of concrete shall be kept continuously in a damp or wet condition by ponding or by covering with a layer of sacking, canvas, hessian or similar materials and kept constantly wet for at least seven days from the date of placing concrete in case of ordinary Portland Cement of 33 or 43 Grade. The period of curing shall not be less than 10 days for concrete exposed to dry and hot weather conditions. Impermeable membranes such as polyethylene sheeting covering closely the concrete surfaces may also be used to provide effective barrier against evaporation.

9.8.4 Striking formwork

In normal circumstances where ambient temperature does not fall below 15^{o}C and where ordinary Portland cement is used and adequate curing is done, the minimum period for striking formwork as given in Table 9.2 may be adopted.

Table 9.2. - Minimum Time for Striking Formwork

Type of Formwork		Minimum Period Before Striking Formwork
A	Vertical formwork to columns, walls, beams	16-24 h
B	Soffit formwork to slabs (Props to be refixed immediately after removal of form work).	3 days
C	Soffit formwork to beams (Props to be refixed immediately after removal of form work)	1 days
D	Props to slab:1) Spanning up to 4.5 m 2) Spanning over 4.5 m	7 days 14 days
E	Props to beams and arches: 1) Spanning up to 6 m. 2) Spanning over 6 m.	 14 days 21 days

9.9 CONCLUSIONS

Earthquakes in India and abroad have proved that good quality in construction and maintenance of buildings prevented their collapse. This has been fully demonstrated in recent Kachchh earthquake also where not only stone or block wall construction but also reinforced concrete frame buildings were severely destroyed even in seismic intensities of MSK VII. Hence, maintaining good construction practices and quality of materials in all buildings used for housing or community purposes is of critical importance.

CHAPTER 10

VULNERABILITY ASSESSMENT AND SEISMIC STRENGTHENING OF BUILDINGS

CHAPTER 10
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10.1 INTRODUCTION

Problem of assessment of safety of existing structures against various loads, including earthquake load, has been recognized world over. In developing countries, about 50 % of the construction industry resources are being utilized for problems associated with existing structures. Many countries have developed standards for assessment of existing structures. In India also the problem has been well recognized and the standard is under development. Performance of our structures in the recent earthquakes has also forced us to think on this issue. Many agencies, within the country, are working on the different aspects of this problem.

Assessment of an existing structure is much more difficult a task than evaluation of a design on paper. Firstly, the construction of the structure is never exactly as per designer's specifications and a number of defects and uncertainties crop up during the construction. Secondly, the quality of the material deteriorates with time and the assessment of an existing structure becomes a time dependent problem. The problem of the assessment involves not only the current status of the structure but also its extrapolation in the life of the structure with or without repairs. There are three sources of deficiencies in structures:

(1) Defects arising from the original design, such as under estimation of loads as per old standards / practices, inadequate section / reinforcement, inadequate reinforcement anchorage and detailing.
(2) Defects arising from original construction, such as under strength concrete, poor compaction, poor construction joints, improper placing of reinforcement and honeycombing.
(3) Deterioration since the completion of the construction due to reinforcement corrosion, alkali – aggregate reaction, etc.

In Indian conditions, it is generally a combination of all the three deficiencies and the retrofitting of the structure has to take care of all the three.

If the design documents are available, the first type of deficiencies can be assessed with a satisfactory level of confidence. However, if the design details are not available, it makes the task of assessment, next to impossible. Till date, no testing technique with sufficient reliability is available to completely outline the reinforcement detailing inside the concrete.

A number of techniques have been developed to detect the other two types of deficiencies. However, almost all of them depend on indirect measurements and have a low reliability. Further, the variation of test results is large and interpretation of results requires experience and skill. This chapter gives a brief account of different techniques available for assessment of structures and in-situ properties of concrete.

10.2 VULNERABILITY ASSESSMENT OF BUILDINGS

According to the Vulnerability Atlas of the country, more than 80 % houses are non – engineered construction, which are mainly load bearing masonry buildings in rubble or coursed stone or brick masonry in mud or cement mortar. However, there are many RC framed urban buildings which have been constructed without any consideration to resist earthquake forces or without using the current codal practices on Earthquake Resistant Design. For such a large number of seismically deficient buildings, a quick assessment method and guidelines have to be developed together with training and capacity building. To handle the mammoth task of seismic evaluation of existing buildings, three levels or Tiers have been suggested.

10.2.1 Rapid Visual Screening (RVS) Procedure (Level – 1, Procedure)

In a city there is very large number of existing buildings, which need to be examined for assessing the Seismic vulnerability of the city and for making policies for mitigation and management of seismic risk. For this purpose two approaches have been developed: (i) Based on Indexes / scores assigned by trained surveyors after visual inspection of the buildings, and (ii) Checklist method, based on the basic structural and earthquake resistant features of the building. Based on the behavior of buildings in the past earthquakes scores or types / classes of buildings have been assigned. These classes are consistent with the MSK or European Intensity scales.

For this screening a team of at least two surveyors visits the building and try to collect the information in a specially designed format. One of such formats is given in **Annexure II**. Once the class or the score of the building is decided, the expected behavior of the building or the expected damage during a future earthquake can be assessed. These methods are based on behavior of the buildings during past earthquakes and have significant subjective component. The results of the evaluation depend to a large extent of the training and skill of the surveyors. The purpose of this screening is to identify the buildings which require further investigation using Level – 2 or Level – 3, procedure.

Rapid visual screening of buildings is also required after a major earthquake which results in large scale damage of existing buildings. The purpose of such a screening is to identify buildings which are severely damaged and should be evaluated. Equally important aim of such a survey is to identify the buildings which are safe and can be used as shelters after the earthquake. **Annexure III** gives a checklist for survey of earthquake damaged buildings.

10.2.2 Simplified Vulnerability Assessment (SVA) Procedure (Level – 2, Procedure)

The buildings which have identified as vulnerable in the Level – 1 procedure need to be investigated further. The Level – 2 procedure involves a more systematic inspection and a limited engineering analysis based on the available structural drawings or on site measurements. Simplified calculations are made for strength and drift of the building based on sizes and strength of critical members. **Annexure IV**, gives a typical format for Level – 2 inspection of existing buildings. The drift of the building has a good correlation with the damage of the building.

This method is more complex than the Level -1 procedure and requires a qualified structural engineer, well experienced in earthquake resistant design of buildings. If should be emphasized that this procedure can be used only for normal and regular types of buildings. For irregular buildings and for buildings with abnormal structural configurations Level -3 evaluation should be used.

10.2.3 Detailed Vulnerability Assessment (DVA) Procedure (Level – 3, Procedure)

The detailed vulnerability assessment is used for those buildings, which are found vulnerable from Level – 2 procedure, for buildings with abnormal or irregular structural configurations and for monumental or important buildings. This procedure is normally more complex than the design of a new building and requires comprehensive engineering analysis considering the expected earthquake motion and in – situ strength of materials.

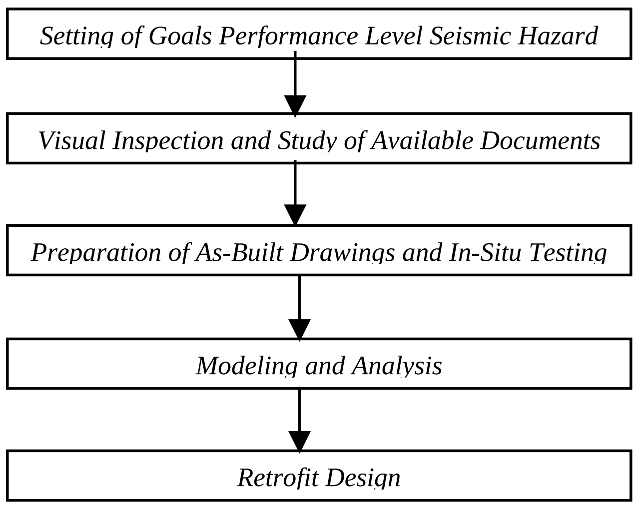


Figure 10.1: Detailed Vulnerability Assessment

The different steps in the detailed vulnerability assessment of a building are shown in Fig. 10.1 and are described in the following sections.

10.3 FIXING OF GOALS

One important question in seismic strengthening / retrofitting of the buildings is “How much retrofitting?” or what is the level of performance expected from the building after retrofitting. Building performance level requirement depends on the usage and importance of Buildings.

Performance level of a building depends on both structural and non – structural components. FEMA – 273 gives a detailed description of different performance levels for structural and non structural components. Here, four overall performance levels, which are compatible with the philosophy of Indian Code of Practice, are described.

10.3.1 Operational Performance Level

Some of the post earthquake importance buildings, such as major hospitals are expected to be fully operational after an earthquake. In these buildings, not only the safety of structural and non – structural components should be ensured, but the smooth functioning of services should also be ensured. The maximum drift should be within tolerance limits of services. There should be no permanent drift and structure should retain its original strength and stiffness. For this performance level, only minor cracking of facades, partitions, and ceiling etc. is acceptable. For operational performance level the building should have standby power and water supply.

10.3.2 Immediate Occupancy Performance Level

Post earthquake importance buildings, which are expected to provide shelter to earthquake victims, are required to have immediate occupancy performance level. These buildings should have minimal or no damage to structural components and only minor damage to non – structural components. The building should have no permanent drift, and only minor cracking of facades, partitions, ceiling and structural elements is acceptable. Elevators and fire protection system should be working after earthquake. However, equipment and component may not work due to mechanical failure and lack of utilities such as water and power supply.

In such buildings, immediate occupancy of the building after earthquake is possible, but some minor repair, restoration of supplies and cleanup may be necessary before normal usage of the building. All the buildings designed as per IS – 1893 are expected to have this performance level for a minor or moderate earthquake. However, only some important buildings such as school and hospital buildings are expected to have immediate occupancy performance level even for a major earthquake.

10.3.3 Life Safety Performance Level

In normal buildings, the damage to structural and non – structural components, after a major earthquake, is extensive but the risk to life should be low. The damage level may sometimes be so extensive that the repair may be uneconomical. The structures have some residual strength left at all the storeys. The non-structural parts should not fall and pose risk to life but these may be extensively damaged.

10.3.4 Collapse Prevention Performance Level

In no case a building should be allowed to collapse after an earthquake, as collapse will result in severe risk to life. This performance level is related to only structural components without any consideration to non – structural components. The building will have very little residual lateral stiffness and columns should function against gravity action, even in damaged state. The building may have large permanent drifts, some of the exits may be blocked, in fills and parapets may fail and building may be in near collapse state. In a severe earthquake most of the buildings with this performance level may result in complete economic loss. This is the mandatory seismic performance level to be ensured in all the buildings. All the buildings designed as per Indian codes are supposed to have at least this performance level in a severe earthquake.

10.4 HAZARD ASSESSMENT

The first step in detailed vulnerability assessment of an important structure is to estimate the likely intensity of the earthquake at the site. Earthquake intensity at a site can be estimated from the seismic zoning map of India. For better estimation site specific earthquake intensity studies are carried out. Seismic microzonation of major cities of India is on cards. Once the seismic microzonation maps are available more accurate estimation of earthquake intensities will be possible.
Levels of earthquakes which should be considered in the evaluation of an important structure are defined as:

(i) **Serviceability Earthquake:** This is the level of ground shaking which has 50 % chance of being exceeded in the 50 years period (normal life time of a structure). This has a mean return period of 75 years. This is typically 0.5 times the level of ground shaking corresponding to Design Earthquake.

(ii) **Design Earthquake**: This level of ground shaking has a 10 % chance of being exceeded in 50 years, which corresponds to a return period of approximately 500 years. This is the same level of ground shaking, defined as Design Basis Earthquake (DBE) by IS 1893: 2002. This represents an infrequent earthquake, which can occur during the life time of structure.

(iii) **Maximum Earthquake**: This is the maximum expected level of ground shaking at the site. This has a 5 % probability of being exceeded in 50 years, which corresponds to a return period of about 1000 years. This level of shaking is about 1.25 to 1.5 times the level of shaking corresponding to Design Basis Earthquake.

IS 1893 defines another level of ground shaking termed as **Maximum Considered Earthquake (MCE)**. Although this term has not been defined by the IS 1893 in terms of probability of exceedance or return period, but the same term has been used and defined in UBC. This is much higher level of ground shaking, which has a return period of about 2,500 years. This corresponds to the upper bound on the expected ground shaking depending on the geological conditions at site. This has only 2 % probability of exceeding in 50 years. This level of level of ground shaking is typically 2 times the ground shaking corresponding to the Design Basis earthquake.

The hazard assessment at a site consists of followings steps:
1. Identification and characterization of all potential earthquake sources, i. e. active faults within the influence zone of the site (normally within a radius of 200 km)
2. Assigning magnitudes to the identified sources, based on source characteristics and past records,
3. Identification of predictive (attenuation) relationships applicable to the area.
4. Consideration of local site effects (soil amplifications, ridge effect and basin effect).
5. Estimation of ground motion parameters and development of design response spectrum.

Vulnerability of building has to be assessed with respect to ground failure hazard, also. The ground failure hazard consists of soil liquefaction, proximity to slope failure / rock fall areas and proximately to surface fault rupture. These can be estimated by considering the expected level of shaking along with the local site conditions.

10.5 VISUAL INSPECTION AND STUDY OF AVAILABLE DOCUMENTS

A systematic visual inspection provides a fair idea about the irregularities in building configuration, construction, consideration defects and deterioration / distress of the structure. Extensive photography of the building is helpful in the study of the building in office. Visual inspection is helpful in deciding the extent of investigation and selection of tests. Visual inspection should start from the roof, which gives the best view of the building plan and configuration. It should concentrate on the irregularities in configuration, construction defects and most importantly, the signs of distress and deterioration. Different distress agents have characteristic cracking patterns and a close inspection of crack patterns may provide a good idea of the cause of distress. Corrosion of reinforcing steel is the most common cause of deterioration in RC buildings and can be easily identified by characteristic cracking pattern parallel to reinforcement and occasional spalling of cover concrete.

Sketches showing the general plan and elevations of building and cracking patterns / crack locations are helpful in later reference in office. Removal of cover at some selected locations may be helpful in identifying extent of corrosion taken place in the building. In masonry buildings, vertical slits may be cut through the plaster to see whether earthquake bends have been provided.

In case of slopping and jack – arch roof buildings, false ceiling should be removed at a few locations to have a view of the trusses / girders. In the absence of proper maintenance, the girders and trusses corrode and should be properly investigated. In case of wooden trusses, the joints should be carefully examined and wooden members should be examined for attack of insects and rottening.

Pounding of adjacent buildings has been observed to cause damage in the past several earthquakes. Therefore attention should also be given to adjacent buildings and the gap between the adjacent buildings. If separation joints have been provided within the building, these should be carefully observed. As these are a maintenance problem, it has been observed that in most of the cases the gap in the separation joints is filled and the desired action is not available during the earthquake.

Maintenance of building records in India is very poor. Generally, the structural drawings are either not available or these are incomplete and in poor condition Attempt should be made to get as much information as possible about the original design, construction, repairs and extensions of the building. Any change in usage of the building should also be recorded.

10.6 DETAILED IN – SITU INVESTIGATION

Visual inspection and preliminary evaluation of a building provides some insight into the potential design and construction deficiencies and causes of deterioration. A detailed investigation is required to estimate the in – situ strength of material and extent of deterioration. A number of testing methods have been developed for estimating in – situ strength of RC. Some of these techniques have also been used for masonry.

10.6.1 Planning and Interpretation of Results

In-situ testing of structures is a costly and time consuming affair. Lot of money and time can be saved by proper planning of the testing program. The visual inspection should be done in a systematic manner and extensive photography or video taping of the structure should be undertaken prior to testing and retrofitting.

Depending on the aim of testing and funds available the optimum number of various tests is to be decided. The number and type of test have to be decided keeping in view the reliability of the test and the accuracy desired.

Results of in-situ testing methods need to be corrected for a number of parameters depending on the testing conditions. There are some tests which have opposite corrections in similar conditions. Combination of such methods can be used for higher accuracy. One such combination is Rebound hammer test and UPV test.

10.6.2 Foundation Capability

Structures have to be assessed for their performance, settlement, depth of foundation, deterioration due to weathering or age, capacity of foundation, stability against overturning, ties between foundation elements, load path for transfer of seismic forces to soil and special requirements in sloping sites.

10.6.3 Non-structural components

Parapets, sunshades, projections, fixtures, cladding, etc. have to be assessed for their capacity to withstand earthquake forces. Safety of non-structural components is particularly important in case of buildings such as hospitals, telephone exchanges, control buildings, etc. The failure of fixtures and connections may lead to not only the disruption of the function but also the loss of life due to disruption as well as due to direct injury from the falling component.

Partitions and infills are another component, which are usually considered as non-structural in the design. Their safety is not ensured in design. Failure of masonry infills in out of plane bending may be fatal to the inmates. Safety of partitions and infills must be ensured by the retrofitting engineer.

10.7 MODELLING AND ANALYSIS

A lot of research has taken place in the area of analysis of buildings for earthquake forces. The analysis methods can be broadly classified into Linear and Non-linear methods. Earthquake resistance design relies heavily on the ductility or post yielding behavior of the structure and therefore, the non-linear methods appear to be more reliable. However, these methods also have inherent assumptions and require skill and computer software, as these are computationally intensive. Another classification is based on the type of load considered in the analysis. Static analysis procedures consider equivalent static force, while the dynamic analysis procedures take into account the time varying nature of the earthquake forces. The dynamic analysis is nearer to reality but require high degree of computation. On the other hand, the static analysis procedure is simple, easy to use and provide insight in to behavior of structure. For normal regular buildings, a linear procedure is considered to be sufficient, but for important buildings and buildings with irregular configurations, a static or dynamic non-linear analysis is necessary.

10.7.1 Mathematical Modeling of Buildings

Development of a mathematical model of the building structure is the first step in its analysis. Depending on the torsional effects in the building, either 2D or 3D modelling of the building may be used. If the maximum horizontal displacement of a point on a floor is more than 120% of the average displacement of the floor, the building is considered to have a torsional irregularity and a 3D space frame model is to be used. For calculating the torsional displacement, both actual and accidental torsions are to be considered.

A number of mathematical models are available with varying degrees of sophistication in the analysis. Earlier research was centered on developing hand calculation methods based on simplified assumptions and understanding of the overall behavior of the structure. Nowadays, the computer hardware and software for analysis of structures is widely available and stress is on more sophisticated mathematical modelling.

The actual structure and its behavior at the micro level is always very complex. It is not possible to model each and every detail of the structure, what so ever being the sophistication of the computer software. The mathematical modelling of the structure is based on certain simplifying assumptions and the understanding of the overall behavior of the structure. Therefore, caution is required to interpret the output of the computer software and the user should have a sound knowledge of the behavior of the structure.

1. Linear Static Procedure (LSP) and Linear Dynamic Procedure (LDP) of Analysis

The LSP for evaluating an existing building is the same as described in IS-1893: 2002 for design of a new building with a predefined distribution of earthquake forces along height of the building.

In LDP also the modelling of the structure is same as in LSP, but in spite of using a predefined distribution of earthquake forces along the height of the building, the distribution is obtained by dynamic analysis of the building. The dynamic analysis can be performed either using a response spectrum (Modal Analysis) or using time histories of earthquake motion (Time History Analysis).

2. Non-linear Static Procedure (NSP) of Analysis

This recently developed method is a revolutionary idea to have a non-linear analysis based on response spectrum method. The method gives an iterative solution for the maximum non-linear displacement of the building. This non-linear displacement is checked for each component to determine its safety and damage state. Different performance levels put different restrictions on the maximum non-linear displacement components.

3. Non-linear Dynamic Procedure (NDP)

NDP is the well-known Non-linear Time History Analysis based on step-by-step solution of the equation of motion. This method simulates the real behavior of a structure during an earthquake and can be used, at least theoretically, to analyze any structure. The main difficulty in use of this procedure is that it requires design time histories, which are difficult to be specified, as the codes specify only design response spectrum. Further, it is computationally very extensive method.

Both NSP and NDP require a comprehensive understanding of building components, their interconnections and their material properties. It is difficult to estimate realistically the non-linear load-deformation relationships for building components and therefore, the practical benefit of NSP and NDP is doubtful. It is warned that in absence of thorough understanding of the building components behavior, NSP and NDP should not be used.

10.8 SEISMIC STRENGTHENING OF BUILDINGS

After the vulnerability assessment of any existing building, the decision as to whether the building needs to be strengthened and to what degree, must be based on assessment that show if the levels of safety demanded by present codes and recommendations are met. As we have seen, difficulties in establishing actual strength arise from the considerable uncertainties related with material properties and with the amount of strength deterioration due to age or to damage suffered from previous earthquakes. The method of repair and strengthening would naturally depend very largely on the structural scheme and materials used for the construction of the building in the first instance, the technology that is feasible to adopt quickly and the amount of funds that can be assigned to the task which are usually very limited. The concepts of repair, restoration and strengthening are described in the following sections.

10.8.1 Repairs

The main purpose of repairs is to bring back the architectural shape of the building so that all services start working and the functioning of building is resumed quickly. Repair does not pretend to improve the structural strength of the building and can be very deceptive for meeting the strength requirements of the next earthquake. The actions will include the following:

1. Patching up of defects such as cracks and fall of plaster.
2. Repair doors, windows, replacement of glass panes.
3. Checking and repairing electrical wiring.
4. Checking and repairing gas pipes, water pipes and plumbing services.
5. Rebuilding non – structural walls, smoke chimneys, boundary walls etc.
6. Re-plastering of walls as required.
7. Rearranging disturbed roofing tiles.
8. Relaying cracked floor at ground level.
9. Redecoration – white washing, painting etc.

The architectural repairs as stated above do not restore the original structural strength of cracked walls or columns and may sometimes be very illusive, since the redecorated building will hide all the weaknesses and the building will suffer even more severe damage if shaken again by an equal shock since the original energy absorbing capacity will not be available.

10.8.2 Restoration

It is the restitution of the strength the building had before the damage occurred. This type of action must be undertaken when there is evidence that the structural damage can be attributed to exceptional phenomena that are not likely to happen again and that the original strength provides an adequate level of safety.

The main purpose of restoration is to carry out structural repairs to load bearing elements. It may involve cutting portions of the elements and rebuilding them or adding more structural material so that the original strength is more or less restored. The process may involve inserting temporary supports, underpinning etc. Some of the approaches are stated below.

1. Removal of portions of cracked masonry walls and piers and rebuilding them in richer mortar. Use of non – shrinking mortar will be preferable.
2. Addition of reinforcing mesh on both faces of the cracked wall, holding it to the wall through spikes or bolts and then covering it suitably. Several alternatives have been used for this purpose so far.
3. Injecting epoxy like material, which is strong in tension, into the cracks in walls, columns, beams etc.

Where structural repairs are considered necessary, these should be carried out prior to or simultaneously with the architectural repairs so that total planning of work could be done in coordinated manner and wastage is avoided.

10.8.3 Strengthening of Existing Buildings

The seismic behavior of old existing buildings is affected by their original structural inadequacies, material degradation due to time, and alterations carried out during use over the years such as making new openings, addition of new parts inducing unsymmetry in plan and elevation etc.
The possibility of substituting them with new earthquake resistant buildings is generally neglected due to historical, artistic, social and economical reasons. The complete replacement of the buildings in a given area will also lead to destroying a number of social and human links. Therefore seismic strengthening of existing damaged or undamaged buildings can be a definite requirement in same areas. Strengthening is an improvement over the original strength where the evaluation of building indicates that the strength available before the damage was insufficient and restoration alone will not be adequate in future quakes.
The extent of the modifications must be determined by the general principals and design methods and should not be limited to increasing the strength of members that have been damaged, but should consider the overall behavior of the structure. Commonly the strengthening procedures should aim at one or more of the following objectives:

1. Increasing the lateral strength in one or both directions, by reinforcement or by increasing wall areas or the number of walls and columns.
2. Giving unity to the structure by providing a proper connection between its resisting elements, in such a way that inertia forces generated by the vibration of the building can be transmitted to the members that have the ability to resist them. Typical important aspects are the connections between roofs and floors and walls, between intersecting walls and between walls and foundations.
3. Eliminating features that are sources of weakness or that produce concentrations of stresses in some members. Asymmetrical plan distribution of resisting members, abrupt changes of stiffness from one floor the other, concentration of large masses, large openings in walls without a proper peripheral reinforcement are examples of defect of this kind.
4. Avoiding the possibility of brittle modes of failure by proper reinforcement and connection of resisting members. Since its cost may go to as high as 50 to 60 % of the cost of rebuilding, the justification of such strengthening must be fully considered.

10.9 STRENGTHENING MATERIALS

Considerable research has taken place in the field of repair and retrofitting materials and a large variety suitable to different applications and working conditions is available. Most of the materials are patented and available in brand names. We need to have information about these materials for designing the retrofit scheme.
The repair and retrofit materials can be broadly classified into three categories:

(i) Grouts for repair of cracks, strengthening of masonry and honeycombed concrete.
(ii) Bonding agents for enhanced bonding between old and new concrete and concrete and reinforcement.
(iii) Replacement and jacketing materials for replacing the damaged portions, increasing the size of members, enhancing the confinement and external reinforcement of the members.

A brief description of different materials available under these categories is given below.

10.9.1 Injection Grouts

Grout is a flowable plastic material, which can be injected into a structural member under pressure. The grout should have negligible shrinkage to fill the gap/void completely and it should remain stable without cracking, delamination or crumbling.
Injection grouts are used to fill interior space within the concrete or masonry created due to cracks, voids or honeycombs. In case of damaged concrete or masonry if the cracks are thin, these can be repaired by injection grouting, otherwise, if the cracks are wide, the material around the cracks is to be removed and replaced by new material. Injection grouts can also be used for strengthening of old masonry structures, in which mortar has degraded and in honeycombed concrete. These are particularly useful in strengthening of monumental structures, but compatibility of original material and the grout must be ensured.

Before injecting grouts into crack, preparation of the crack is to be done as following:
(i) Cleaning of crack with compressed air and removal of loose material, if any.
(ii) Drilling of holes (5 to 10 mm) at several places along the length of the crack.
(iii) Placing of 'Ports' or 'Nipples' at the mouth of holes. If the cracks are wide and accessible from surface 'T' ports can be installed.

After injecting resins through the ports, the cracks and ports are sealed by quick hardening resin paste.

On vertical surfaces, the injection is started from the lowest port till it comes out from the upper nipple. Then the port is sealed and injection is started from the upper port. After hardening of the epoxy in a day or so, the sealing resin paste is removed. The effectiveness of injection grouting in concrete can be tested either by USPV test or by visual inspection of cores drilled through the injected crack.

10.9.2 Bonding Agents

Bond between existing concrete, new concrete and reinforcement is very important for effectiveness of repair/retrofitting. There are three methods available for enhancing the bond:

(i) Application of adhesives at the interface
(ii) Surface interlocking
(iii) Mechanical bonding

Polymers and epoxy are the adhesives used for bonding between old and new concrete and reinforcement. After removal of the concrete cover, the existing concrete surface and steel are cleaned by sand or water blasting. After cleaning and drying, concrete and steel is painted by epoxy/polymer or polymer modified cement grout. If the new steel is to be welded, it is welded prior to coating of the concrete and steel. This coating provides enhanced bond between the old and new material and reduces the risk of corrosion in steel.
To improve the surface interlocking, the existing concrete surface is coated with epoxy/polymer and a layer of coarse sand is applied above the coating. Mechanical bonding consists of keys and anchors provided in the existing members at regular interval.

10.9.3 Replacement and Jacketing Materials

In case of damaged structures, material in some parts of members is to be replaced by new material. For strengthening existing members in deficient buildings, additional material including reinforcement is to be provided. The material used for replacement should have good bond with existing material and it should be non-shrinking. A variety of strengthening and replacement materials is available.

Ordinary Portland Cement Concrete and Mortar

The advantage of using ordinary concrete and mortar is that these have similar thermal movement and appearance as the existing concrete. Further, these are cheap and do not require special skills for application. Generally, these consist of high early strength cement and an expansive component to compensate the shrinkage. The expansive component also results in good bond. The common expansive agents used are aluminum powder, coke powder, anhydrous calcium sulfoaluminate and calcium oxide.

In case of concrete, use of higher strength (at least by 5 MPa) then the existing concrete is recommended. Maximum size of coarse aggregate is limited to 20 mm for ease in pouring the concrete through narrow spaces. To ease the compaction, workability is enhanced by using super plasticizers. The surface of existing concrete is made as rough as possible and cleaned properly. After placing the forms a final dusting should be done using compressed air to remove dust from the surface.

Sometimes a special application of ordinary concrete ‘preplaced concrete’ is also used. In this method, the aggregate is first packed in the space to be concreted and the cement is applied in the form of grout intrusion. The concrete has very little shrinkage but requires skill in application.

Dry pack is another application method of ordinary concrete. In this method the concrete has very little water and has almost zero slump. The moisture is just sufficient to stick the material together when molded into a ball by hand. The low water content results in reduced shrinkage, but makes compaction difficult and there are chances of voids being left.

Dry packs are available under several commercial names and usually consist of fine sand, super plasticizers and an expansive agent in appropriate proportion. This mixed with water attains very high strength in very short time. This high strength is a result of formation of a special silica calcium hydrate from the reaction of the cement with expansive agent. The expansive agent also result in no – shrinkage of the material. This material is very suitable for jacketing.

Shotcrete

Shotcrete or guniting has the same characteristics as ordinary concrete but it has smaller aggregate size and it is applied under pressure with low water content. It requires no framework and can be applied on any surface including inclined and vertical surfaces and even on ceilings. This results in very good adhesion between old and new concrete and good compaction due to application under pressure. The low water cement ratio results in high strength and low shrinkage. The permeability of shotcrete is also lower than that of ordinary concrete and results in better protection of steel against corrosion.

Shotcrete requires special equipment. Two types of equipment are used depending on dry or wet mix type of application. In dry mix application, the proportioned or pre-pakaged cement aggregate mixture is transferred to nozzle using highly compressed air. Water is introduced at nozzle under pressure. The mixture is impacted on the surface to be shotcreted. In wet mix type shotcrete, proportioned mixture of cement aggregate water and admixtures are discharge into a conventional concrete pump through a discharge nozzle. Compressed air is used to project the material from nozzle.

Before application of shotcrete, damaged concrete is removed and the surface is thoroughly cleaned by sand blasting to remove all dirt and to expose the aggregate. Steel is cleaned on full circumference of bar to bare metal. Usually a melded wire mesh is applied over the surface to be shotcreted and attached to the existing concrete through nailing. This wire mesh reduces the shrinkage and improves the bond between existing concrete and shotcrete. Sometimes, to improve the bond between old and new material, surface coatings, such as epoxy bonding agents, latex modified cement slurries or neat cement slurries are also used.

In case of dry mix shotcrete, the water / cement ratio cannot be controlled quantitatively as it is mixed at nozzle and controlled visually by the operator. Therefore, the skill of the crew is very important. The variation in density of shortcrete is more than that of normal concrete. Also, the shortcrete results in a rough surface.

Polymer Modified Concrete and Mortar

Polymers are long molecule hydrocarbons, built by combination of single units called monomers. The process is called polymerization. Small diameter particles of polymers emulsified in water are called polymer latexes. These latexes form continuous film at drying. Adding polymer latexes to ordinary mortar and concrete is the most common method of making Polymer Modified Mortar (PMM) and Polymer Modified Concrete (PMC). Cement hydration in PMM/PMC results in drying of latex and formation of the film of polymers. This film binds the cement hydrates together to from a monolithic network in which the polymer phase interpenetrates throughout the cement hydrate phase. The resulting matrix binds the aggregate more strongly and enhances the properties of mortar/concrete.

The polymer can also be mixed in the form of re-dispersible powder in the dry cement-aggregate mix. When water is added to this mixture, a process similar to that described above takes place. Some polymers are water soluble. When added to mortar/concrete, these result in enhanced workability but no increase in strength. In some liquid thermosetting resins, polymerization is initiated by water. These are also added to concrete/ mortar to result in enhancement similar to that resulting from latex.

The PMM/PMC has better workability and water retention properties than ordinary concrete/mortar. This reduces the requirement of water curing considerably. Polymer modification does not result in any appreciable increase in compressive strength of concrete, but it results significant increase in tensile and bending strength of concrete.

The main advantage of PMM/PMC is its improved adhesion and bond with existing concrete and significantly reduced permeability. Reduced permeability results in reduced risk of corrosion of reinforcing steel.

Steel Plate Bonding

Steel plates can be bonded to concrete members as external reinforcement to increase their strength. The plates are glued to the member surface by epoxies. This requires a careful preparation of the member surface and application of epoxy layer. Steel plates can also be provided in the form of jackets either by gluing to surface or by grouting. However, these jackets are not very effective as these try to separate out from the members due to Poisson's effect, loosing confinement.

Fibre Reinforced Plastics (FRP)

Fibre-reinforced polymers/plastics is a recently developed material for strengthening of RC and masonry structure. This is an advanced material and most of the development in its application in structural retrofitting has taken place in the last two decades. It has been found to be a replacement of steel plate bonding and is very effective in strengthening of columns by exterior wrapping. The main advantage of FRP is its high strength to weight ratio and high corrosion resistance. FRP plates can be 2 to 10 times stronger than steel plates, while their weight is just 20% of that of steel. However, at present, their cost is high.

FRP composites are formed by embedding a continuous fibre matrix in a resin matrix. The resin matrix binds the fibre together and also provides bond between concrete and FRP. The commonly used fibres are Carbon fibres, Glass fibres and Aramid fibres, and the commonly used resins are polyester, vinyl ester and epoxy. FRP is named after the fibre used, e.g. Carbon Fibre Reinforced Polymer (CFRP), Glass Fibre Reinforced Polymer (GFRP), and Aramid Fibre Reinforced Polymer (AFRP).

The fibres are available in two forms (i) Unidirectional tow sheet, and (ii) Woven fabric. The application of resin can be in-situ or in the form of prefabrication of FRP plates and other shapes by pultrusion. The in-situ application is by wet lay-up of a woven fabric or tow plate immersed in resin. This method is more versatile as it can be used on any shape. On the other hand, prefabrication results in better quality control. The manufacturers supply these materials as a package and each brand has specific method of application, which is to be followed carefully. Specialized firms have developed in India also, which take up the complete execution work and supply of material. It is important to note the difference between the properties of steel and FRP and it should be understood that FRP cannot be treated as reinforcement in conventional RC design methods. Table 10.1 gives a typical range of properties for three types of fibres. This range may change from one brand to another and with change in fibre content.

Table 10.1 Typical Properties of GFRP, CFRP and AFRP

Unidirectional advanced composite materials	Fibre content (% by weight)	Density (kg/m^3)	E (Long.) (GPa)	Tensile strength (MPa)
Glass fibre/ polyester GFRP laminate	50-80	1600-2000	20-55	400-1800
Carbon/epoxy CFRP laminate	65-75	1600-1900	120-250	1200-2250
Aramid/epoxy AFRP laminate	60-70	1050-1250	40-125	1000-1800

Figure 10.2 shows the qualitative stress-strain curves for mild steel, CFRP, AFRP and GFRP. It can be seen that not only there is drastic difference in tensile strength and modulus of elasticity; unlike to mild steel, FRP is elastic right up to failure. This shows total lack of ductility in case of FRP. This brittleness of FRP must be considered while predicting the behavior of retrofitted members. This brittleness does not allow the redistribution of stress in RC members and therefore, the conventional design theories are not valid for FRP reinforced concrete members.

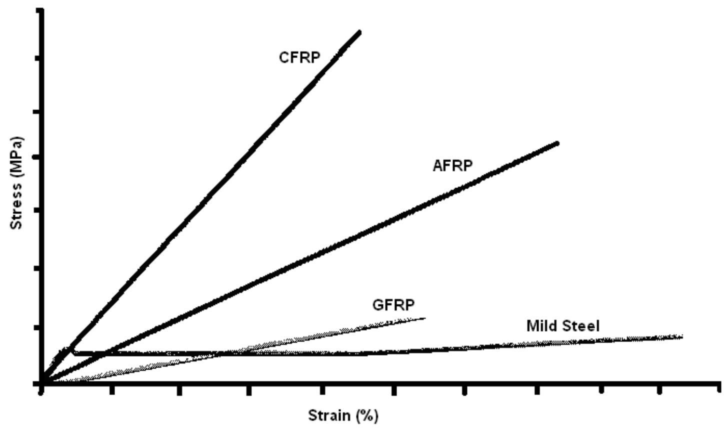


Figure 10.2: Stress – strain behavior of FRP and mild steel

10.10 RETROFITTING OF LOAD BEARING WALL BUILDINGS

Buildings in which the roof and floor slabs are directly supported on the walls are called load-bearing wall buildings. These walls serve as partitions and also bear the load from slabs. The lateral load resulting from earthquake and wind is also resisted by these walls and transferred to ground. Individual unreinforced masonry or mud walls are very weak in out-of-plane bending due to lack of tensile strength. These are generally not capable of bearing out-of-plane bending moment, even resulting from their own inertia. These walls act as shear-walls in their in-plane action and possess sufficient in-plane strength, if not weakened by too many openings. In a building, there are four or more than four walls, which act as a box under lateral load. The walls parallel to the lateral load, act as webs and the walls orthogonal to load act as flanges. The resistance of box is much higher than the resistance of individual walls. The box action involves considerable interaction between webs and flanges at corners of building. It has been observed in past earthquakes that in many cases, the damage initiates at corners, resulting in loss of box action and walls start acting independently leading to collapse of building. The basic principle of seismic safety of load bearing wall buildings lies in their integral box action during earthquake. In new buildings it can be ensured by providing seismic bands. In existing buildings, the integral box action is to be ensured by providing external bandage (Fig 10.3) or pre-stressing (Fig 10.4).

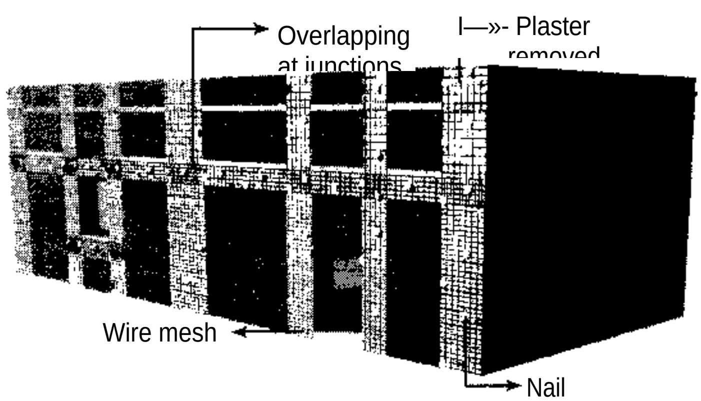


Figure 10.3: Retrofitting of load bearing buildings by bandage

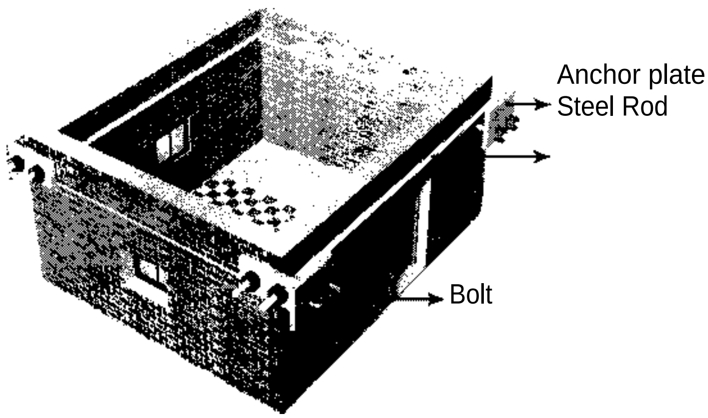


Figure 10.4: Retrofitting of load bearing buildings by pre-stressing

Openings in walls are the source of weakness. Openings result in reduction of effective cross-sectional area of wall resisting lateral loads. If the openings are very near to corners, these hamper the integral box action by weakening the joints. The piers between openings are subjected to higher stresses than the portion of the wall above and below the openings. It has been observed in the past earthquakes that diagonal X- shaped cracks in the piers originate from the corners of openings. To avoid this damage the opening perimeter need to be strengthened by proper reinforcement.

10.11 RETROFITTING OF RC BUILDINGS

The retrofitting schemes for RC buildings are based on two principles: (i) reduction in earthquake demand by reducing mass, by base-isolation or by supplemental energy dissipation, and (ii) enhancing the capacity of the structure to withstand the earthquake forces. The capacity may be enhanced either by strengthening the deficient members or by improving the ductility and deformation capacity resulting in increased hysteretic damping. There is another important aspect of retrofitting - completion of load path and removal of configurational irregularities.

10.11.1 Completion of Load Path

A large number of buildings in India have incomplete load paths mainly to take advantage of the loopholes in the building by-laws and sometimes due to market compulsions and our quest for creating new shapes. For example, floating column constructions are not uncommon in Indian cities to take maximum advantage of floor area with restrictions on ground area.
The general seismic load path in a building is as follows - the inertial forces originating throughout the building are first transferred to horizontal floor diaphragms, the diaphragms transfer these forces to vertical framing system resisting lateral loads; the vertical framing system consisting of beam-column frames and shear walls, transfers the seismic force to foundation and supporting soil. If there is a discontinuity in load path, the building is incapable of transferring the load to ground and it is unable to resist the lateral load during earthquake, irrespective of strength of existing members.
The common examples of such building are those in which shear walls or columns are not

started from ground but started at first floor (or at a higher level). Such columns are commonly known as floating columns. This is done to increase the floor area at first floor level or to have large open spaces at ground floor for commercial purposes. In such buildings, the first floor beams are subjected to very high forces as the forces from floating columns/shear walls are transferred to other columns and walls through these beams.
The remedy to this deficiency is to complete the load path by providing the missing part of the column/shear wall. In case of a floating column, a new column is to be erected below the floating column (Fig 10.5). This column should have footing connected with the foundation of the existing building and the reinforcement of the new column should be welded with the reinforcement of existing column. Shrinkage compensating agents should be used in the new concrete to avoid shortening of the new column resulting in separation between new and old concrete.

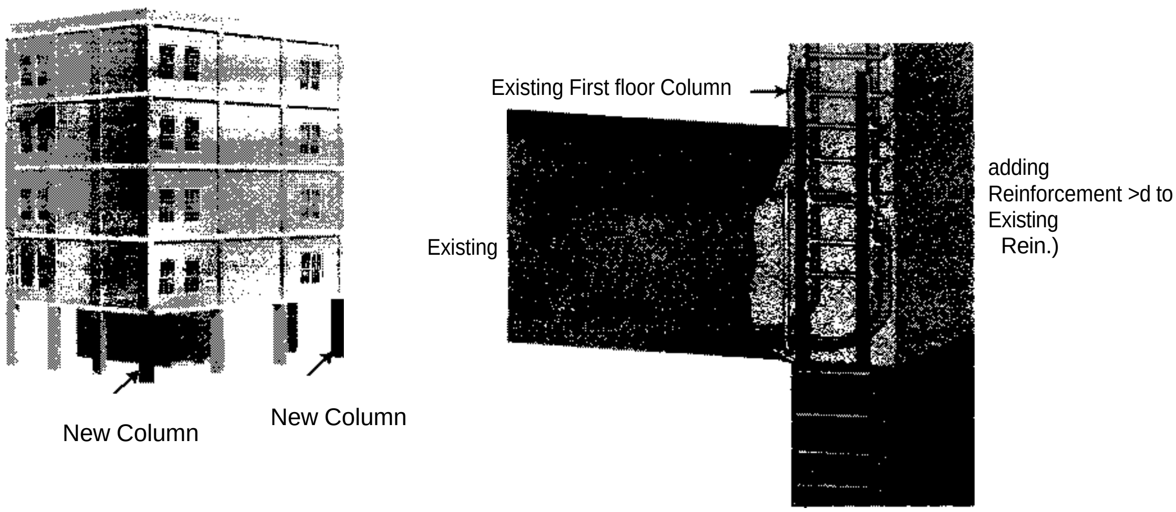


Figure 10.5: Adding a new column below a floating column

Similarly, a shear wall panel (Fig 10.6) is to be provided below the existing shear wall. This panel should have rigid shear connections with adjacent columns, beam/slab above it, and foundation.

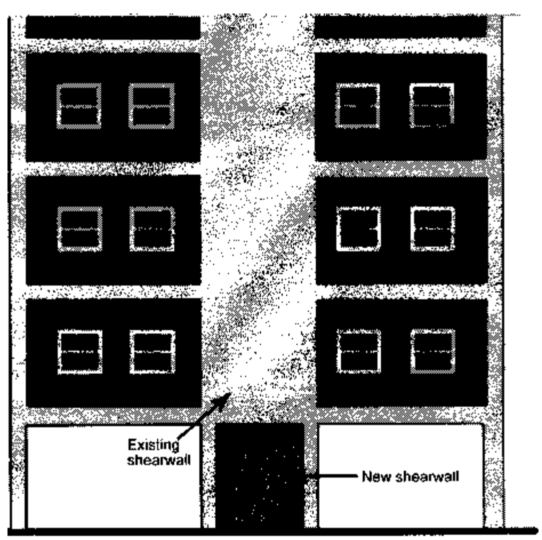


Figure 10.6: Adding a new shear wall at ground Floor

10.11.2 Removal of Configurational Irregularity due to Soft/Weak Storey

In buildings having soft/weak storey, most of the ductility demand is concentrated in the soft/weak storey, resulting in excessive lateral displacement of the storey leading to failure of the building due to formation of unstable mechanism.
In Indian cities there is a lack of parking space. In multistory buildings, the ground storey is usually kept open (free of masonry in-fills) while the upper storeys have masonry infills for partitions. It has been seen that such a configuration results in the stiffness of the ground storey about one-third of the stiffness of the upper storeys. IS: 1893-2002 has addressed this problem and suggests that either a non-linear analysis of such buildings should be performed or the ground storey beams and columns should be designed for a storey shear 2.5 times of that obtained from analysis of a bare frame without in-fills. In case of existing buildings the ground storey is to be stiffened so that the increased stiffness of the ground storey is nearly equal to the stiffness of the upper storeys.

10.11.3 Strengthening by Addition of New Members

Addition of new members is perhaps the easiest option to strengthen an existing building. Addition of new members is possible externally, without disturbing the space inside the building. The main concerns in addition of new members are the connection of new and old members and foundation of the new members. It is possible to provide only a few stiff members to take most of the earthquake force of the existing structure, but the connections should be capable of transferring this load. Similarly, the foundations should be capable of transferring this load to the ground.
Several options, in the form of frames, shear walls and vertical trusses, are possible for strengthening an existing building. Addition of new members changes dynamic characteristics of the building. Sometimes, new members are also added to reduce eccentricity. Therefore, re-analysis of structure is required after addition of members.

10.11.4 Strengthening of Existing Members

In most of the cases, strengthening of at least a few of existing members will be required in seismic retrofitting of a building. A number of techniques based on steel/FRP plate bonding, RCC jacketing and FRP jacketing are available for strengthening of individual members. The choice of the technique depends on the specific weakness and demand on the member.
Strengthening of individual members require good knowledge of the different materials available in the market for repair and retrofitting. The load transfer mechanism between the old and new material is complex and proper bonding between the two is difficult to be ensured. Following points are to be considered in strengthening of individual members:

- A variety of materials, discussed above is available for strengthening of existing members. A detailed study of manufacturer claimed properties of these materials is required before selecting a suitable material. Short-term as well as long-term properties are to be considered.
- The load transfer between old and new material can take place through several mechanisms, such as, compression against pre-cracked interfaces, adhesion between non-metallic materials, friction between non-metallic materials, load transfer through resin/glue layers, clamping effect of steel, dowel effect of steel, etc. Modelling of this

interaction is complex and not well understood, yet. It should be ensured that more than one mechanism of load transfer between new and old material are present.

- Anchorage lengths of reinforcement in new concrete should be as per codal specifications. However, in case of anchorage into old concrete, smaller anchorage lengths may be sufficient if special grouts are used to anchor the bars in drilled holes. This anchorage length should be in accordance with the manufacturer's specifications and should be verified by Pull-out tests.
- Anchoring of additional bars can also be accomplished by welding them with existing bars. For this purpose, spacers can be provided between old and new bars to provide a gap for intrusion of concrete. The weld is to be designed to develop full strength in the new bar.

10.11.5 Enhancing Deformation Capacity

Post yielding deformation capacity of a building plays a very important role in reducing the effective seismic force on the building. The members of a building are expected not to lose their vertical load carrying capacity, while undergoing large plastic deformations in lateral direction. Sometimes, a few poorly designed members can limit the capacity of the whole building to deform laterally. These members may be modified to increase their deformation capacity and this will result in large reduction in effective seismic force on the building. If the number of the members to be modified is small, this strategy does not disrupt the functioning of the building. But, if a large number of members are to be modified, this becomes costly and disruptive.

10.11.6 Earthquake Demand Reduction

An alternative approach for retrofitting of existing buildings is to reduce the earthquake demand (forces and displacements). This can be achieved either by reducing the mass of the building or using base-isolation/energy dissipation devices. Reduction of building mass is not always possible and it is mainly the use of base-isolation/supplemental energy dissipation devices, which is employed to reduce the earthquake demand on the buildings.
Use of base-isolation/supplemental energy dissipation devices is costly and it is recommended only for those building which are required to have operational performance level after an earthquake or which house sensitive equipment. Base-isolation has been found to be particularly useful for historic buildings, where it is not possible to modify the structure significantly. However, it is important to note that base-isolation and supplemental energy damping cannot be used in all buildings. In many cases, the structure is also to be strengthened in addition to base-isolation/energy dissipation.

Seismic Base-Isolation

Base-isolation is based on the principle of elongating the time period of the building by providing compliant bearings at the base of the building (Fig 10.7). The bearings have sufficient stiffness and strength against vertical load, but relatively low stiffness and large deformation capacity in lateral direction. Sometimes, these bearings are also provided with enhanced energy dissipation characteristics or with additional dampers.

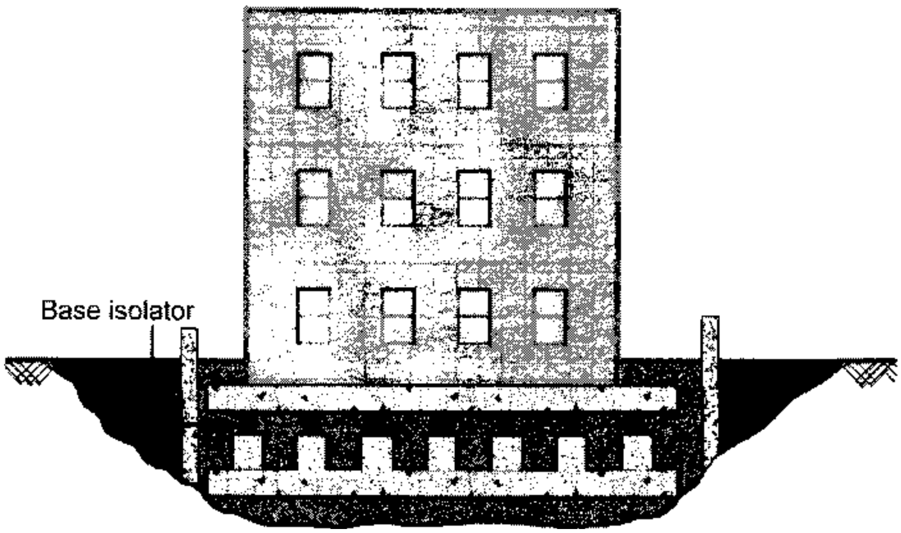


Figure 10.7: Schematic representation of a base isolated building

The base-isolation results in significant increase in fundamental time period of the structure and damping. Further, as the stiffness of the bearings is much smaller compared to structure, the lateral deformation gets concentrated into bearings, resulting in greatly reduced earthquake deformation demand in the portion of the structure above bearings.

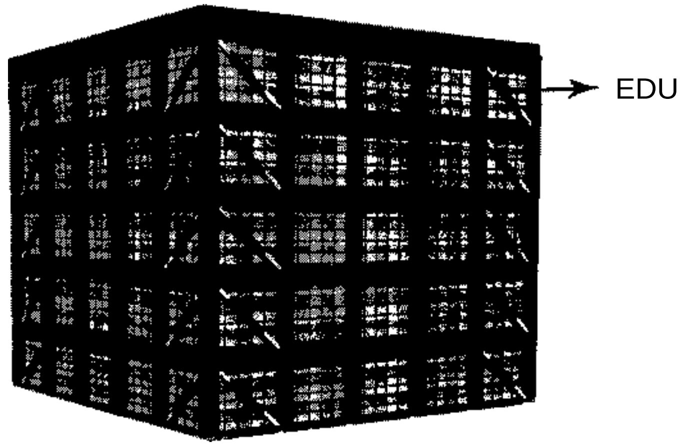
Base-isolation is considered to be useful for buildings having a fundamental time period of one second or less, as it requires a relatively stiff building to have concentration of lateral deformation in bearings only. Further, the building should remain elastic under the residual demand transmitted to the structure by the isolators. In order to achieve this, in many cases, the building structure is also required to be strengthened in addition to base-isolation.

Base-isolation is considered to be very effective for historical buildings, believing that no intervention/modification is required in the building, preserving its historical character. But, as described above, this belief may not be always true and significant strengthening of the structure may be required.

Base-isolation provides an effective solution for retrofitting of buildings having enhanced performance objectives. Base-isolation results in significant reduction of displacement and force response of the building. This is a preferable condition for better performance of sensitive equipment, systems and other non-structural components.

Supplemental Energy Dissipation

Supplemental Energy Dissipation Systems dissipate the energy transmitted to the structure by the earthquake, in addition to the energy dissipated by the structure in normal course. This results in significant reduction in the displacement and acceleration response of structure. For this purpose, energy dissipation devices (EDDs) are installed in the lateral load resisting system of the building (Figure 10.8). These EDDs work either on viscous or on hysteretic damping.



(a)

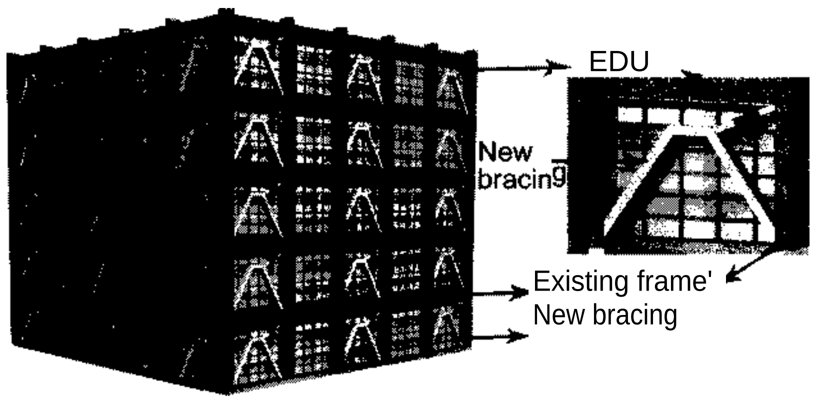


Figure 10.8: EDD's in a building frame: (a) along the diagonal, (b) mounted on a platform

EDDs can be mounted either along the diagonals (Fig. 18(a)) of the frames or on a rigid platform (Fig. 18(b)). Contrary to base-isolation, the energy dissipation system is more effective in flexible buildings with large lateral deformations, as the energy dissipated by EDUs is directly proportional to the force developed by EDUs and displacement across these EDUs.

For a rigid building, the small lateral displacement during earthquake will results in smaller energy dissipation and the reduction in effective earthquake forces will not be significant.

Similar to base-isolation, supplemental energy dissipation system is also a costly method and is suitable for buildings with high post-earthquake importance. The energy dissipation results in reduced seismic response of building and better performance of equipment, systems and non-structural components.

10.12 CONCLUSIONS

The need to improve the ability of an existing building to withstand seismic forces arises usually from the evidence of damage and poor behavior during a recent earthquake or from calculations or by comparisons with similar buildings that have been damaged in other places. Assessment of an existing structure is complicated task because of the defects arising from the original design, construction and deterioration since the completion of the construction. Vulnerability of building has also to be assessed with respect to earthquake hazard and ground failure hazard. Vulnerability assessment of buildings should start with visual inspection and study of all available documents followed by detailed in – situ investigations. The results could be analyzed using various mathematical modeling techniques. Vulnerability assessment of buildings is carried out in three levels as rapid visual screening (RVS) procedure (level – 1, procedure); simplified vulnerability assessment (SVA) procedure (level – 2, procedure) and detailed vulnerability assessment (DVA) procedure (level – 3, procedure). Strengthening of existing buildings is carried by using various repair, restoration and strengthening techniques. Different strengthening materials including grouts, bonding agents and replacement and jacketing materials should be used for this purpose. Various contemporary retrofitting techniques for load bearing as well as RC constructions discussed in the chapter have proved to be very effective in India as well as other countries.

CHAPTER 11

TECHNO-LEGAL AND TECHNO – FINANCIAL ASPECTS IN BUILDING PROJECTS

CHAPTER 11
TECHNO-LEGAL AND TECHNO – FINANCIAL ASPECTS IN BUILDING PROJECTS

11.1 INTRODUCTION

It was realized during some recent Indian earthquakes, namely Latur Maharashtra Earthquake 1993, Jabalpur Earthquake of 1997, Chamoli Earthquake of 1999 and the major earthquake in Kutch Gujarat in 2001, that, much of destruction has been due to the buildings constructed without earthquake safety measures as specified in Indian Standard Building Codes. Hence, the Ministry of Home Affairs GOI, appointed an Expert Group (consisting of a Senior Town Planner, five Architects and six Structural Engineers) to study the existing Municipal Byelaw's etc. and propose model Byelaws and regulations to be incorporated in the various legal documents for saving the constructions from earthquake and other hazards. The Expert Group studied the town and country planning legislations, development control regulations as well as building byelaws adopted in several states in the past. The Expert group submitted its report in two volumes, Volume I dealing with town and country planning legislation, land use zoning regulations, development control regulations, and building byelaws in *model form* which could be adopted by the States and the Cities by incorporating them in their existing documents. Volume II consisted of all the documents studied by the Expert Group. This chapter presents the same in abridged form for ready reference. The reader is encouraged to go through Volume I of this report, available in hard as well as soft copy from the NDM division of Ministry of Home Affairs, Government of India, North Block, Central Secretariat New Delhi.

11.2 RECOMMENDATIONS FOR AMENDMENT IN MODEL TOWN & COUNTRY PLANNING ACT, 1960

11.2.1 Definition under (Section 2)

Some of the important terms defined in section 2 of the act are given below:

Natural Hazard (16 a)
The probability of occurrence, within a specific period of time in a given area, of a potentially damaging natural phenomenon.

Natural Hazard Prone Areas (16 b)
Areas likely to have (i) moderate to very high damage risk zone of earthquakes, OR (ii) moderate to very high damage risk of cyclones OR (iii) significant flood flow or inundation, OR (iv) landslide potential or proneness, OR (v) one or more of these hazards.
Note: Moderate to very high damage risk zones of earthquakes are as shown in Seismic Zones III, IV and V specified in 1S:1893; moderate to very high damage risk zones of cyclones are those

areas along the sea coast of India prone to having wind velocities of 39 m/s or more as specified in IS:875(Part 3) and flood prone areas in river plains (unprotected and protected) are indicated in the Flood Atlas of India prepared by the Central Water Commission, besides, other areas can be flooded under conditions of heavy intensity rains, inundation in depressions, back flow in drains, inadequate drainage, etc. as identified through local surveys in the Development Plan of the area and landslide prone areas as identified by State Government/Local surveys.

Natural Disaster (16 c)

A serious disruption of the functioning of a society, causing widespread human, material or environmental losses caused due to earthquake, cyclone, flood or landslide which exceeds the ability of the affected society to cope using only its own resources.

Mitigation (16 d)

Measures taken in advance of a disaster aimed at decreasing or eliminating its impact on society and on environment including preparedness and prevention.

11.2.2 State Planning Board

Functions and Power of the Board as mentioned in Section 4(2) (a) is to direct the preparation of Development plans keeping in view the natural hazard proneness of the area by Local Planning Authorities

11.2.3 Functions and Power of Local Planning Authorities (Section 11)

The functions and power of local planning authorities essentially include:

- An Existing Land Use Map indicating hazard proneness of the area(11 a);
- An Interim Development Plan keeping in view the Regulations for Land Use Zoning for Natural Hazard Prone Areas (11 b)
- A Comprehensive Development Plan keeping in view the Regulations for Land Use Zoning for Natural Hazard Prone Areas (11 c)

11.2.4 Interim Development Plans (Section 18)

The interim development plans (section 18) indicate broadly the manner in which the planning authority proposes that land in such area should be used keeping in view the natural hazard proneness of the area; 18(2)(a)

11.2.5 Development Plan (Section 20)

If any local authority has been declared as a planning authority for a planning area & the local authority has prepared a development plan for the planning area before the application of this Act to that area, the development plan already prepared may be deemed to be a development plan under Section 18 or Section 19 of this Act. However, when such plans are implemented due care should be taken while formulating the projects based on such plans to follow the Regulations pertaining to Land Use Zoning and necessary protection measures prescribed by the Regulations.

11.2.6 Prohibition of Development without payment of Development Charges and without Permission (Section 29)

Any person or body (excluding a department of Central or State Government or local authority) intending to carry out any development on any land shall make an application in writing to the planning authority for permission in such form and containing such particulars and accompanied by such documents and plans as may be prescribed by the rules or the regulations including Development Control, Building Regulation/Byelaws for Natural Hazard Prone Areas. Provided that in the case of a department of Central or State Government or local authority (where the local authority is not also the planning authority) intending to carry out any development on any land, the concerned department or authority, as the case may be, shall notify in writing to the planning authority of its intension to do so, giving full particulars thereof and accompanied by such documents and plans "complying with development control, building regulations/bye-laws for natural hazard prone areas" as may be prescribed by the State government from time to time. [29(2)]

11.2.7 Power to make Regulations (Section 73)

Power to make regulations include:

- Any other matter which has to be or may be prescribed by rules under Section 72(1), Development Control and Building Regulations / Byelaws for Natural Hazard Prone Areas [73(e)],
- Any other matter which has to be or may be prescribed by regulation including Regulation for Land Use Zoning for Natural Hazard Prone Areas [73(f)]

Referred & Amendments suggested by identifying the relevant clauses in:

b. Model Regional and Town Planning and Development Laws 1985

c. Model Urban and Regional Planning and Development Law (Revised) (Part of UDPFI Guidelines)

11.3 RECOMMENDATIONS FOR LANDUSE ZONING REGULATIONS

The regulations for Land Use Zoning for Natural Hazard Prone Areas are to be notified under section:

1. u/s 73(f) of Model Town & Country Planning Act, 1960; OR
2. u/s 143(f) of Model Regional and Town Planning and Development Law; OR
3. u/s 181(f) of Model Urban & Regional Planning and Development Law

All these from UDPFI Guidelines (Revised) as may be applicable in the respective states under the existing provisions of Town & Country Planning Legislation as and when Master Plan / Development Plan of different cities / town / areas are formulated. However, these zoning regulations are to be implemented through the provisions of Development Control Regulations/ Building Bye-Laws, wherever the Master Plan is not in existence or not formulated.

11.3.1 Classification of urban land uses

It is based upon the requirements of the various plans. For example, a perspective plan, which is a policy document, need not show many details of a specific land use and may only show the main use which could be, say, residential or commercial. In the case of a development plan, which is a comprehensive plan indicating use of each parcel of land, there is a need to show more details of a specific land use. It has to indicate for the land designated as, say, commercial, the further details as to which land is for retail commercial, or for wholesale trade or for go-downs. In the case of layouts of projects of a shopping centre further details shall be necessary, indicating which block of retail commercial is for, say, cloth or electronics or vegetables. There could be three levels in land use classification shown under:

Level I For Perspective Plans
Level II For Development Plans
Level III For Layouts of Projects/Schemes

11.3.2 Objectives of Land Use Zoning

The objectives of land use zoning are as mentioned below:

1) The main purpose of the land use zoning is to provide regulations for development of a particular area to serve the desired purpose efficiently and to preserve its character. It also provides for the kind of buildings to be constructed. Zoning regulations are legal tools for guiding the use of land and protection of public health, welfare and safety.

2) Such regulations also include provisions for the use of premises /property and limitations upon shape, size and type of buildings that are constructed or occupy the land. Further, these provide both horizontal as well as vertical use of land.
These regulations also improve the quality of life in urban centres For instance in flood zones, the land use may be parks, playground & gardens while restricting any building activity in such vulnerable areas.

3) Life line structures should also be protected likewise while either proposing land uses or otherwise.

4) Zoning protects residential areas from harmful invasions of other uses like industrial use and commercial use. It does not prohibit use of lands and buildings that are lawfully established prior to coming into effect of such zoning regulations. If such uses are contrary to regulations in a particular 'use zone' and are not to be allowed, such uses are designated as 'non-conforming uses'. These are to be gradually eliminated without inflicting unreasonable hardship on the property owners/users.

5) The suggested list of uses/activities for various use zones should be comprehensive, keeping in mind the local and special characteristics of various sizes of settlements (large, medium and small). Depending upon the specific situation this list could be further enhanced or reduced, as the case may be.

11.3.3 State Perspective Plan/Regional Plan

While formulating Perspective Plan/Regional Plan, Development Plan (Master Plan/Zonal Development Plan) for any notified area, the proposals should indicate, Natural hazard prone areas with the type and extent of likely hazards,

11.3.4 Areas not covered Under Master Plan

In areas where there are no Master Plans or Development Plans, general guidelines & recommendations on natural disaster mitigation should be issued to the various local bodies, Municipalities and Town Area Committees and Panchayats to enable them to take these into consideration while sighting various projects and deciding on construction of buildings etc.

Technical help may be required by some of the local bodies in implementation of the recommendations and for interpretation of the guidelines.

11.3.5 Earthquake Prone Areas

The identified earthquake hazard prone areas are as given below:

1) Macro Seismic Zones III, IV & V

2) Area liable to liquefaction have greater risk.

3) Those hilly areas which are identified to have poor slope stability conditions and where landslides could be triggered by earthquake or where due to prior saturated conditions, mud flow could be initiated by earthquakes and where avalanches could be triggered by earthquake will be specially risk prone.

4) Special risky areas have to be determined specifically for the planning area under consideration through special studies to be carried out by geologists and geo-technical engineers.

11.3.6 Cyclone prone areas

The identified cyclone hazard prone areas are as given below:

1) Those areas likely to be subjected to heavy rain induced floods or to flooding by sea-water under the conditions of storm surge, are specially risky.

2) Areas under those where special risk have to be identified by special contour survey of the planning area under consideration and study of the past flooding and storm surge history of the area. Survey of India or locally appointed survey teams, and by reference to the Central Water Commission, Government of India and the department of the State or U.T dealing with the floods.

11.3.7 Flood prone areas:

The identified flood hazard prone areas are as given below:

1) These are in river plains (unprotected and protected by bunds) are indicated in the Flood Atlas of India prepared by the Central Water Commission and reproduced on larger scale in the state wise maps in the Vulnerability Atlas of India.

2) Besides, other areas can be flooded under conditions of heavy intensity rains, inundation in depressions, backflow in drains, inadequate drainage, failure of protection

works, etc. These have to be identified through local contour survey and study of the flood history of the planning area (Survey of India or local survey teams, and by reference to the Central Water Commission and the departments of the state or U.T dealing with the floods).

11.3.8 Land Slide Prone Areas

The susceptibility of the various areas to landslide varies from very low to very high. Landslide zoning naturally requires mapping on large scale. Normally medium scale of 1:25000 is at least chosen. In preparation of the landslide zone map, two types of factors are considered important as listed here below:

Geological/Topographic Factors/Parameters

Lithology, Geological Structures/Lineaments, Slope-dip (bedding, joint) relation,
•Geomorphology, Drainage, Slope angle, slope aspect and slope morphology,
•Land use, Soil texture and depth, Rock weathering

Triggering Factors

Rainfall, Earthquake, Anthropogeny

11.3.9 Alternatives

Various alternatives for land use zoning are as:

a) Leaving the area unprotected. In this case it will be necessary to specify Land Use Zoning for various development purposes as recommended.

b) Using protection methods for the areas as a whole or in the construction of buildings, structures & infrastructure facilities to cater for the hazard intensities likely in the planning area.

c) It will be appropriate to prioritize buildings, structures & infrastructures in terms of their importance from the point of view of impact of damage on the socio-economic structure of the society as recommended under Regulation no. 6. In regard to Land Use Zoning, different types of buildings and utility services are grouped under three priorities as indicated below.

- <u>Priority 1.</u> Defence installation, industries, public utilities, life line structures like hospitals, electricity installations, water supply, telephone exchange, aerodromes and railway stations; commercial centres, libraries, other buildings or installations with contents of high economic value.
- <u>Priority 2.</u> Public and Semi Public institutions, Government offices, and residential areas.
- <u>Priority 3.</u> Parks, play grounds, wood lands, gardens, green belts, and recreational areas.

d) Installations and Buildings of Priority 1 to be located above the levels corresponding to a 100 year flood or the maximum observed flood levels whichever higher.

Buildings of Priority 2 to be located outside the 25 year flood or a 10 year rainfall contour, provided that the buildings if constructed between the 10 and 25 year contours should have either high plinth level above 25 year flood mark or constructed on columns or stilts, with ground area left for the unimportant uses;
Activities of Priority 3 viz. play grounds, gardens and parks etc. can be located in areas vulnerable to frequent floods.

In order to ensure environmentally sound development of hill towns, the following restrictions and conditions may be proposed for future activities.

1) An integrated development plan may be prepared taking into consideration environmental and other relevant factors including ecologically sensitive areas, hazard prone areas, drainage channels, steep slopes and fertile land.
2) Water bodies including underground water bodies in water scarce areas should be protected.
3) Where cutting of hill slope in an area causes ecological damage and slope instability in adjacent areas, such cuttings shall not be undertaken unless appropriate measures are taken to avoid or prevent such damages.
4) No construction should be ordinarily undertaken in areas having slope above 30° or areas which fall in landslide hazard zones or areas falling on the spring lines and first order streams identified by the State Government on the basis of available scientific evidence.
5) Construction may be permitted in areas with slope between 10° to 30° or spring recharge areas or old landslide zones with such restrictions as the competent authority may decide.

11.3.10 Open Spaces

Out of the open spaces ear-marked as district parks, neighborhood parks and local parks in the development plan, zonal plans and local plans, suitable and approachable parks/ open spaces should be identified for the use during the emergency to provide shelter and relief caused by a natural hazard. Such pockets should be clearly marked on the city maps.

11.4 AMENDMENTS IN DEVELOPMENT CONTROL REGULATIONS

This part deals with the development control rules and general building requirements to ensure health and safety of the public. The regulations for Land Use Zoning in Hazard Prone Areas are to be taken into consideration while formulating the Development Plan and Area Plan under the Town Planning and Urban Development Act.

Every person who gives notice under relevant section of the Act shall furnish all information in forms and format prescribed herein and as may be amended from time to time by the Competent Authority. The following particulars and documents shall also be submitted along with the application.

1) The forms, plans, sections and descriptions to be furnished under these Development Control Regulations shall all be signed by each of the following persons:

- •A person making application for development permission under relevant section of the Act.
- •A person who has prepared the plans and sections with descriptions who may be Architect on Record or Engineer on Record.
- •A person who is responsible for the structural design of the construction i.e. a Structural Engineer on Record.
- •A Construction Engineer on Record who is to look after the day-today supervision of the construction.
- •A Developer, Promoter

2) A person who is engaged either to prepare plan or to prepare a structural design and structural report or to supervise the building shall give an undertaking:-

Certificate in the prescribed **Form No.l** by the "Owner, Developer, Structural Engineer on Record and Architect on Record"; **Form No.2** by the "Architect on Record"/ "Engineer on Record"; **and Form No. 3** by the "Structural Engineer on Record; **Form No. 4** by the Construction Engineer on Record" has to be provided.

No land shall be used as a site for the construction of building-

i) If the site is found to be liable to liquefaction by the Competent Authority under the earthquake intensity of the area, except where appropriate protection measures are taken.

ii) If the Competent Authority finds that the proposed development falls in the area liable to storm surge during cyclone, except where protection measures are adopted to prevent storm surge damage.

iii) In hilly terrain, the site plan should include location of land slide prone areas, if any, on or near the site, detected during reconnaissance. The Authority in such case shall cause to ensure that the site is away from such land slide prone areas.

iv) The site plan on a sloping site may also include proposals for diversion of the natural flow of water coming from uphill side of the building away from the foundation.

11.4.1 Grant or Refusal of the Permission for Development

On receipt of the application for Development Permission, the Competent Authority after making such inquiry and clearance from such an expert whenever considered necessary for the safety of building, as it thinks fit may communicate its decisions granting with or without condition including condition of submission of detailed working drawing/ structural drawing along with soil investigation report before the commencement of the work or refusing permission to the applicant as per the provisions of the Act.

The Competent Authority, however, may consider to grant exemption for submission of working drawing, structural drawing and soil investigation report in case the Competent Authority is satisfied that in the area where the proposed construction is to be taken, similar types of structure and soil investigation reports are already available on record and such request is from an individual owner/developer, having plot of not more than 500 sq mt. in size and for a maximum 3 storeyed residential building.

If the local site conditions do not require any soil testing or if a soil testing indicates that no special structural design is required, a small building having upto ground + 2 floors, having load bearing structure, may be constructed.
If the proposed small house is to be constructed with load bearing type masonry construction technique, where no structural design is involved, no certificate from a Structural Engineer on Record will be required (to be attached with **Form** No.2). However, a Structural Design Basis Report **(Form** No. 6), has to be submitted, duly filled in.
Notwithstanding anything stated in the above regulations it shall be incumbent on every person whose plans have been approved to submit revised (amended) plans for any structural deviations he proposes to make during the course of construction of his building work and the procedure laid down for plans or other documents here to before shall apply to all such Revised (amended) plans.

11.5 RECOMMENDATIONS FOR AMENDMENT IN BUILDING BYELAWS

11.5.1 List of BIS Codes to be complied with

List of BIS codes has to be complied with documents:

- For General Structural Safety
- For Cyclone/Wind Storm Protection
- For Earthquake Protection
- For Protection of Landslide Hazard

A list is given in Annexure V.
In compliance of the design with the above Indian Standard, the Structural Engineer on Record will submit a structural design basis report in the prescribed Proforma covering the essential safety requirements specified in the Standard. The"Structural Design Basis Report (SDBR)"consists of four parts

- Part-1 - General Information/ Data
- Part-2 - Load Bearing Masonry Buildings
- Part-3 - Reinforced Concrete Buildings
- Part-4- Steel Buildings

This report is to accompany the application for Building Development Permission.

11.5.2 Structural Design Review Panel

The Competent Authority shall create a Structural Design Review Panel (SDRP) consisting of senior SER's and SDAR's whose task will be to review and certify the design prepared by SER or SDAR whenever referred by the competent authority.
The Reviewing Agency shall submit addendum to the certificate or a new certificate in case of subsequent changes in structural design.

11.5.3 Supervision

All construction except load bearing buildings upto 3 storeys shall be carried out under the supervision of the Construction Engineer on Record (CER) or construction Management Agency on Record (CMAR) for various seismic zones.

11.5.4 Certification of Structural Safety in Construction

CER/CMAR shall give a certificate of structural safety of construction as per proforma given in Form-13 at the time of completion

Inspection

All the construction higher than 7 storeys, public building & special structures shall be carried out under quality inspection program prepared and implemented under the Quality Auditor on Record (QAR) or Quality Auditor Agency on Record (QAAR) in Seismic Zones IV & V

11.5.5 Certification of Safety in Quality of Construction

Quality inspection to be carried on the site shall be worked out by QAR/QAAR in consultation with the owner, builder, CER/CMAR. QAR/QAAR shall give a certificate of quality control as per proforma given in Form – 15

11.5.6 Other Issues

Vol I of the Expert Group also covers the following issues:

A).Structural Requirements of Low Cost Housing

B).Inspection

- General Requirements
- Record of Construction Progress
- Issue of Occupancy Certificate

C). Protective Measures in Natural Hazard Prone Areas

D). Registration of Professionals

E). Appointment of Professionals

F) Protection against Hazard

G) Registration, Qualifications and Duties of Professionals

H) General Duties and Responsibilities Applicable To All Professionals

1) Certificate of Undertaking for Hazard Safety Requirement
2) Certificate of Undertaking of Architect on Record/Engineer on Record
3) Certificate of Undertaking of Structural Engineer on Record (Ser)
4) Certificate of Undertaking of the Construction Engineer on Record
5) Development Permission
6) Structural Design Basis Report
7) Progress Certificate
8) Progress Certificate - First Storey
9) Progress Certificate - Middle Storey In Case Of High-Rise Building
10) Progress Certificate - Last Storey
11) Completion Report
12) Building Completion Certificate by Architect on Record
13) Building Completion Certificate by Construction Engineer on Record
14) Building Completion Certificate By structural Engineer On record
15) Model Proforma for Technical Audit Report
16) Structural Inspection Report

11.6 HURDLES AND BOTTLENECKS

Hurdles and bottlenecks in the enforcement of all the above mentioned acts are due to lack of:

- Legal support system
- Adequate trained manpower with the local bodies
- Proper infrastructure and facilities for testing of materials
- Trained masons/work force for construction of earthquake resistant structures
- Awareness amongst general public about the importance of providing earthquake resistant features and cost implication
- Awareness about the implication of using substandard building material s and improper use of technologies
- Coordination in processing the plans for sanction by various authorities

11.7 TECHNO-FINANCIAL REGIME

All civil constructions funded by public funds should incorporate disaster resistant technologies and it should be mandatory as a part of the financial package.

Financial institutions should make it mandatory for the client agencies to strictly adhere to codes and standards relating to safety requirements against natural hazards.

Institutional changes should be undertaken that would allow market forces to absorb catastrophic losses through the use of more optimally structured risk sharing arrangements. Insurance sector will have to be increasingly involved in risk reduction by evolving innovative mechanisms for risk reduction in their system.

11.8 COST BENEFIT ANALYSIS

The degree of damage is determined by direct and indirect damage. Apart from direct physical damage, natural disasters have effects on industrial and agricultural production, employment, drinking-water supply, and medical care, up to the point of losses in the trade balance and effects on the investment climate. Costs of damage due to different extreme events have to be compared with the costs for different possibilities of prevention and preparedness measures.

The expert group has prepared disaster scenario simulations under specific hazards for particularly vulnerable locations, with the intention to create a methodology of assessing the cost benefit aspects of damage forecasting and prevention. Three types of disaster scenarios are generated:

Scenario 1: All existing buildings are as they stand today.

Scenario 2: All existing buildings were actually constructed to be disaster resistant as per IS codes.

Scenario 3: All existing buildings were retrofitted to become disaster resistant.

Three locations were selected for the simulation of model viz. Kangra in Himachal Pradesh (repeat of 1905 earthquake), Coastal area of Andhra Pradesh (repeat of 1977 cyclone) and Burhi Dehang sub-basin in Assam (repeat of 1988 floods). The simulation studies for cost benefit analysis for Kangra earthquake of 1905 is given here:

11.8.1 Recreating the Kangra Earthquake of 1905

The 1905 Kangra earthquake of 8.0 on Richter scale killed 20,000 people in a relatively sparsely populated terrain. If that earthquake had happened again in 1991 (the year of Census, from where figures are taken), assuming a housing stock of 18,15,858 houses in the affected district, figures which are estimated as shown in Table 11.1:

Table 11.1 Recreation of Kangra Earthquake

	1905	(A) 1991	(B) 1991
		(Buildings as they are)	(Buildings constructed / retroffited with earthquake resistant features)
Loss of Lives	20,000+	65,000	12,000
Total Collapse	?	1,36,339	8,298
Partial Collapse	?	2,63,356	94,997
Heavy Damage	?	9,15,602	3,12,382
Moderate Damage	?	3,57,510	6,48,040
Monetary Losses	?	Rs. 5,104 cr.	Rs. 1,960 cr.

B1: Cost of earthquake resistant features in all houses if built initially as per code: 635 cr
B2: Cost of seismic retrofitting in all existing houses: 1525 crores

Net savings due to preventive measures:

Lives saved in both cases, B1 and B2	53,000 persons
Cost savings in B1	Rs. 2509 crores
Cost saving in B2	Rs. 1619 crores

In both cases, B1 and B2, cost of relief, rehabilitation and trauma will be reduced to less than one fifth.

11.9 CONCLUSIONS

The expert group appointed by the Ministry of Home Affairs GOI, to study the existing Municipal Byelaw's etc. and propose model Byelaws and regulations to be incorporated in the various legal documents for saving the constructions from earthquake and other hazards, submitted its report in two volumes. The techno legal actions suggested essentially include:

- Amending the Town & Country Planning Act to include consideration of Natural Disaster
- Amending Master Plan & Development Area Rules & Regulations to take into account the Hazard Proneness in Land Use Zoning.
- Amending the Building Byelaws of Local Bodies to enforce Earthquake Resistant Designs & Construction of Buildings including the Extensions/Modifications in the existing buildings.

The techno financial actions suggested facilitation of finances for the effective enforcement of the suggested regulations.

Studies regarding the costs of damage due to different extreme events are carried out by the expert group and presented in the form of models which when simulated for different past disasters showed the large amount of lives and money that could have been saved due to incorporation of disaster mitigation measures.

CHAPTER 12

ARCHITECTURAL DESIGN PROJECTS

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ARCHITECTURAL DESIGN PROJECTS

12.1 INTRODUCTION

People living in the seismic zones are now becoming increasingly aware of the seismic safety of the structures in which they live and work. Structural consultants are no longer designing only to meet government building code requirements but also going by the performance criteria laid down by their clients. Since, building codes are applicable to all buildings belonging to all strata of society the socio-economic conditions need to be carefully looked into before formulating them.

Most building codes in the developing world are aimed at only preventing the total collapse of a building, which can be referred to as “Life-Safety", or "Minimum Code Design”. The codes do not specify that there should be no or minimal structural damage to the building. For the user, even if building codes are followed strictly, there is no guarantee that the building will be habitable for living or doing business after an earthquake. Even if the building does not collapse, the structure may be so badly damaged that the building could be unusable and subsequently condemned. Then the owner would face additional costs to have the building demolished, loss of the entire real estate investment and the cost of reconstruction of the building. The Indian earthquake building code also follows the "Life-Safety" principal and hence suffers from the same disadvantage. With growing public awareness - many have decided against taking the associated risk. Clients are laying down additional conditions and safeguards to their architects and structural engineers as to how their building should perform in a seismic event. This method of structural design is popularly known as “Performance Based Design" because the client is spelling out a performance criteria which should be achieved for the structure he is paying.

Another huge factor governing these trends is the risk assessment exercise by the major insurance companies. Many insurance companies in Japan had to suffer huge losses after the Kobe earthquake of 1995; today they do not insure businesses and buildings that do not employ the state-of-the-art earthquake protective measures. Insurance companies are laying down performance conditions that the structures must comply with before getting insurance coverage. Insurance companies are refusing to insure against EQ losses unless buildings adhere to standards specified by them.

The chapter talks about some contemporary and popular methods of seismic safety and numerous architectural projects all over the world, with some outstanding concepts for seismic safety.

12.2 PERFORMANCE BASED DESIGN

These days in most cases the clients are not willing to accept the associated risk that the earthquake damage may expose them to. One Client may demand a building designed to resist up-to 6.5 magnitude earthquake while another may demand for a design to resist 8.0 magnitude on the Richter scale. Earthquake safety becomes very crucial for companies whose sole existence is on sensitive equipment that will make them inoperable in case it fails. It can also be mandatory for the Computer Data Centers in USA to be designed to withstand severest earthquakes as they house the sensitive data of not one but many hundreds of businesses. Also the US government has laid down very strict compliance criteria for seismic performance of hospitals. Stating that, "you cannot have deaths due to building collapse in a place that is meant to treat earthquake victims". Various government departments are also paying much attention on keeping essential infrastructure like bridges and airports operational even in case of major earthquake.

Solution to this problem is in developing a seismic system to be incorporated into the building which could somehow absorb the earthquake energy. This would result in decreasing the energy dissipation demand on the structural components i.e. beams, columns, slabs etc. This will increase the survivability of the building structure.

Several methods can be employed by which a building can withstand an earthquake with minimal structural damage. Some of the most popular ones used in contemporary architecture are discussed below.

12.3 ENERGY DISSIPATION SYSTEMS

The primary reason for introducing energy dissipation devices into a building frame is to reduce the displacement and damage in the frame. However with recent advances in seismic engineering, it is well proven that – stronger and stiffer buildings will have to dissipate or absorb more earthquake energy. Displacement reduction is archived by adding stiffness and/or energy dissipation (damping) to the building frame. (see section 10.11.6)

12.3.1 Passive Energy Dissipation Systems

Passive energy dissipation devices have been successfully used to reduce the dynamic response of structures subjected to earthquakes. These are conventional fixed-base systems which rely on strength and ductility to control seismic response. The enhancing of the seismic performance of fixed-base systems involves, dissipating the seismic energy through various Energy Dissipating Devices (EDD). These devices are like 'add-ons' to conventional fixed-base system to share the seismic demand along with primary structural members. A good design reduces the inelastic demand on primary structural members, leading to significant reduction in structural and non-structural damage

12.3.2 Active Control Systems

They control the seismic response through appropriate adjustments within the structure, as the seismic excitation changes. In other words, Active control systems introduce elements of dynamism and adaptability into the structure, thereby augmenting the capability to resist exceptional earthquake loads. A majority of the proposed techniques involves adjusting lateral strength, stiffness and dynamic properties of the structure during the earthquake to reduce the structural response.

12.4 DAMPERS OR ENERGY DISSIPATERS

As we have seen earlier dampers soak up the energy of earthquake-induced motion and instead of the building swinging back and forth repeatedly as earthquake vibrations are transmitted; it remains stationary as the motion of the dampers absorbs the energy (see section 6.3.8). There are four types of dampers as discussed below:

1. Traditional Viscoelastic dampers: These are stacked plates separated by inert polymer materials
2. Friction dampers: These consist of sliding steel plates and work on the principal that when two metal surfaces slide, friction heat is produced and energy gets dissipated
3. Metallic dampers: These consist of multiple steel plates which yield when a threshold force is reached.
4. Fluid viscous dampers: These are fluid filled metal cylinders with pistons and work like shock absorbers.

We will discuss the friction dampers and the fluid viscous dampers in detail here since they are the most widely used for earthquake energy dissipation.

12.4.1 Friction Dampers

Friction damper consists of three steel plates, rotating against each other around a pre-stressed bolt, which presses the plates together. Between these steel plates there are two circular friction pad discs, which provide dry friction lubrication in the unit, ensuring stable friction force of the movements (Fig 12.1).

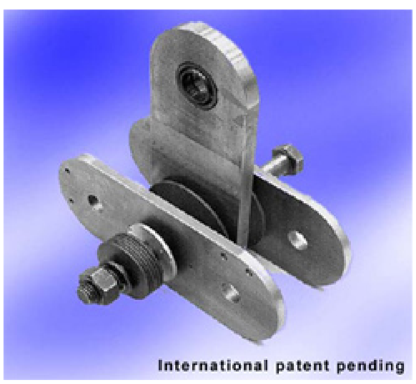


Figure 12.1: Friction damper

The central plates are connecting the damper device to the girder of the frame structure by a hinge, and the two sides are connected to the bracing system. (Fig 12.2 a). The bracing system consists of pre-tensioned bar members in order to avoid buckling. The bracing bars are pin-connected at both ends to the damper and to the beam–column joint. (Fig 12.2 b).

The magnitude of prestress applied to the bolt and to the bracing bar is according to earthquake type and peak ground acceleration (PGA) level.

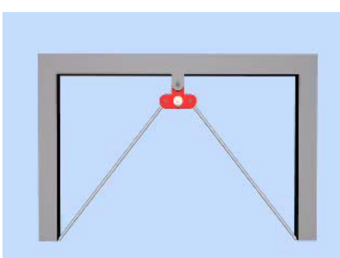


Figure 12.2 a: Connections of Friction Damper

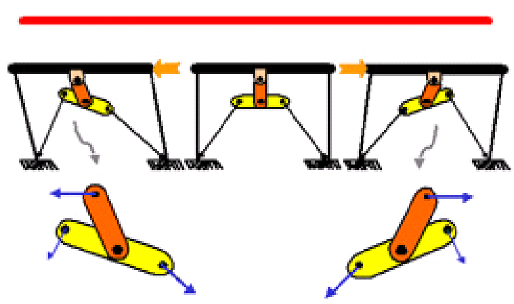


Figure 12.2 b: Mechanism of friction damper

These types of Friction dampers are also termed as wall dampers as it is fixed with the walling unit. The design of these dampers is very economical and can be installed within very short duration. Fig 12.3 and table 12.1 shows different types of Friction damper or wall dampers and their properties.

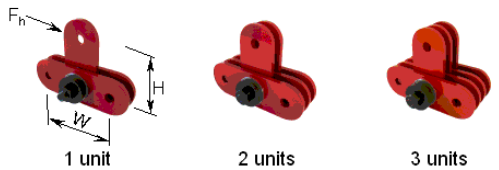
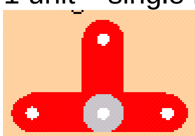
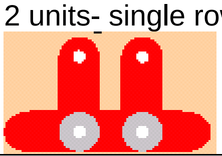
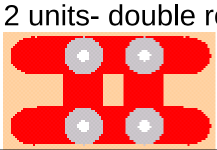
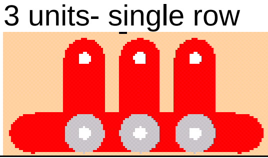
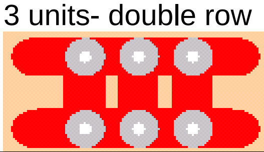
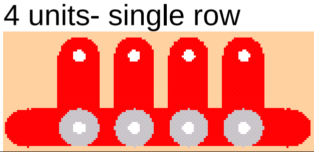
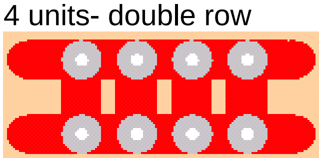


Figure 12.3: Types of friction dampers

Table 12.1: Types of friction dampers

Type	No. of units	Dimensions				Nominal slip capacity, Fh [kN]	
		Width (mm)		Height (mm)			
		min	max	min	max	min	max
1 unit – single row 	1	160	650	100	500	7	88
	2					13	176
	3					20	264
	4					27	352
2 units- single row 	1	240	1100	100	500	13	176
	2					27	352
	3					40	528
	4					54	703
2 units- double row 	1	240	1100	100	500	27	352
	2					54	703
	3					81	1055
	4					108	1407
3 units- single row 	1	320	1550	100	500	20	264
	2					40	528
	3					61	791
	4					81	1055
3 units- double row 	1	320	1550	100	500	40	528
	2					81	1055
	3					121	1583
	4					162	2110
4 units- single row 	1	400	2000	100	500	27	352
	2					54	703
	3					81	1055
	4					108	1407
4 units- double row 	1	400	2000	100	500	54	703
	2					108	1407
	3					162	2110
	4					216	2814
	5					270	3517

12.4.2 Fluid Viscous Dampers

Only fluid dampers can reduce both stress and deflection in a structure during a seismic event. They are relatively smaller in size and are self-contained. They can be easily installed in a structure as diagonal braces or as part of a base isolation system. Some of the advantages of fluid viscous dampers are:

- Long life, no maintenance;
- Highly effective in soft soil;
- Dramatically decrease earth quake induced motion;
- Less displacement; over 50% reduction in drift in many cases;
- Decreased base shear and inter-story shear, up to 40%;
- Reduced displacements and forces can mean less steel and concrete; and
- Less material means overall cost reduced and an energy efficient structure

Most simple example of fluid viscous damper is a shock absorber in motorcycle. Design of fluid viscous damper is shown in Fig 12. 4. and Fig 12.5.

Figure 12.4: Fluid viscous damper

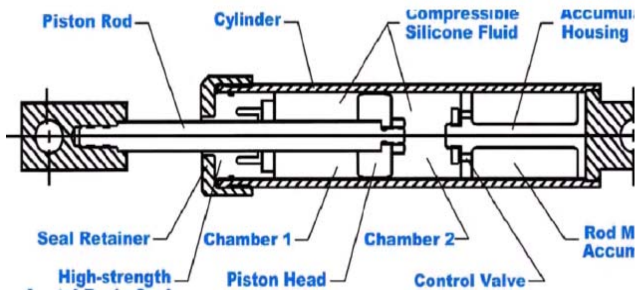


Figure 12.5: Design of fluid viscous damper.

The functioning of fluid viscous damper is as given below:

- Piston transmits energy entering the system to the fluid in the damper;
- Movement of the fluid within the damper absorbs this kinetic energy by converting it into heat
- After forces are removed the piston uses the same energy to regain its position

Hence, building columns protected by dampers will undergo considerably less horizontal movement and damage during an earthquake. Thus these dampers unlike other means are much effective in long run. Viscous dampers are especially effective in minimizing forces acting on building columns. They reduce drifts and shear forces without introducing axial column forces. Some uses of fluid viscous dampers are shown in Fig 12.6 a and Fig 12.6 b.

Figure 12.6 a

Figure 12.6.b

Figure 12.6: Uses of fluid viscous dampers

12.4.3 Base-Isolated Systems

As discussed earlier in section 10.11.6, base isolated systems are where; the superstructure is isolated from the foundation by certain devices, which reduce the ground motion transmitted to the structure. These devices help decouple the superstructure from damaging earthquake components and absorb seismic energy by adding significant damping. This technique considerably reduces the structural response and damages to structural as well as non-structural components.

Three types of base isolation systems are viz. Pile head isolation, Foundation isolation and Mid-level isolation. One more type is Hybrid Base Isolation System which consists of:

- Linear sliders
- Rubber bearings with lead plugs
- Multifunctional dampers

Fig 12.7 shows the representation of hybrid base isolator systems.

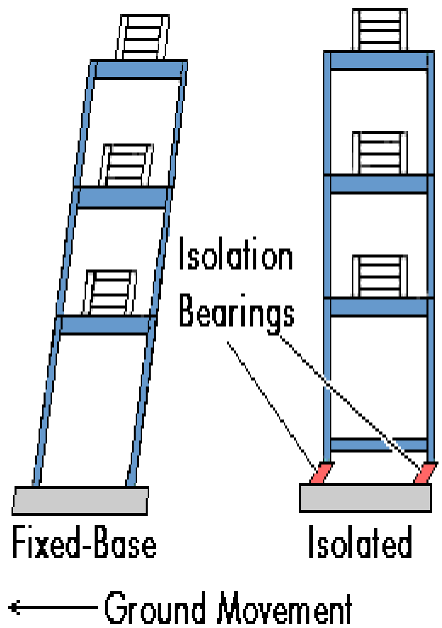


Figure 12.7: Base isolator system

The advantages of using Hybrid Base Isolation System are:

- Reduces the impact of very high seismic forces on buildings to minimum.
- Loss of life and property least.
- Function of building maintained during high earthquakes
- Can be installed in existing structural frame.

12.5 ARCHITECTURAL PROJECTS

This section talks about some of architectural projects all over the world designed for seismic safety.

12.5.1 Head Office of Himeji Shinkin Bank Japan, 1972, Yamashita Sekkei

Structure: Steel-framed reinforced concrete (partly steel structure)

Number of floors: 1 floor below and 8 floors above ground

Total floor space: 12,601.20 sqm.

Building area: 1,806.03 sqm.

Construction: Takenaka Corporation

Figure 12.8: Head Office of Himeji Shinkin Bank, Japan

In 1995 there was enactment of law for promoting improvements to increase earthquake resistance of buildings. Also, there was a need to use existing buildings for longer time periods due to low economic growth. This resulted in retrofitting for improving earthquake resistance of buildings without disrupting the normal operations of occupants such as in computer office buildings, banks, hospitals and collective housing.

Head Office of Himeji Shinkin Bank Japan (Fig 12.8) was constructed in 1972. For this purpose of retrofitting, costs of construction was to be kept as low as possible.

Making the right choice of the EQ system

To meet these demands, originally they reviewed adopting the base isolation method. For this, the gap between the underground exterior walls and the site boundary was narrow making installation of base isolation systems difficult, so mid-level isolation was adopted. The planning was carried out to isolate the first floor which was mainly a car park and to concentrate deformation on this floor in the event of an earthquake.

Retrofitting using Base Isolation Systems

Earthquake resistance strengthening and renewal work was carried out between May 1999–April 2000 by the Takenaka Corporation.

Mid-level isolation system was installed while the building was still being used. This new method entailed classifying and improving the columns on intermediate floors of an existing building into flexible columns that incorporate rubber bearings (base isolation systems) and rigid columns wrapped in steel plates to add to their toughness. A combination of these two types of columns was used to improve the earthquake-resistant performance of the whole building.

<u>It was first method of improving earthquake resistance in Japan that classified columns on the same floor as flexible columns and rigid columns, and first case in west Japan (Kansai region) of attaching rubber bearings by cutting columns on the intermediate floors of an existing building.</u>

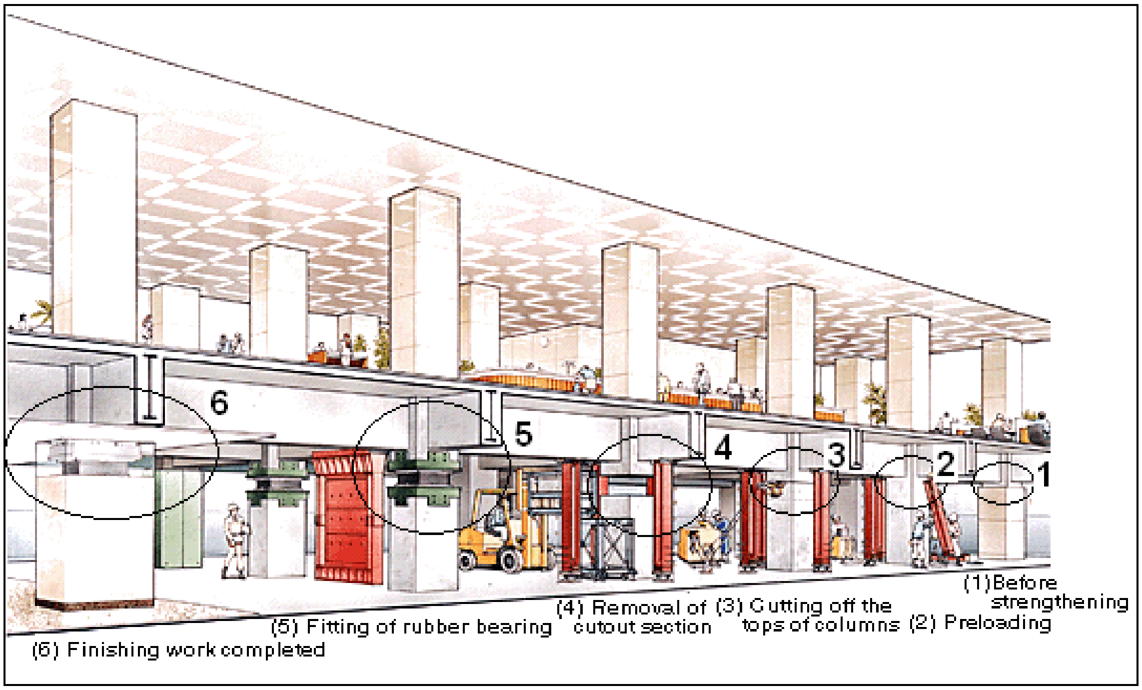


Figure 12.9: Retrofitting on the 1st Floor, 2nd Floor and above is business as usual)

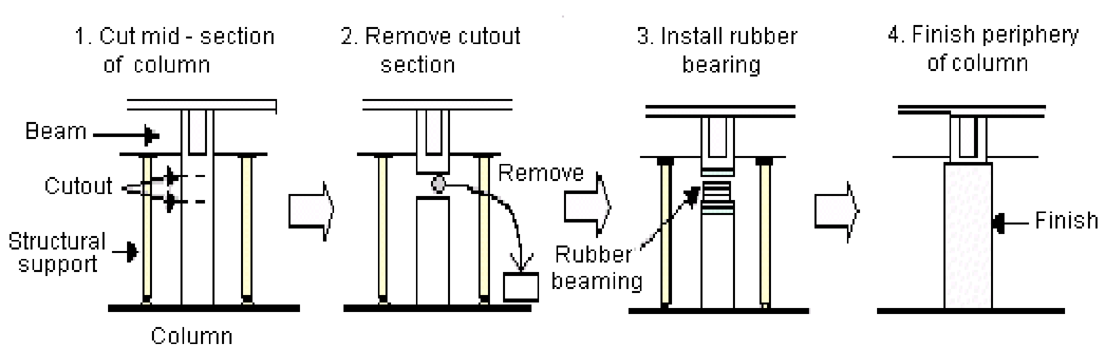


Figure 12.10: Base isolation improvement procedure for first floor columns (1) to (4)

Procedure

- Base Isolation System : 28 out of 44 columns on 1st floor were cut by a thickness of 50 cms., and rubber bearings were inserted into the gaps
- Strengthening: Remaining 16 columns were covered with steel plate of thickness 9-22 mm. (13 columns with 9 mm., and 3 columns with 22 mm.), thereby adding a toughness to the existing rigidity
- Dampers: Total 6 vibration control units incorporating viscous materials with high energy absorption performance were installed in walls, to play the role of dampers. This reduced the swaying of the building.
  (Fig 12.9 and Fig 12.10)

Earthquake resistant performance after retrofitting

- Tested assuming seismic motion on a par with the 1995 Great Hanshin Earthquake
- The acceleration that occurred on the 2nd floor and higher reduced to approximately 40 % of the figure before improvements.
- The damage to columns on 2nd floor and higher was limited to cracks, with the concrete not dropping and steel bars not being exposed

Shortcomings

By cutting all columns and walls horizontally on a specific intermediate floor (the first floor on this occasion) and installing rubber bearings in the columns that have been cut, that floor becomes extremely flexible, and the building will sway horizontally with the large sway amplitude of 40-50 centimeters under maximum level earthquakes. It therefore becomes possible that the finishing materials, piping and existing elevators may not be able to keep pace with the deformations and break, perhaps resulting in their protruding from the site of the building

12.5.2 DT Project Osaka Japan

DT Project Osaka Japan (Fig: 12.11)

- Location: Osaka Japan
- Work Period: Nov 2000-Dec 2002
- No. of Floors: 27
- Maximum Height:130 mts.
- Site Area: 3609 sq. mt.
- Building Area: 1613 sq. mt
- Total Floor Space:47613 sq. mt

Figure 12.11: DT Project Osaka, Japan

Linear Slider Base Isolator

- Installed under the third floor level
- Installed at twelve locations
- Blocks with internal bearings slide across rails, and by crisscrossing these rails, they can base isolate the building against all directions of shaking (Fig 12.12 and Fig 12.13)

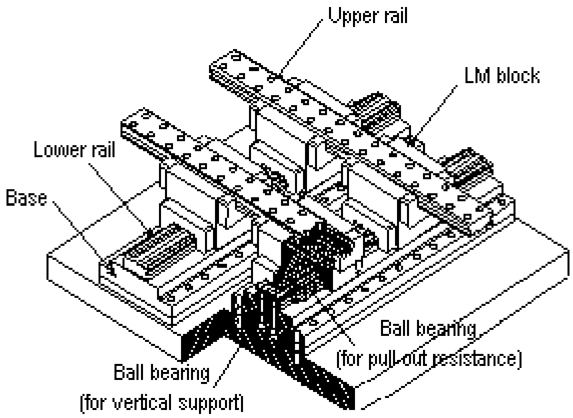


Figure 12.12: Linear slider base isolator

Figure 12.13: View of linear slider

Characteristics

- The friction coefficient is extremely low, around 0.01
- Flexibility in support load, 9000 tones during earthquake
- Maximum pull out resistance

Rubber bearings with lead plugs

- Rubber bearings act as springs
- The lead plugs inside the rubber bearings absorb the vibration energy. (Fig 12.14)

Multifunction dampers

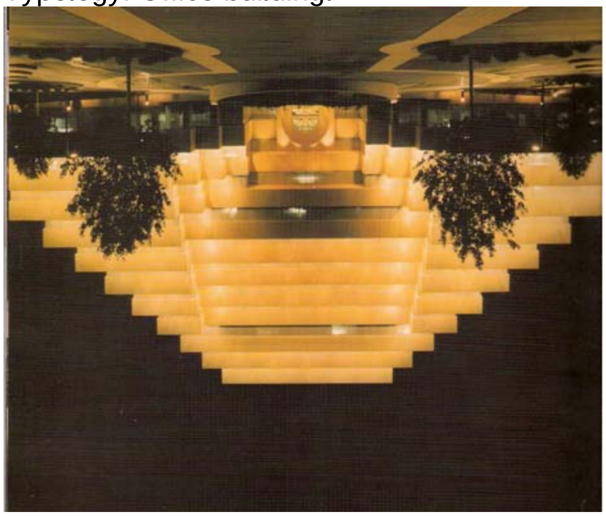
As the building sways, the flow of the oil inside cylinders absorbs the vibration energy. (Fig 12.15)

Figure 12.14: Rubber bearings with lead plugs

Figure 12.15: Multifunction dampers

12.5.3 Money Store Headquarter California, USA

Client: The Money Store Inc.
Architect: F.M. Kado & Associates.
Structural Engineer: Man Shatter & Miyamoto Inc.
Typology: Office building.



Established: Spring 1998

Cost of project: $85,000,000.

Structural system: Metal Frame work.

No. of floors: 11

Height of the building: 154 ft.

Floor area: 450,473 sq. ft.

Figure 12.16: Money Store Headquarter California, USA

Money Store Headquarter California, USA (Fig 12.16) was one of the first structures in the USA using seismic shock absorbers. Requirement was to create a building that could function after an earthquake with as little disruption as possible. The building's structural system is a pyramid-shaped, seismic-resistant frame. A Ziggurat shape was chosen because it provides a feeling of firmness & stability. The building steps back as it ascends; each floor is smaller than the floor below. The top level is rectangular measuring 92 by 128 feet.

Earthquake in area:

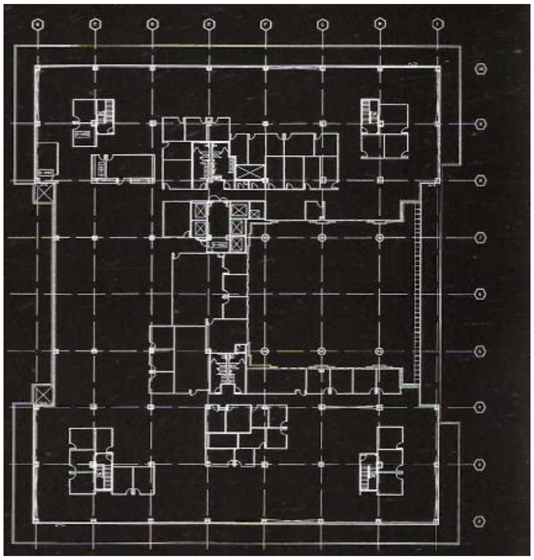
The strongest earthquake, magnitude 6.75 on the Richter scale, occurred in 1892. Its epicenter was 25 miles west of the site.

Seismic Design

- Very economical viscous liquid shock absorbers.
- A 60% reduction in the lateral forces affecting the foundation of the building and a 30% reduction in structural displacement.
- The horizontal movement of one floor with respect to the next is almost totally eliminated.

Cost of Seismic Design

Less than 1% of the total building cost.



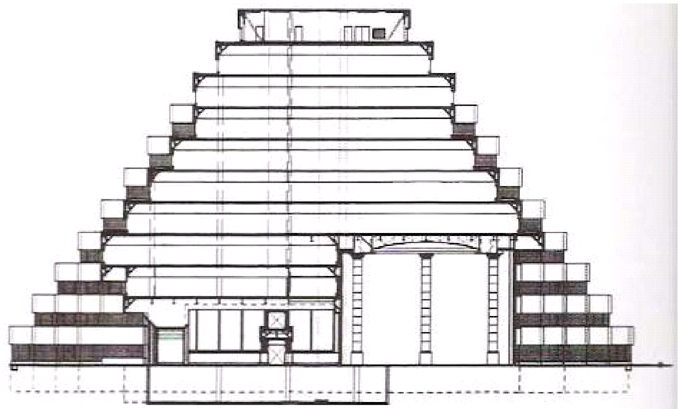


Figure 12.17 a: Plan of building   Figure 12.17 b: Sectional elevation of building
Figure 12.17: Building form of Money Store Headquarter California, USA

The stepping of form extends the space of each floor. This permits easy maintenance of the façades. This also makes it possible to evacuate quickly in an emergency. (Fig 12.17, Fig 12.18)

The superstructure is reinforced by a lateral system which enables the building to withstand the most critical moments

The elastic, resistant frames act as braces for the post-and-lintel structure, to which viscous liquid shock absorbers have been added. (Fig 12.19)

Figure 12.18: View of onsite construction

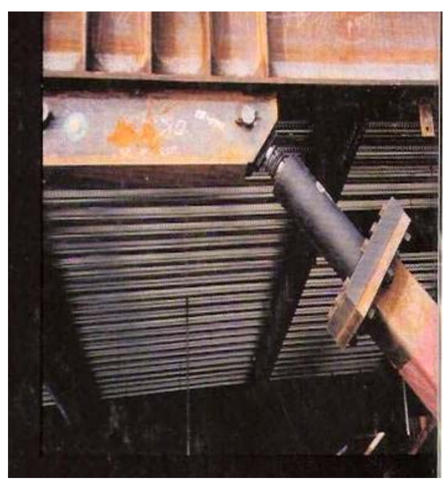


Figure 12.19: Close-up view of damper

Result
A highly seismic resistant building.

12.5.4 Retrofitting of School building in Himachal Pradesh, India

An existing reinforced concrete school building in lower Himalayan region of India is a double storey structure with use of Friction dampers / Wall dampers (Fig 12.20). The design of these dampers is very economical and can be installed within very short duration. The Friction dampers are installed on every floor between structural members. By these dampers, building can withstand during earthquake shocks up to 6.5 – 8 on Richter scale and able to maintain the shape i.e. it will not deform at the time of induced motion due to earthquake.

Figure 12.20 a

Figure 12.20 c

Figure 12.20 d

Figure 12.20 e

Figure 12.20 f

Figure 12.20: Retrofitting of School building in Himachal Pradesh, India

12.5.5 The Yapi Kredi Bank Operations Center, Gehze, Turkey, John Mcaslan & Partners, 1993-98

High risk seismic zone

Prediction – massive earthquake every 500 years

Designed to prevent total collapse of framework

No structural damage expected in small tremors

Has faced 2 earthquakes – minimal damage

Figure 12.21: The Yapi Kredi Bank Operations Center, Gehze, Turkey,

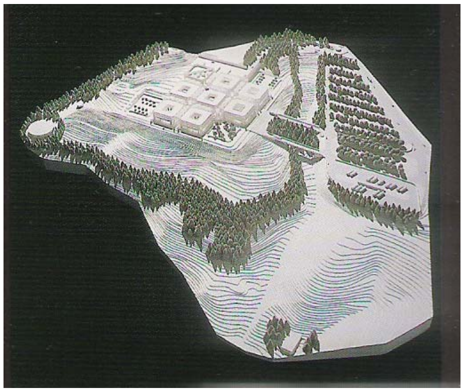


Figure 12.22: Site Model

Figure 12.23: Site View

General view

The Yapi Kredi Bank Operations Center, Gehze, Turkey, sits on top of the hill taking advantage of sea views and natural setting (Fig 12.21 to Fig 12.23). It is an excellent example of seismic safety in a building on a hilly site. It is a safe building on a high seismic risk zone without compromising flexibility in design, i.e., adaptable to changing needs of firm along with architectural quality to reflect the company’s prestige. All floors have same amount of space. Great sensitivity is provided to site landscape. The vegetation is used to differentiate arcades from each other for visitor orientation.

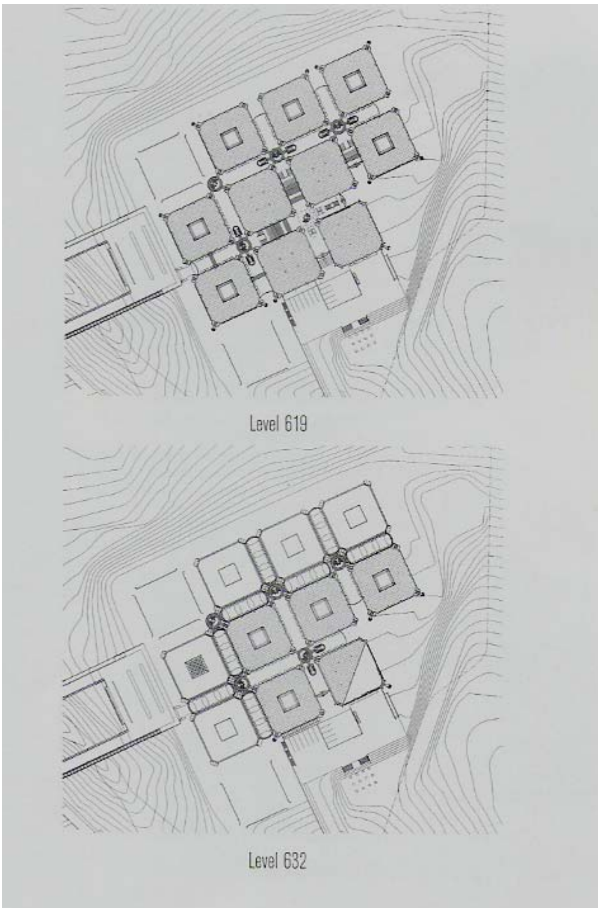


Figure 12.24: Various level Plans

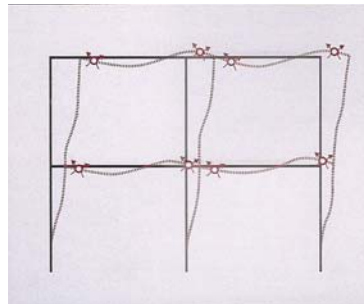


Figure 12.25 a

Figure 12.25 b

Figure 12.25: Structural system

Structural Layout

- Structural layout ensures that centre can be expanded indefinitely
- Guarantees adaptability and enhances seismic resistance as each unit is independent(Figure 12.24 and Fig 12.25)
- Steel building skeleton, stairs, bridges over various passageways, panel coverings – all designed to neutralise horizontal/vertical movements

Seismic performance

- In case of massive earthquake (1/500 years) – plastic articulations in beams will absorb energy released during tremor, preventing collapse of framework
- Seismic standard – beam flexibility over column flexibility
- Aluminum frames to allow for variations in concrete and the capacity of primary structure to absorb seismic movements
- 16 columns on perimeter + 4 on interior patio absorb vertical forces
- Stability of columns from exterior framework
- Since all 10 units/components structurally independent , arcades between them to absorb building oscillations upto 2.36”
- Metal framework with arches supported at 2 points
- Flexible covering needed - textile membrane

12.5.6 The National Te Papa Tongarewa Museum Of New Zealand, Wellington, New Zealand, Jasmax Architects, 1993-98

Figure 12.26:
The National Te Papa Tongarewa Museum of New Zealand, Wellington, New Zealand,

Wellington is one of the most earthquake prone regions in NZ and also susceptible to strong winds. The National Te Papa Tongarewa Museum of New Zealand, Wellington, New Zealand, is spectacularly located on the waterfront in Wellington surrounded by mountains (Fig 12.26). It is housing country's cultural treasures in an area of 395,000 sq.ft. Its fundamental aim is integrating the European and indigenous cultures. Activities in various areas of building are linked according to their cultural origins

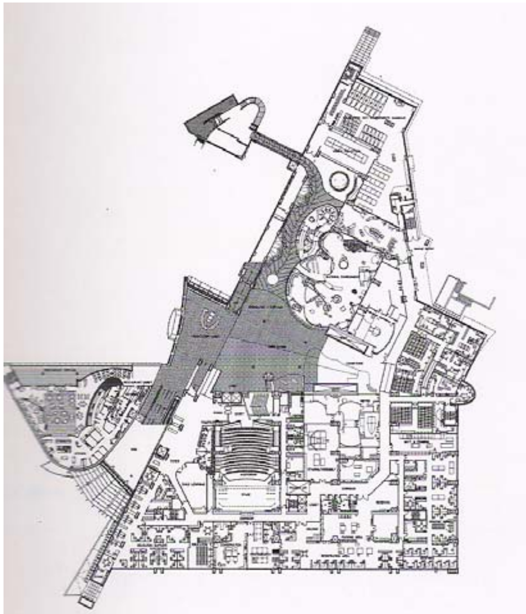


Figure 12.27: A Floor Plan

Figure 12.28: Model

Seismic safety in an irregularly shaped building

Building's structure and form are conditioned by local geographic and atmospheric characteristics (Fig 12.27 and Fig 12.28)

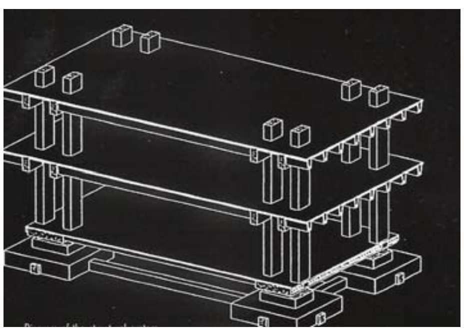


Figure 12.29: Structural System

Structural system

Basic building structure a series of 5 storey high concrete porticos reinforced by walls braced in only one direction with most beams and slabs prefabricated

At the base, structure has economical, highly earthquake-resistant isolation system employing shock absorbing elements

Principal advantage – freedom to the remaining structure (walls, external coverings and floor slabs) + the consequent design flexibility

Reminder of the Domino system of Le Corbusier (Fig 12.29)

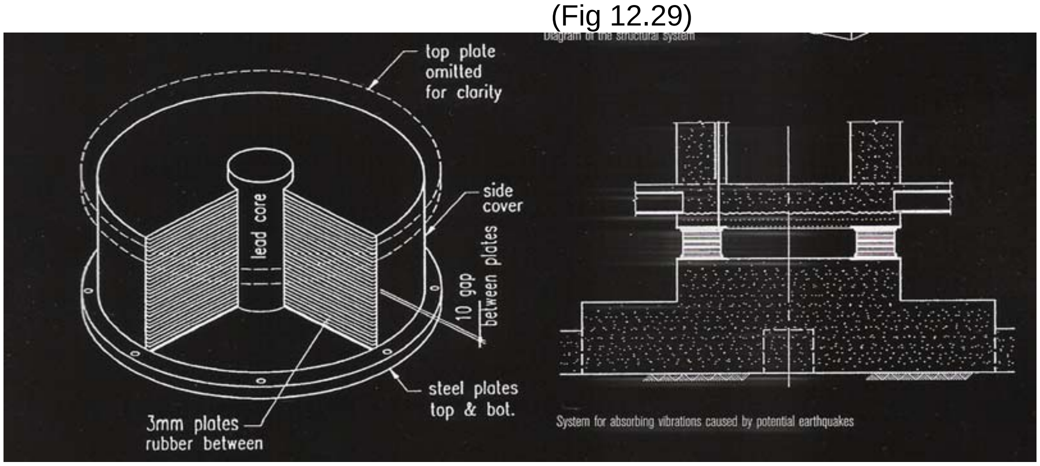


Figure 12.30: Base isolator

Use of Base Isolator

- Isolating base with 142 steel and rubber, lead cored elements attached with screws (Fig 12.30)
- Each isolator placed between foundation and building structure under columns – absorbing part of seismic energy transmitted through ground
- Damage through tremors mitigated as horizontal earthquake oscillations withstood by isolators – preventing structural collapse

12.5.7 Expansion and Renovation San Francisco Court Of Appeals, California,Som, 1993-97

Figure 12.31: San Francisco Court Of Appeals, California, Som

Preserving a historical building from earthquake

There were two demanding goals to be acheived:

1. Almost 100 year old building needed appropriate technological and structural renovation to prevent earthquake damage
2. Need for blending of the old and new

During the earthquake in 1989 (magnitude 7.1), a serious damage had occurred to building requiring repair and renovation. Hence, Two-pronged earthquake resistance strategy for formulated involving:

1. Finding the appropriate isolators &
2. Placing of these isolators.

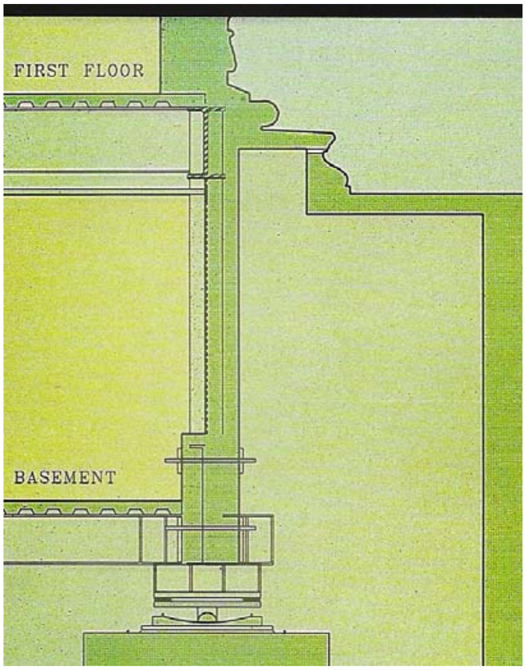


Figure 12.32: Friction Pendulum Bearing

Seismic Solution

- A Friction Pendulum System (FPS) was placed between the foundation and basement. (Fig 12.32)
- Each main column was cut and then supported on hydraulic jacks
- 256 pendulum bearings, concave stainless steel cylinders, attached under the top part of the severed columns
- These were repositioned on sliders attached to the top of the lower section of the columns

(Fig 12.33 and Fig 12.34)

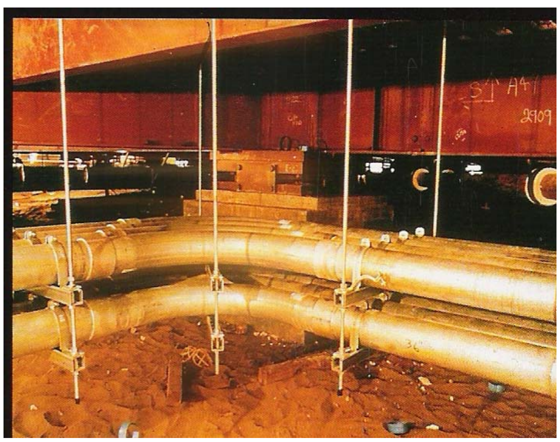


Figure 12.33: Building under construction

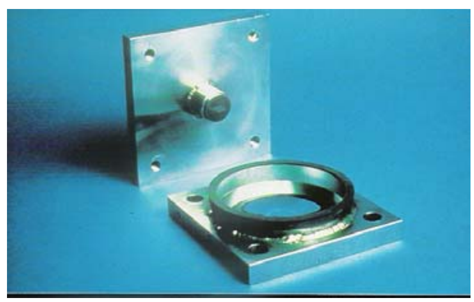
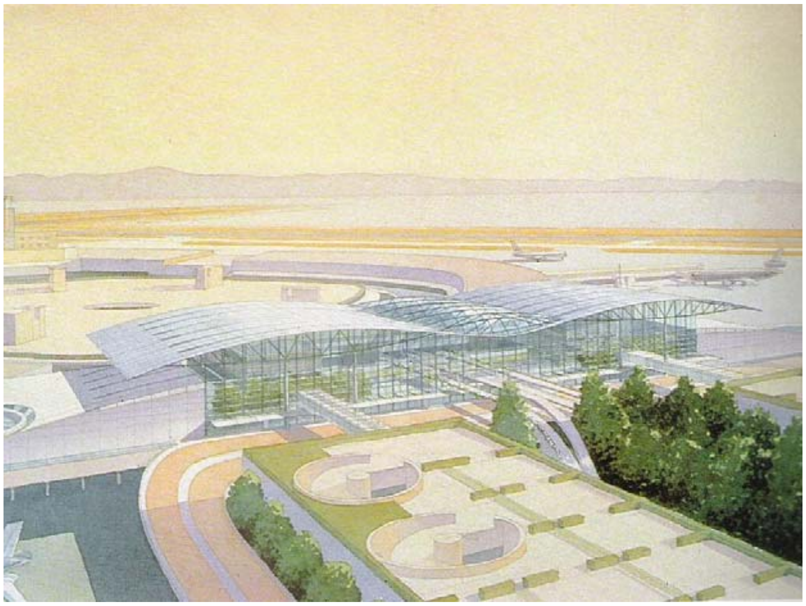


Figure 12.34: Sliding disks in a FP Bearing

In this way the whole building can rock as a single unit in an earthquake. Due to these concave disks – building can even rise slightly, sliding about 2 feet on the steel bowl. It was tested by University of Berkeley and structure can withstand magnitude 8.0 earthquake was proved. Also FPS resulted in a gain of 40,000 sq.ft. of floor space in basement.

12.5.8 New International Terminal, San Francisco Airport, California, Som, 1993-2000



The New International Terminal, San Francisco Airport, California had two main architectural elements – the roof structure and the façade. (Fig 12.35 and Fig 12.36)
The few columns resulted in fluidity of space.
The Glass & steel curtain wall – diffused sunlight and provided transparency and lightness to the structure.

Figure 12.35: New International Terminal, San Francisco Airport, California, 1993-2000

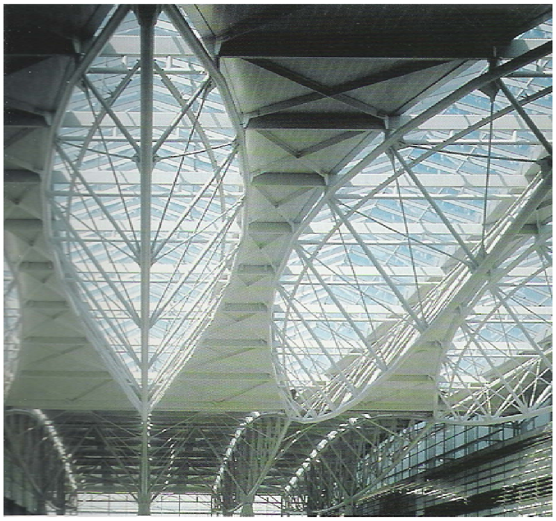


Figure 12.36: Roof of terminal

Figure 12.37: Detail of Friction Pendulum Bearing Isolator

This main terminal building is designed for a 1000 year 'earthquake' period. Base isolators are provided for all 267 columns to stabilize the building and damp movements in an earthquake (Fig 12.37). Pendulum system which is installed, allows a 20" displacement of steel piece on the non-stick concave surface. It is assumed that as earthquake subsides, self-weight of building compensates for possible displacement. Damping system was compensated by 20" wide space left around entire building to avoid damage caused to adjacent buildings due to terminal's sway.

12.5.9 First Interstate World Center, Los Angeles, California, Pei Cobb Freed & Partners, 1987-89

Figure 12.38: First Interstate World Center, Los Angeles, California

First Interstate World Center, Los Angeles, California is an example of Vertical architecture which is not suited for California because there are many faults beneath resulting in very high seismic risk. But commitment to preserve historic architecture and desire to have a landmark city centre was rationale behind this project. It is designed for seismic resistance of magnitude 8.3 on the Richter scale.

In this project local earthquake conditions have defined structural system & resulting composition. The framework required to meet conflicting demands of earthquake (flexibility) and wind (rigidity) using simple structural elements. Hence the structural system includes geometric and structural elements combined to create rich visual perspective & a specific architectural identity. (Fig. 12.39)

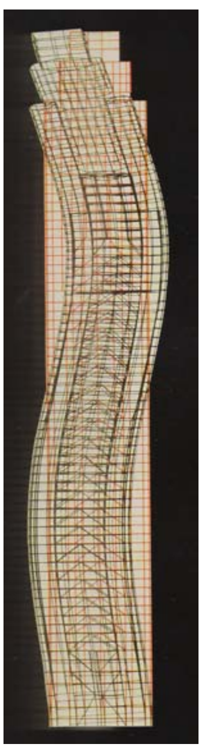


Figure 12.39: Building reaction to 1 of 32 vibrations was studied

A combination of 2 structural systems was employed:

- A resistant steel frame on the outer perimeter designed as ductile element to absorb seismic movement (Fig 12.40)
- A square rigid internal core for the entire height of building – lateral strength for high-velocity winds (Fig 12.41)

Base of core reinforced with 2-storey high braces, makes tower's large window openings possible and apparent lack of structural elements

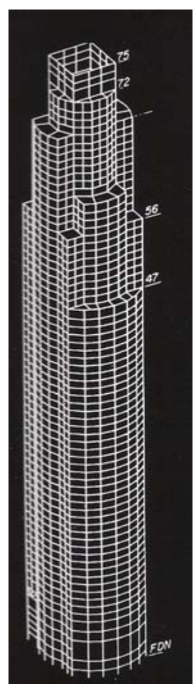


Figure 12.40: Perimeter of building

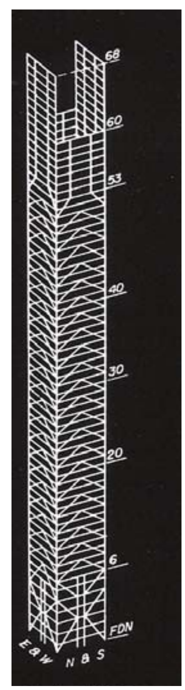


Figure 12.41: Core of building

12.5.10 Kansai International Airport, Osaka Bay, Kansai, Japan, Renzo Piano Building Workshop, 1988-94

Figure 12.42: Kansai International Airport, Osaka Bay, Kansai, Japan

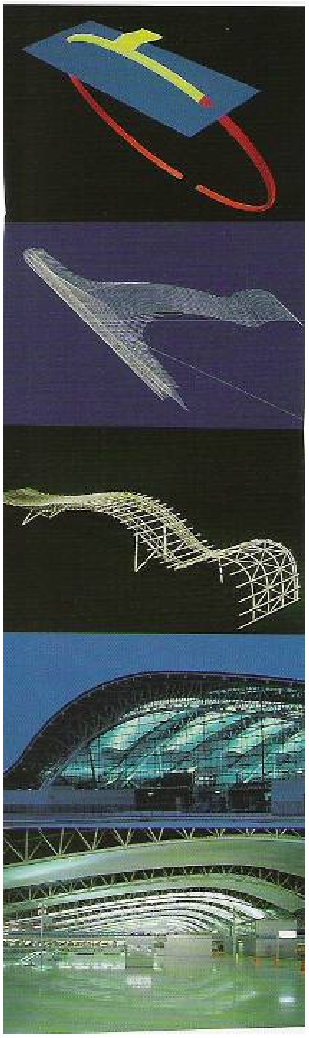


Figure 12.43: General structure reminds of wave

Kansai International Airport, Osaka Bay, Kansai, Japan is built on a man-made island 3 miles from Osaka and serves Osaka, Kobe and Kyoto. It is built on more than a 1000 piles in unstable terrain (Fig 12.42). These piles pass thru 65' water, 65' mud and anchored in 140' of rock. A system of sensors is employed in the piles to warn of shift which is acceptable at 4/10". Each pile has own calibration system which readjusts the depth using powerful hydraulic jacks. There are strict standards not only to deal with earthquakes & tsunamis but also for passenger volume. During Kobe earthquake 1995 (magnitude 7.2, epicentre 28 miles) – no damage occurred to the building except for light settling at some spots in island perimeter.

Building form

- Asymmetrical shape helps passengers orient themselves
- Higher portion overlooks the runways, handles passengers and guides them to their correct destination
- Each of 4 levels have specific function – international arrivals, domestic flight departures & arrivals, shopping areas, restaurants and international departures

(Fig. 12.43 to Fig 12. 47)

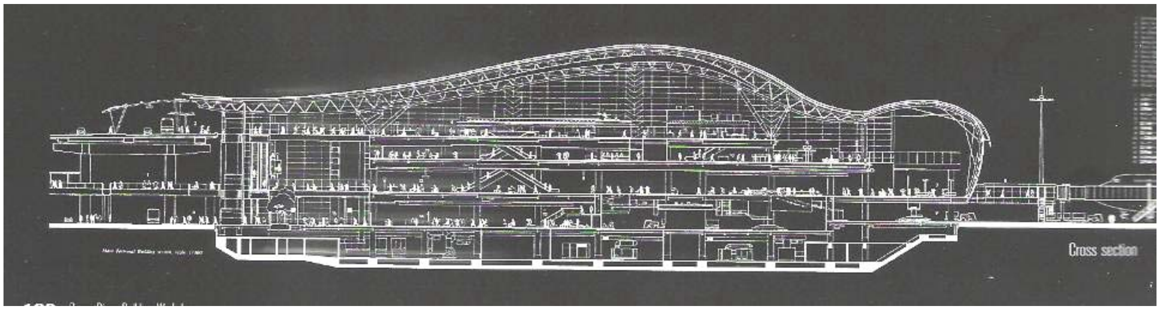
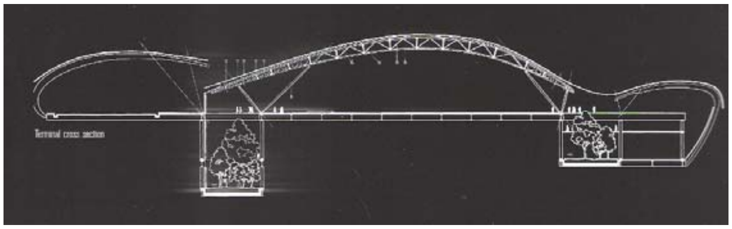


Figure 12.44: Section of Kansai International Airport, Osaka Bay, Kansai, Japan

Figure 12.45: Light, fluid structure of the airport but does not sacrifice any resistance

Figure 12.46: All building systems near G.L. to minimise wt. on each floor



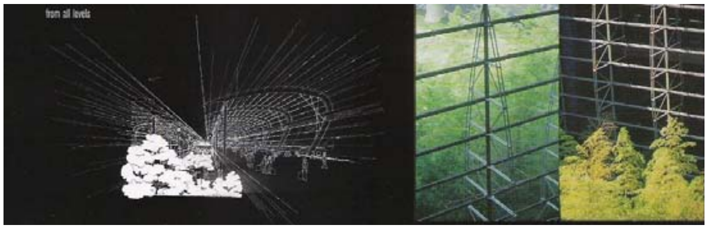


Figure 12.47: Indoor vegetation with trees bathed in sunlight

12.5.11 Emergency Management Centre In Foligno, Italy

Figure 12.48 a: Emergency management centre in Foligno, Italy

The significant challenge in design of Emergency Management Centre in Foligno, Italy was visually expressing “earthquake-resistant” devices. Most common was to install seismic isolators is the basement which is often not an area readily exposed to public view. Isolators often blend into the shadows and look like traditional bearing or support to passerby, even when exposed on a building or bridge. One obvious way to accent these features is use of color. Active control devices are usually hidden away on the roof or in a special room the way mechanical equipment is always concealed from view.

Emergency management centre in Foligno, Italy can be used to direct post- earthquake emergency operations. Overall configuration of the structure as well as the use of isolators clearly expressed the earthquake safety. This requires care in the structural and architectural detailing and effective use of contrasting colors.

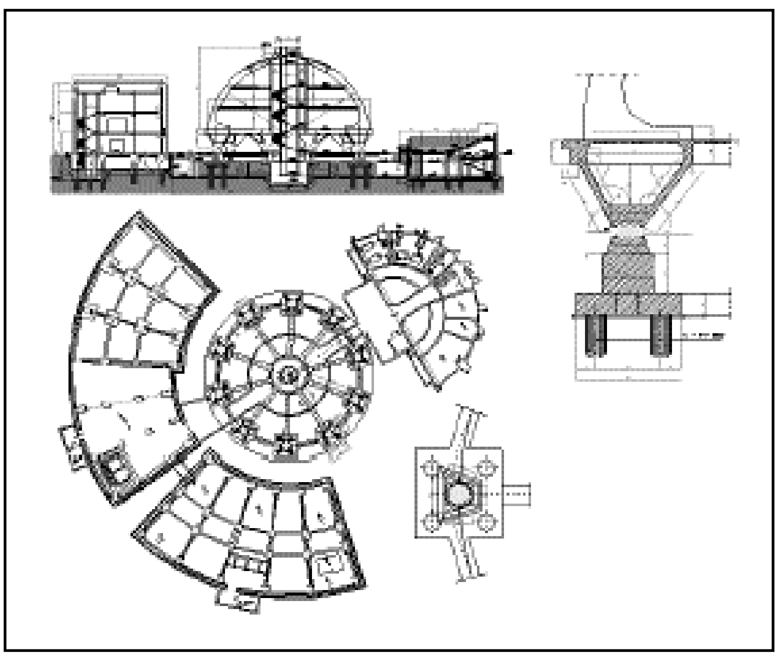


Figure 12.48 b: Plan, section and detail of isolator of Emergency management centre in Foligno, Italy

12.5.12 Taipei 101, Taipei, Taiwan.

Architect: C. Y. Lee and Associates. Structural Design: Thornton-Tomasetti

The Taipei 101, Taipei, Taiwan with height of 509 meters (1670 ft) holds several “tallest building” records, including the highest occupied floor. The design is inspired by traditional Chinese architecture, with a shape resembling a pagoda. The sectioned tower is also inspired by bamboo plant which is a model of strength, resilience, and elegance. (Fig 12.49)

The structural design of the Taipei 101 includes its mega-column and mega-frame layout. Superstructure is composed of large multi-story units that are said to form an architectural effect resembling a bamboo shaft. At the base, the large built-up steel columns slant outward as they go down, this has a beneficial effect in resisting lateral forces.

Figure 12.49: The Taipei 101, Taipei, Taiwan

Figure 12.50: Ball shaped damper in the Taipei 101, Taiwan

Damper

The damper is an enormous 800-ton, 6 m dia. ball of welded steel plates (Fig 12.50 and Fig 12.51). It is constructed of thick plates of steel and is suspended from cables like a pendulum. It hangs inside the top of the building and is visible from the restaurant and bar which encircles the space around the ball. Damper helps to stabilize the tower in high winds and earthquakes. It can reduce tower's movements by up to 40%. The device's components were made in Canada, France and Italy -- except for the massive steel plates, which were made in Taiwan.

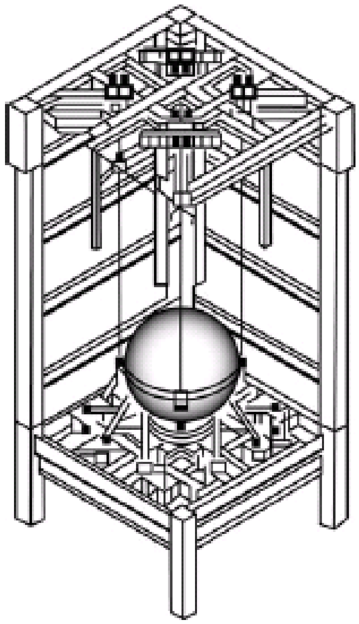


Figure 12.51: Massive suspended Damper Sphere

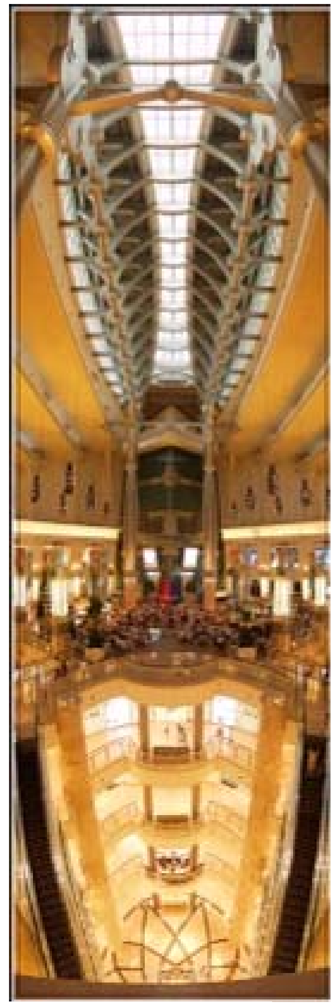


Figure 12.52 a

Figure 12.52 b

Figure 12.52: Visitors to restaurants, bars and observation decks between the 88th and 92nd floors are able to see the huge gold ball

Figure 12.52 c

For very high winds, and for significant earthquakes, the swinging of the pendulum is restrained at the bottom of the sphere by a large steel pin 60 cm (2 ft) in diameter that engages piston dampers in a surrounding restraint ring. The pendulum system offsets much of the wind-induced motion that would otherwise be experienced by occupants

For the Taipei 101 Russ Thomas, director of the council's Fire Risk Management Program, after visiting the construction site said that, "The building is safe with the damper in it. With the damper, people will feel more comfortable when working inside."

12.6 CONCLUSIONS

These days with increasing awareness about seismic safety in buildings, people living in seismic zones are building on the basis of performance than just meeting the “Life-Safety", or "Minimum Code Design”.
Various architectural projects are discussed in the chapter from all over the world. All the projects have employed very innovative measures for seismic safety of structures. This proves that the incorporating seismic safety measures in the design do not restrict the architect’s ability to create great designs, but make the structures imperishable.

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ANNEXURE I

COMPREHENSIVE INTENSITY SCALE (MSK 64)*

The Scale was discussed generally at the inter government meeting convened by UNESCO in April 1964. Through not finally approved the scale is more comprehensive and describes the intensity of earthquake more precisely. The main definitions used are followings:

A) Type of Structures (Buildings)

Type A	Building in field-stone, rural structures, un burnt-brick houses, clay houses.
Type B	Ordinary brick buildings, buildings of large block and prefabricated type, half timbered structures, buildings in natural hewn stone.
Type C	Reinforced buildings, well built wooden structures

B) Definition of Quantity

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

C) Classification of Damage to Buildings

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

D) Intensity Scale

1. Not noticeable – The intensity of the vibration is below the limits of sensibility: the tremor is detected and recorded by seismograph only.
2. Scarcely noticeable (very slight) – Vibration is felt only by individual people at rest in houses, specially on upper floors of buildings.
3. Weak, partially observed only – The earthquake is felt indoors by a few people, outdoors only in favourable circumstances. The vibration is like that due to the passing of a light truck. Attentive observers notice a slight swinging of hanging objects. Somewhat more heavily on upper floors.
4. Largely observed – The earthquake is felt indoors by many people, outdoors by few. Here and there people awake, but no one is frightened. The vibration is like that due to the passing of a heavily loaded truck. Windows, doors, and dishes rattle. Floors and Walls crack. Furniture begins to shake.

* **Source IS 1893 (Part 1) : 2002, Appendix D**

Hanging objects swing slightly. Liquid in open Vessels are slightly disturbed. In standing motor cars the shock is noticeable.

5. Awakening

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

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

iii) Sometimes changes in flow of springs.

6. Frightening

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

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

iii) In few cases, cracks up to widths of 1 cm possible in wet ground; in mountains occasional landslips; change in flow of springs and in level of well water are observed.

7. Damage of Buildings

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

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

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

8. Destruction of Buildings

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

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

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

9. General Damage of Buildings

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

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

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

10. General Destruction of Buildings

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

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

11. Destruction

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

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

12. Landscape Charges

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

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

ANNEXURE II

RAPID VISUAL SCREENING OF BUILDINGS IN VARIOUS SEISMIC ZONES IN INDIA*

Basis of the Methodology

The methodology proposed here below is based on the classification of buildings as per MSK Intensity scale as well as the new European Intensity scale, modified by the author to some extent based on his experience of buildings in India. The Grades of damage are also based on the two Intensity scales taken together. The relationship of the MSK Intensities adopted in IS: 1893-2002 (Part 1) and that adopted in the European scale have been studied and made use of in developing the table of damage grades of various building types under Intensities VI to IX. Based on this table the rapid visual screening of buildings in various seismic zones has been arrived at.

1. Seismic Zones in India (18:1893-2002, Part 1)

Zone V - MSK Intensity IX or higher (Destructive or Very Destructive intensities)
Zone IV - MSK Intensity VIII (Heavily damaging intensity)
Zone III - MSK Intensity VII (Damaging intensity)
Zone II - MSK Intensity VI or lower (Slightly damaging or no damage intensities)
Note: In a zone of higher intensity occurrence, lower intensities will occur around higher intensity area.

2. Building Types in India

From the damage vulnerability consideration the buildings can be classified as follows :
2.1 Masonry load bearing wall buildings

Building Type	Description
A	Rubble (Field stone) in mud mortar or earthen walls
A+	As above but one storey only having light roof
B	Semi-dressed, rubble, brought to courses, with through stones and long corner stones; unreinforced brick walls with country type wooden roofs; unreinforced CC block walls
B+	As above of only single storeys and/ or better quality of construction
C	Fully dressed (ashler) stone masonry or CC block or burnt brick walls built using good lime or cement mortar.Unreinforced walls but having RC floor/ roof.
C+	As at C but having horizontal RC bands (IS: 4329, 13828).
D	Masonry construction as at C but reinforced with bands & vertical reinforcement, etc (IS: 4329), or confined masonry using horizontal & vertical reinforcing of walls.

Note: In rural areas, there are huts or shacks made from bio-mass & metal sheets etc. Their vulnerability to earthquakes is very very low.

*** Developed by Dr. A. S. Arya, Professor Emeritus, Department of Earthquake Engineering, Indian Institute of Technology Roorkee**

2.2 Reinforced Concrete Frame Buildings (RCF) and Steel Frames (SF)

Frame Type	Description
C	RCF without ERD or WRD, built in non-engineered way; RCF with hollow plinth (open ground storey); SF without bracings having hinge joints; RCF of ordinary design without ERD or WRD, SF of ordinary design without ERD or WRD
C+	MR-RCF/MR-SF of ordinary design without ERD or WRD
D	MR-RCF with ordinary ERD without special details as per IS: 13920, with ordinary infill walls (such walls may fail earlier similar to C in masonry buildings; MR-SF with ordinary ERD without special details as per plastic design hand book SP:6(6)-1972.
E	MR-RCF with high level of ERD as per IS: 1893-2002 & special details as per IS: 13920 MR-SF with high level of ERD as per IS: 1893-2002 & special details as per Plastic design hand book, SP:6(6)-1972
E+	MR-RCF as at E with well designed infills walls MR-SF as at E with well designed braces
F	MR-RCF as at E with well designed & detailed RC shear walls MR-SF as at E with well designed & detailed steel braces & cladding; MR-RCF/MR-SF with well designed base isolation.

Notes: RCF = Reinforced concrete column- beam frame system
SF = Steel column- beam frame system
ERD = Earthquake Resistant Design
WRD = Wind Resistant Design
MR = Moment Resistant jointed frame

3. Grades of Damage to Buildings

Classification of damage to masonry buildings

Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)
Hair-line cracks in very few walls. Fall of small pieces of plaster only.
Fall of loose stones from upper parts of buildings in very few cases.

Grade 2: Moderate damage (Slight structural damage, moderate non-structural damage)
Cracks in many walls.
Fall of fairly large pieces of plaster.
Partial collapse of smoke chimneys on roofs.

Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)
Large & extensive cracks in most walls.
Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).

Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)
Serious failure of walls (gaps in walls), inner walls collapse; partial structural failure of roofs & floors.

Grade 5: Destruction (very heavy structural damage)
Total or near total collapse of the building.

Classification of damage to buildings of reinforced concrete

Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)
Fine cracks in plaster over frame members or in walls at the base.
Fine cracks in partitions & infills.

Grade 2: Moderate damage (Slight structural damage, moderate non-structural damage)
Cracks in columns & beams of frames & in structural walls.
Cracks in partition & infill walls; fall of brittle cladding & plaster. Falling mortar from the joints of wall panels.

Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)
Cracks in columns & beam column joints of frames at the base & at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods.
Large cracks in partition & infill walls, failure of individual infill panels

Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)
Large cracks in structural elements with compression failure of concrete & fracture of rebars; bond failure of beam reinforcing bars; tilting of columns. Collapse of a few columns or of a single upper floor.

Grade 5: Destruction (very heavy structural damage)
Collapse of ground floor parts (eg. Wings) of the building

4. Relationship of Seismic Intensity, Building Type & Damage Grades

Few : Less than(15±5)%; Many: Between(15±5) to(55±5)%;
Most: Between (55±5) tolOO%

	Type of Building	Zone II MSK VI or less	Zone HI MSK VII	Zone IV MSK VIII	ZoneV MSK IX or More
MASONRY BUILDINGS	A and A+	Many of grade 1 Few of grade 2 (rest no damage)	Most of grade 3 Few of grade 4 (rest of grade 2 or1)	Most of grade 4 Few of grade 5 (rest of grade 3,2)	Many of grade 5 (rest of grade 4 &3)
	B and B+	Many of grade 1 Few of grade 2 (rest no damage)	Many of grade 2 Few of grade 3 (rest of grade 1)	Most of grade 3 Few of grade 4 (rest of grade 2)	Many of grade 4 Few of grade 5 (rest of grade 3)
	C and C+	Few of grade 1 (rest no damage)	Many of grade 1 Few of grade 2 (rest of grade 1,0)	Most of grade 2 Few of grade 3 (rest of grade 1)	Many of grade 3 Few of grade 4 (rest of grade 2)
	D		Few of grade 1	Few of grade 2	Many of grade 2 Few of grade 3 (rest of grade 1)
RCF / SF BUILDINGS	C and C+	Few of grade 1 (rest no damage)	Few of grade 2 (rest of grade 1,0)	Many of grade 2 Few of grade 3 (rest of grade 1)	Many of grade 3 Few of grade 4 (rest of grade 2)
	D	-	Few of grade 1	Few of grade 2	Many of grade 2 Few of grade 3 (rest of grade 1)
	E and E+	-	-	-	Few of grade 2 (rest of grade 1 orO)
	F				Few of grade 1

Rapid Visual Screening of Indian Buildings for Potential Seismic Hazards

Seismic Zone

Elevation to Scale	Building Name
	Use
	Address:
	Pin
	Other Identifiers
	No. Stories Year Built
	Total Floor Area (sq.m)
Plan to Scale	PHOTOGRAPH

OCCUPANCY		SITE	FALLING HAZARDS
/?es/:Ord/lmp. School Health ssembly Office Commercial Historic Emer. Service Industrial	Max. Number of Persons 0-10 11-50 51-100 >10< Residents Floating	High W.T. (within 3 m) Liquefiable (if sandy soil) Land Slide Prone	D D D D Chimneys Parapets Cladding Other

<u>Probable Maximum Grade of Damage</u>

[Building Type	Masonry Building				RC or Steel Frame Building				URM infill	Wood
	A,A+	B,B+	C,C+	D	C,C+	D	E,E+	F		
Damage grade in Zone II	G2	G2	Gl	.	Gl	.	.	.	01	.

Note: i sign indicates higher strength hence somewhat lower damage expected than that stated. Also average damage in one building type in the area may be lower by one grade point than the probable maximum indicated.

Surveyor will identify the Building Type, encircle it, also the corresponding damage grade.

Recommended Action : Surveyor's & Signature _______

Name

Date

1) Ensure adequate maintenance

ANNEXURE III

VULNERABILITY ASSESSMENT OF EARTHQUAKE DAMAGED BUILDINGS

1 . TOWN / VILLAGE (NAME)	BUILDING PLAN & ELEVATION (SCHEMATIC)
2. BUILDING IDENTIFICATION	
3. POSITION BUILDING IN THE BLOCK a. CORNER b. MIDDLE c. FREE	
4. NUMBER OF STOREYS 4. 1 STOREYS 4.2 APPENDAGES 4.3 MEZZANINES 4.4 BASEMENTS	
5. GROSS AREA OF THE BUILDING (m')	
6. USAGE	
7. NUMBER OF APARTMENTS:	
8. CONSTRUCTION PERIOD	
9. TYPE OF THE STRUCTURE	
10. FLOORS: RC STEEL WOOD OTHER	
11. ROOF: RC STEEL WOOD OTHER	
12. QUALITY OF WORKMANSHIP: GOOD AVERAGE POOR	
13. TYPE OF LOAD CARRYING SYSTEM a. BEARING WALLS b. FRAMES c. FRAMES WITH INFILL WALLS d. MIXED e. OTHER (SPECIFY)	
14. FIRST STOREY-STIFFNESS RELATIVE TO OTHERS: a. LARGER b. ABOUT EQUAL C.. SMALLER	
15. REPAIRS FROM PREVIOUS EARTHQUAKES:	

16. DEGREE OF DAMAGE IN STRUCTURAL ELEMENTS
a. NONE b. SLIGHT c. MODERATE d. HEAVY e. SEVERE

16.1 BEARING WALLS:		16.2 COLUMNS:	
16. 3 BEAMS:		16.4 FRAME JOINTS:	
16.5 SHEAR WALLS:		16.6 STAIRS:	
16. 7 FLOORS:		16.8 ROOF:	

17. DEGREE OF DAMAGE IN NONSTRUCTURAL ELEMENTS AND INSTALLATIONS
a. NONE   b. SLIGHT   c. MODERATE   d. HEAVY   e. SEVERE

17.1 INTERIOR WALLS:		17.2 PARTITIONS:	
17.3 EXTERIOR WALLS:		17.4 ELECTRICAL INSTALLATIONS:	
17. 5 PLUMBING:			

18. DAMAGE OF ENTIRE BUILDING a. NONE, d. HEAVY

b. SLIGHT   c. MODERATE
e. SEVERE   f. TOTAL

19. INDIRECT DAMAGE (FIRE, SLAMMING, ETC.)	*NO*	*YES*

20. OBSERVED SOIL INSTABILITIES AND GEOLOGICAL PROBLEMS
a. NONE   b. SLIGHT SETTLEMENTS   c. INTENSIVE SETTLEMENTS   d. LIQUEFACTION
e. LANDSLIDE   f. ROCK FALLS   g. FAULTING   h. OTHER (SPECIFY):

21. USABILITY CLASSIFICATION AND POSTING
POSTED: ______a. GREEN   b. YELLOW   c. RED
NOT POSTED: ______d. JO BE POSTED GREEN AFTER REMOVAL OF LOCAL HAZARD
e. SOIL AND GEOLOGICAL PROBLEMS, RE-INSPECTION REQUIRED
f. UNABLE TO CLASSIFY, RE-INSPECTION NECESSARY   g. BUILDING INACCESSIBLE

GREEN -—> ORIGINAL SEISMIC CAPACITY HAS NOT BEEN DECREASED —> UNLIMITED USAGE
YELLOW—> ORIGINAL SEISMIC CAPACITY HAS BEEN DECREASED —> TEMPORARILY UNUSABLE —-> LIMITED ENTRY
RED —> BUILDING DANGEROUS AS SUBJECT TO SUDDEN COLLAPSE —> ENTRY PROHIBITED

MAIN REASONS FOR YOUR CLASSIFICATION AND POSTING

22. RECOMMENDATIONS FOR EMERGENCY MEASURES
a. NONE   b. REMOVE LOCAL HAZARD   c. PROTECT BUILDING FROM FAILURE
c. PROTECT BUILDING FROM FAILURE   d. URGENT DEMOLITION

23. ADDITIONAL DATA (ATTACH PHOTO /SKETCHES AND COMMENTS)

24. ESTIMATED PRESENT VALUE OF BUILDING
25. ESTIMATED LOSS (% OF ESTIMATED VALUE)
26. HUMAN LOSSES (DEATHS AND INJURIES
a. NO   b. POSSIBLY   c. YES

IF INFORMATION AVAILABLE PLEASE INDICATE

NO. OF DEATHS   NO. OF INJURIES

27. DATE OF INSPECTION:

SIGNATURES

NAME OF INSPECTION ENGINEERS

ANNEXURE IV

CHECKLIST FOR SIMPLIFIED SEISMIC VULNERABILTY ASSESSMENT OF EXISTING BUILDINGS

Building Identification

GENERAL INFORMATION

Building name:_
Ownership: *Public* *Private*
Owner name: _
Address:

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1. Building type *Stone in mud mortar D Brick masonry in mud mortar RCC Frame Building D*
Earthen building f_]
Stone in cement mortar D
Brick masonry in cement mortar Q
RCC Frame-shear wall Building Q
Mixed construction (specify): ____

2. Usage of the building: ResidentiaO Business D *Offices* *PublicG StorageD*
Others (specify): ______________________

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3. Number of occupants (approximately):
4. Building location: IsolateoD Internal Q EndQ CornerQ
5. Plan dimensions:
6. Total number of stories:
7. Average inter-storey height:
8. Construction age:
9. Building construction quality: Goodrj Average G Poor D
10. Building condition and maintenance: Good D Average CD PoorD
11. Building site located at *Plain D River bed D*
Hill top D *High slope of hill* *Mild slope I*
Others (specify):__________

12. Soil type
Rock / Hard soil D

Medium soil D Soft soil D
Reclaimed/filled land d Partially filled land D Loose sand d Others *(Specify):*
____

13. Ground Water Table depth ________________
14. Gap between adjacent buildings:
15. Plan of building: Symmetric d Asymmetricd
16. Reentrant corner is: Less than 15% d more than 15%d
17. Regularity in elevation: Regulard Irregular d
If irregular, approximate Shape in elevation is of the type
Percentage reduction/increase in dimensions:
18. Location of staircase in building is: Symmetric d Asymmetric d
19. Staircase is: Separated from main structured] Connected to main structured
If connected to main structure, it is
Enclosed by rigid walls d Not enclosed by rigid walls D
20. Basement provided: Yes Q No d
21. Overhang length: __________________________
22 Parapet height: _______
Secured against falling Yes d No d

MASONRY BUILDING

1. Type of construction: Engineered construction d   Non-Engineered construction d
2. Layout of masonry *(Tick in case of stone masonry)*

Random rubbled   Semi-dressed stond   Ashlar masonry d

3. Through stones *(in case of random rubble masonry):* Providedd   Not provided d
4. Earthquake resistant features provided in the building

Lintel band d   Roof band d   Plinth bandd

Vertical reinforcement at Corners d   Junctions d   Jambs of openings d

5. Minimum pier dimensions: _______________________
6. Existence of floating walls: Yes d   No d

7. Foundation type: Strip foundation D   Raft foundationD   Pile foundation D Others (specify): _________________________

8. Foundation material used for construction

StoneD   Brick D   Cement concrete D

RCCD

9. Percentage openings in main walls: _______

10. Thickness of Exterior wall _____mm   Interior wall _____mm

11. Roof type: Flat roof D   Sloping roof D   Hipped Roof D

12. If sloping provided, then

Ties provided: Yes D   No D ;   Bracing provided: Yes D   No D

Roof and Gable band provided: Yes D   No D

13. Roofing material used

RCC slab D   Tiles D   Corrugated iron sheeting D   Asbestos sheeting D

Others, specify ______

RCC BUILDING

1. Irregularities in structure Open ground floor D Heavy mass at roof D Any other (Specify): ____   *Floating columns D   Mezzanine floor D Floating shear walls*   *Partially filled panels D*

2. Designed by :   Architect d   Structural engineer I

3. Enhanced ductility design: Yes fj   NoD

Mason D

4. Column size at ground floor: ___________

5. Beam size: ______________________

6. Spans between columns: _____________

7. Shear wall(s) provided: Yes D   No I—I

If provided,   Symmetrically D   asymmetrically D

Thickness of shear wall __________

8. Infill type :   Brick masonry D   Stone masonry Q   Solid concrete blocks

Hollow concrete blocks D   Timber D   Any other (Specify):

*Thickness of Infill: Exterior*________   *Interior*

9. Basement Provided: Yes D

No D

ANNEXURE V

List of BIS Codes

•IS: 456:2000 "Code of Practice for Plain and Reinforced Concrete
•IS: 800-1984 "Code of Practice for General Construction in Steel
•IS:801-1975 "Code of Practice for Use of Cold Formal Light Gauge Steel Structural Members in General Building Construction
•IS:875 (Part 2):1987Design loads (other than Eq.) for buildings &structures Part2 Imposed Loads
•IS:875 (Part 3):1987Design loads (other than Eq.) for buildings and structures Part 3 Wind Loads•IS:875 (Part 4):1987Design loads (other than Eq.) for buildings and structures Part 4 Snow Loads•IS:875 (Part 5):1987Design loads (other than Eq.) for buildings and structures Part 5 Special loads and load combination
•IS:883:1966 "Code of Practice for Design of Structural Timber in Building
•IS:1904:1987 "Code of Practice for Structural Safety of Buildings: Foundation"
•1S:1905:1987 "Code of Practice for Structural Safety of Buildings: Masonry
•IS 2911 (Part 1): Section 1: 1979 "Code of Practice for Design and Construction of Pile Foundation Section 1

Part 1: Section 2 Based Cast-in-situ Piles
Part 1: Section 3 Driven Precast Concrete Piles
Part 1: Section 4 Based precast Concrete Piles
Part 2: Timber Piles
Part 3: Under Reamed Piles
Part 4: Load Test on Piles IS: 875 (3)-1987 "Code of Practice for Design Loads (other than Earthquake) for Buildings and Structures, Part 3, Wind Loads"

Guidelines (Based on IS 875 (3)-1987) for improving the Cyclonic Resistance of Low rise houses and other building

•IS:1893-2002 "Criteria for Earthquake Resistant Design of Structures (Fifth Revision)"
•1S:13920-1993 "Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces - Code of Practice"
•1S:4326-1993 "Earthquake Resistant Design & Construction of Buildings - Code of Practice (Second Revision)"
•1S:13828-1993 "Improving Earthquake Resistance of Low Strength Masonry Buildings - Guidelines"
•1S:13827-1993 "Improving Earthquake Resistance of Earthen Buildings- Guidelines",
•1S:13935-1993 "Repair and Seismic Strengthening of Buildings - Guidelines"•IS:14458 (Part 1): 1998 Guidelines for retaining wall for hill area: Part 1 Selection of type of wall.
•IS:14458 (Part 2): 1997 Guidelines for retaining wall for hill area: Part 2 Design of retaining/breast walls
•IS:14458 (Part 3): 1998 Guidelines for retaining wall for hill area: Part 3 Construction of dry stone walls
•IS:14496 (Part 2): 1998 Guidelines for preparation of landslide – Hazard zonation maps in mountainous terrains.