HANDBOOK ON
SEISMIC RETROFIT OF BUILDINGS

(April 2007)

(DRAFT FOR COMMENTS)

CENTRAL PUBLIC WORKS DEPARTMENT

&

INDIAN BUILDING CONGRESS

In association with

INDIAN INSTITUTE OF TECHNOLOGY - MADRAS

CONTENTS

PREFACE

Earthquakes are one of nature's greatest hazards to life on this planet. The impact of this phenomenon is sudden with little or no warning to make preparations against damages and collapse of buildings/structures. The hazard to life in case of earthquake is almost entirely associated with man made structures such as buildings, dams, bridges etc.

Prevention of disasters caused by earthquake has become increasingly important in recent years. Disaster prevention includes the reduction of seismic risk through retrofitting existing buildings in order to meet seismic safety requirements. The planning of alterations to existing buildings differs from new planning through an important condition; the existing construction must be taken as the basis for all planning and building actions. The new structure can be built sufficiently earthquake resistant by adoption proper design methodology and construction quality control. But the existing old structures which have mostly been planned without considering this important aspect, pose enormous seismic risk, in particular to human life and historical monuments.

India is one of the most earthquake prone countries in the world and has experienced several major/moderate earthquakes during the last 15 years. About 50-60 % of the total area of the country is vulnerable to seismic activity of varying intensities. The earthquakes at Latur (1993), Jabalpur (1997), Chamoli (1999) and Bhuj (2001) had exposed the vulnerability of buildings in India. The codes of practice on earthquake resistant design (IS 4326:1993), earthquake resistance of earthen buildings (IS 13827:1993), earthquake resistance of low strength masonry buildings (IS 13828:1993), ductile detailing of reinforced concrete structures (IS 13920:1993) and seismic strengthening of buildings (IS 13935) were published almost simultaneously to meet the urgency of seismic design of buildings. IS 1893 code of practice for earthquake resistant design and construction of buildings was first prepared in 1962 and subsequently revised in 1966, 1970, 1975, 1984 and the fifth revision was published recently in 2002. Part 1 and other parts of this code are under revision.

Many existing buildings do not meet the seismic strength requirement. The need for seismic retrofitting in existing building can arise due to any of the following reasons: (1) building not designed to code (2) subsequent updating of code and design practice (3) subsequent upgrading of seismic zone (4) deterioration of strength and aging (5) modification of existing structure (6) change in use of the building, etc. In India, almost 85% of total buildings are non-engineered buildings made up of earthen walls, stone walls, brick masonry walls, etc. These buildings are more vulnerable, and in the event of a major earthquake, there is likely to be substantial loss of lives and property.

This Handbook has been envisaged to address this serious and widespread problem of seismic vulnerability of buildings. The Handbook covers the seismic retrofitting of both engineered and non-engineered buildings.

For evaluation of the vulnerability of any building, some rapid visual methods and preliminary evaluation are to be carried out first. On the basis of this study, one can arrive at a conclusion as to whether the building is safe or needs further detailed evaluation to assess its adequacy. This Handbook is intended to be user-friendly, catering to not only the practising engineer but also to a common man who can broadly understand and note the absence or presence of seismic-resistant features in the building and also the possibilities of seismic retrofit. The owner of the building will also appreciate when he or she needs to consult a structural consultant for detailed evaluation and retrofit measures that need technical know-how.

Seismic retrofit is primarily applied to achieve public safety, with various levels of structure and material survivability determined by economic considerations. In recent years, an increased urgency has been felt to strengthen the deficient buildings, as part of pro-active disaster mitigation, and to work out the modifications that may be made to an existing structure to improve the structural performance during an earthquake. However, it is not intended by CPWD and India Building Congress to prepare this Handbook as guidelines for comprehensive "how-to" manual for full-fledged retrofitting of structures, as that is best to be done by engineering specialists. There are some tasks, such as enhancing the integrity of a masonry building that can be taken up by the initiative of the home-owners. In other cases, such as multiple level houses, buildings built upon poor foundations, and those built upon questionable soil conditions, a specialized and licensed expert should be consulted. When retrofitting an existing building or designing and constructing a new, custom-built structure, one should be cautions about possible transfer of forces to unmodified portions. The guidelines given in this Handbook are more to give a general sense of safe/unsafe nature of the existing building/structure so that owners can take further measures to prevent loss of life and property.

This Handbook gives adequate details for academicians, professionals, builders and owners to take a few major and positive steps to ensure that structures can be made adequately safe/ductile to save loss of life and property in case of earthquakes.

Although, there are a number of publications available on the subject of seismic strengthening outside India, such as FEMA and ASCE, these are not directly suitable in the

Indian context. Considering this in mind, the Central Public Works Department (CPWD), which is a leading Government Organization engaged in construction and maintenance of building throughout the length and breath of the country and the Indian Building Congress (IBC) conceived the idea of preparing a comprehensive Handbook on ‘Seismic Retrofitting of Buildings’ covering various topics related to seismic retrofit. CPWD is pleased to associate the Indian Institute of Technology, Madras, in this endeavour and the whole project is funded by CPWD.

It is the sincere desire of CPWD and also of IBC that this Handbook, which covers all features of assessment of the need and methods of retrofitting of existing structures, will be of immense use to the student community, academicians, consultants, practicing professional engineers/scientists involved in planning, design, execution, inspection and supervision for proper retrofitting of buildings, and also lastly the owners. We, as professionals, believe that a Handbook is only a guide and it is the expert who will have to take final decision about the actual extent of work to be done, but an attempt has been made to cover all aspects. Feedback from all after perusal and using this Handbook will be highly appreciated by Central PWD and IIT Madras for its future updation. As professionals, we feel all documents have to be dynamic.

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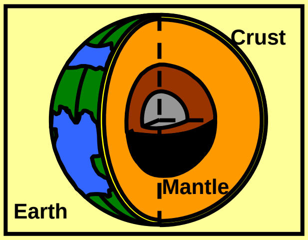
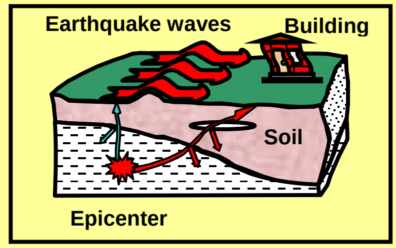
How to make your building earthquake-safe

How does an earthquake occur?



The inside of our earth consists of many layers (*crust*, *mantle*, *inner* and *outer cores*). Formed by complex processes over countless years, they continue to be active. Once in a while, the disturbances below the earth get transmitted to the surface, causing earthquakes.

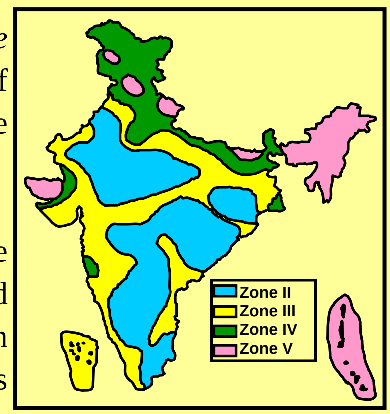


The *waves* generated in the soil during an earthquake travel long distances in many directions in a very short time, shaking the ground. The buildings that cannot resist this ground shaking can collapse, causing disaster and loss of human life.

Earthquakes are natural occurrences

Earthquake hazard in India

Based on historical occurrences, regions in India are classified into *low*, *moderate*, *severe* and *very severe* earthquake-prone zones (refer seismic zone map of India). More than half of the country's population lives in moderate to very severe regions, where high-magnitude earthquakes can occur.



The extent of damage to a building during an earthquake depends not only on the magnitude of the earthquake, but also on the soil, building configuration, quality of design and construction. In developed countries, because of better awareness and regulation of design and construction practices, the buildings survive earthquakes and damage and loss of life is less. India should also achieve this standard.

Earthquakes do not kill; unsafe buildings do!

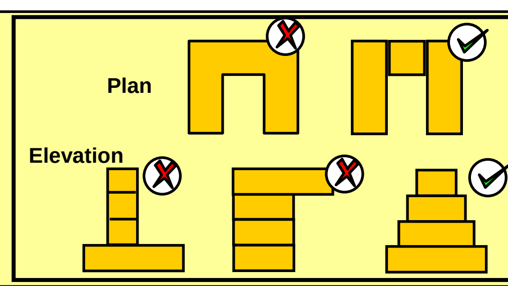
Is your building safe?

Consult a competent engineer if you have doubts about your building. Get it assessed and, if found deficient, get it suitably retrofitted. Information given here will give you some idea on what makes a building unsafe and how it can be retrofitted. ***Note that mere patchwork is not structural retrofit,*** and this will be exposed by an earthquake.

Also, if you are planning to invest in a new building, make sure that the builder provides the required earthquake resistant features. Use the information given here to ask the builder pointed questions.

Prevention is better than cure!

Importance of building configuration



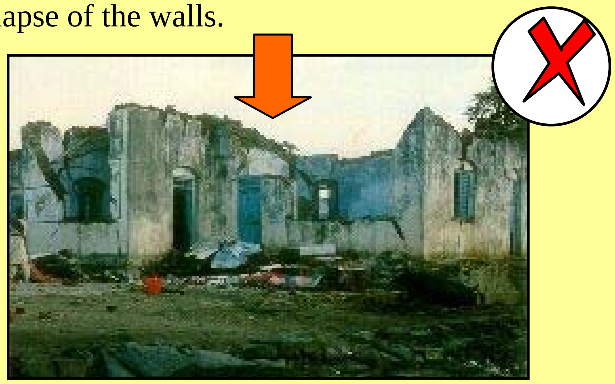
The building configuration should be simple and regular in plan (as you see from top) and elevation (as you see from the front or side). Otherwise, the building becomes highly vulnerable.

Good configuration is more important than fancy looks!

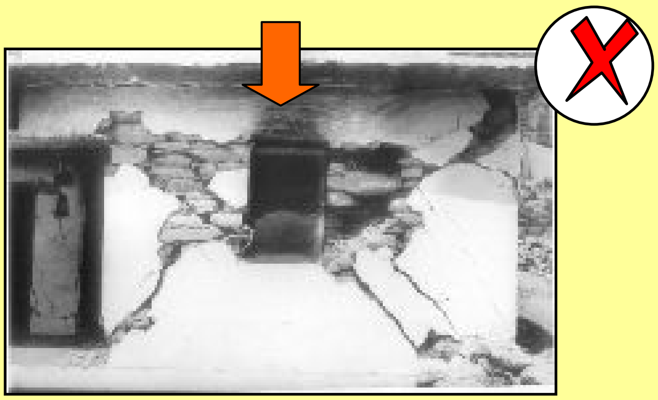
Masonry Buildings Do's and Don'ts

Deficiencies

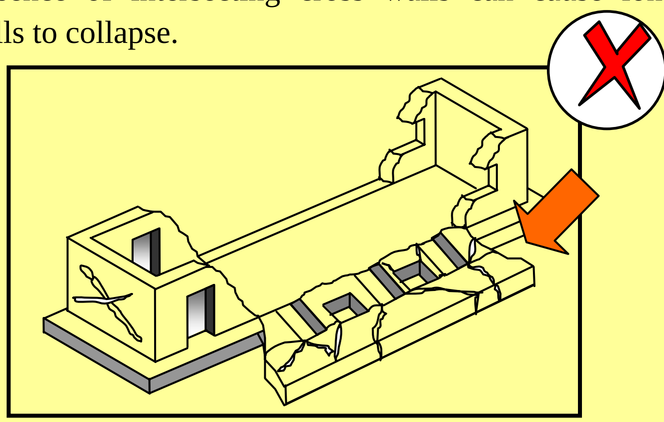
- Absence of plinth, sill, lintel and roof bands causes collapse of the walls.



- Absence of vertical steel bars at corners and around openings causes extensive 'X' type cracking.



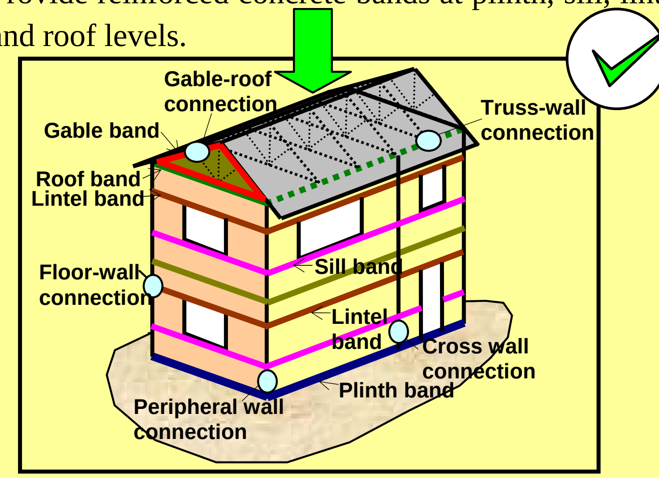
- Absence of intersecting cross walls can cause long walls to collapse.



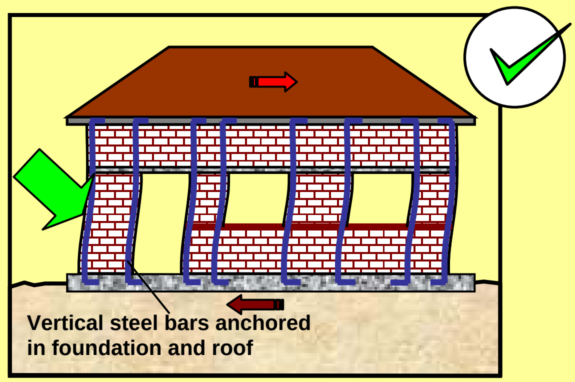
- Corners of walls collapse due to high stresses and lack of integrity.

Correct Design / Remedial Measures

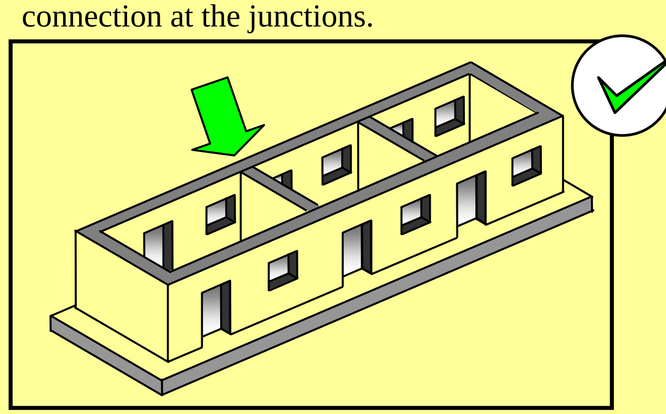
- Provide reinforced concrete bands at plinth, sill, lintel and roof levels.



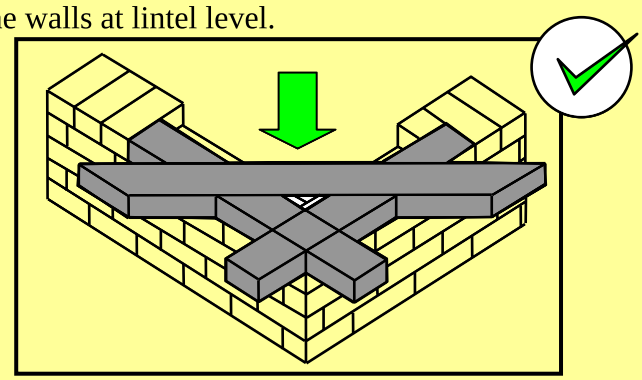
- Provide vertical steel bars at corners of wall segments and between openings to improve seismic resistance.



- Provide adequate cross walls, with proper connection at the junctions.



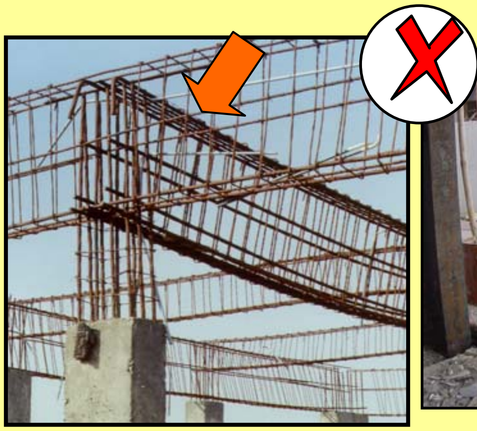
- Provide collar bands (wooden or RC) at the corners of the walls at lintel level.



Concrete Buildings Do's and Don'ts

Deficiencies

- Inadequate frames (consisting of beams, columns and footing with proper joints) in both directions.
- Poor detailing of reinforcing bars, especially at joints.



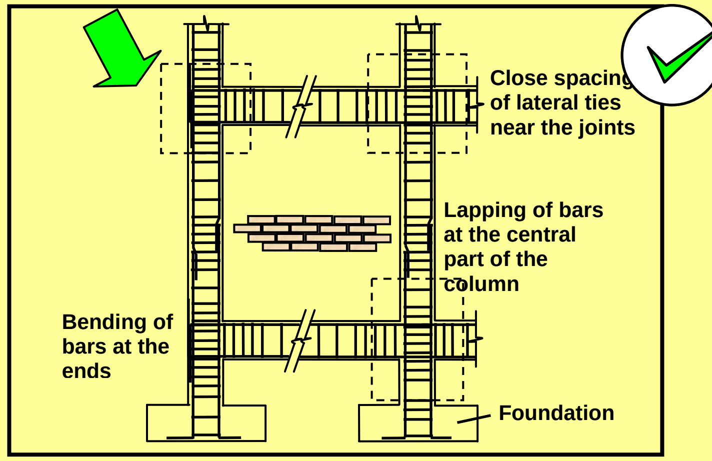
- A ground storey without walls (for car parking) can cave in.

- Weak beams and columns without proper reinforcing bars and stirrups; bar lapping at floor levels.

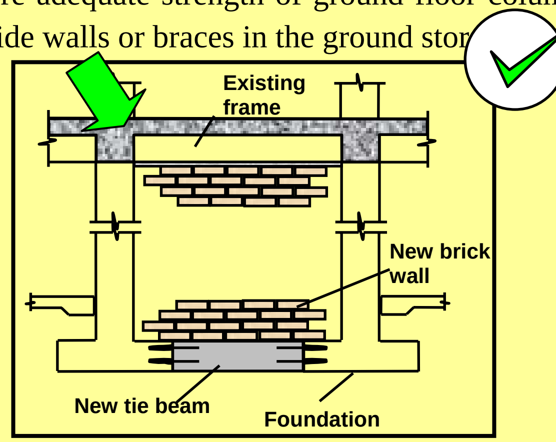
- Staircases are often the first to collapse, blocking escape from the building.

Correct Design / Remedial Measures

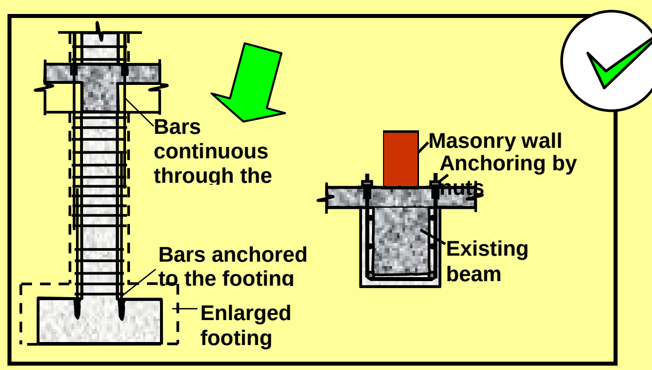
- Framed buildings should be specially designed to resist earthquake loads as per the code.
- Provide detailing of reinforcing bars as per the code.



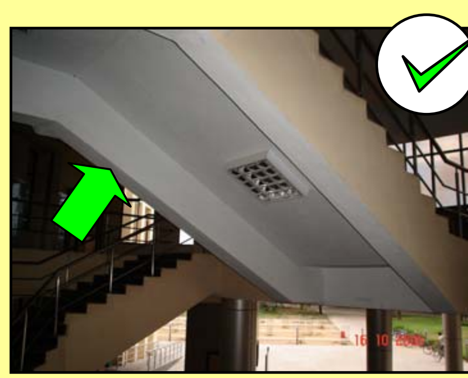
- Ensure adequate strength of ground floor columns or provide walls or braces in the ground stor



- Proper strengthen the footing, columns and beams.

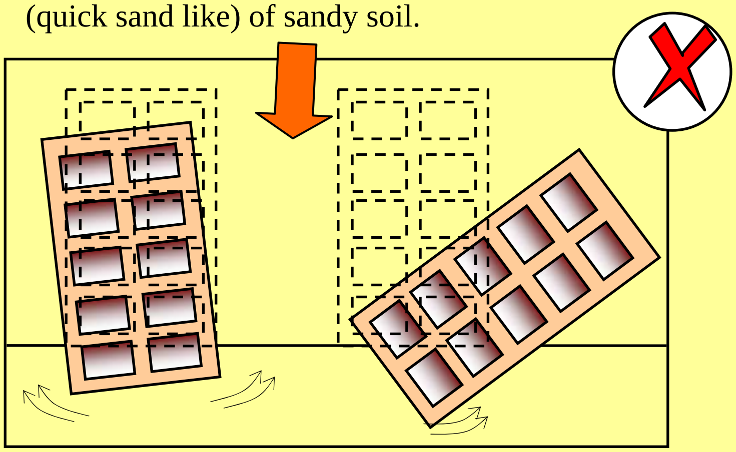


- Stair slabs connected to inclined beams framing into columns provide integrity to survive the earthquake.



Other Issues

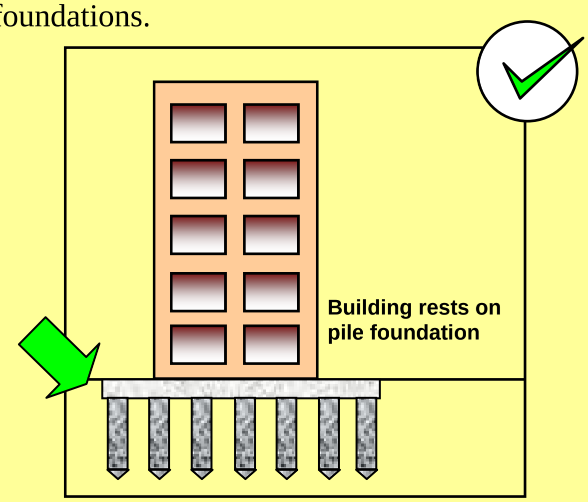
- Tilting or overturning of the building due to liquefaction (quick sand like) of sandy soil.



- Increased vibrations in soft soils (site amplification).
- Collapse of parapet walls and projections.
- Collapse of frames with partially infilled masonry walls or with short columns.
- Improperly fitted ceilings, light fittings and heavy and tall furniture.

Correct Design / Remedial Measures

- Consolidation by injection, by providing pile foundations.



- Retrofit building for higher forces
- Anchoring the projections to the adjoining strong body.
- Separate such infill walls from column or strengthen columns
- Fasten such items firmly to the structure.

Key questions to ask to the builder before buying a house

- Is the configuration of the building simple and regular?
- What special features have been provided for seismic resistance in a one- or two-storeyed house?
  - Plinth bands? Sill bands? Lintel bands? Gable and roof bands?
- Are large overhanging projections (balconies) avoided? If not, have they been specially designed as per code?
- For a building with more than two storeys, whether the structure was designed for earthquake resistance as per the prevailing codes and proof-checked for compliance with ***National Building Code*** of India?
- If the building has ground storey without walls, have the columns been specially designed as per the code?
- For a concrete building in seismic zones III and above, have the special detailing requirements been incorporated?
- Have the stair slabs been integrally connected to the frames to prevent their possible collapse?
- Was a proper soil study conducted for the site?
- Were the potentials for soil liquefaction / slope failure properly assessed?

Additional questions to ask before buying an old house

- Is there any sign of deterioration, such as cracks, corrosion stains, tilting of walls?
- Have additional storeys been added after initial construction?
- Has any major repair work been carried out recently?
- If any of the above is true or the quality of construction appears to be poor, consult an expert.

References: 1. *Handbook of Seismic Retrofit* (2007) 2. *National Building Code* of India (2005) 3. *Earthquake Tips* (2005)

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MAKING BUILDINGS SAFE AGAINST EARTHQUAKES

0.1 PREAMBLE

This introductory chapter gives a simple overview of the main aspects of the *Handbook on Seismic Retrofit of Buildings*, and is meant especially for the lay reader — the common man who is not expected to have a background in the design of buildings.

Is your building safe against earthquakes? This is a question that everybody should be concerned about. Many earthquakes have taken place recently in India and its neighbourhood [e.g., Uttarkashi (1991), Latur (1993), Jabalpur (1997), Bhuj (2001), Sumatra (2004), Kashmir (2005)], and these have proved to be disastrous. A large number of buildings have collapsed and countless lives have been lost.

There is usually a huge public outcry after every seismic disaster. But public memory is short-lived, and people tend to forget, until another major tremor occurs to wake them up. When will that quake happen in our neighbourhood? Nobody can predict this. But when it comes one day, unexpectedly, it could destroy our buildings and take our lives. Can we do something to save our lives and protect our buildings? Yes!

Because earthquakes do not kill; unsafe buildings do.

This introductory chapter provides basic education on earthquake safety of buildings. Find out how safe your existing building against earthquakes, by getting it assessed for seismic safety. Understand the risk involved and the necessary action to be taken. If the building is unsafe and the strengthening of the building is financially viable, get it done. It is an insurance worth having. The strengthening of a building for earthquakes is referred to as seismic retrofitting. The material in this chapter will give you some ideas on what may make your building unsafe to earthquakes and how it could be retrofitted.

Also, if you are planning to invest in a new building, the information given in this chapter will enable you to identify and demand the desirable earthquake resistant features in the building. You will know what questions to pose to your builder. You will realize the importance of getting multi-storeyed buildings (such as apartment blocks) proof-checked and certified for compliance with the prevailing design standards, so as to make sure that your investment and life are reasonably safe.

Prevention is better than cure.

0.2 FACTS YOU NEED TO KNOW ABOUT EARTHQUAKES

0.2.1 Why Does the Earth Quake?

An earthquake is a natural phenomenon associated with violent shaking of the ground. To understand why the earth quakes, it is necessary to know something about the interior of the earth. The earth is not the simple solid sphere it appears to be on the surface. It is made up of several layers (crust, mantle, outer core, inner core), some of which are far from solid (Figure 0.1). The outermost layer (soil and rock), called the *crust* of the earth (thickness ~5 to 40 km), is supported on a layer called the *mantle* (thickness ~2900 km), which is fluid in nature, and which in turn is supported on a *core*. The outer portion of the core (thickness ~2200 km) is made of a very hot and dense liquid, while the inner core (radius ~1290 km) is solid and metallic.

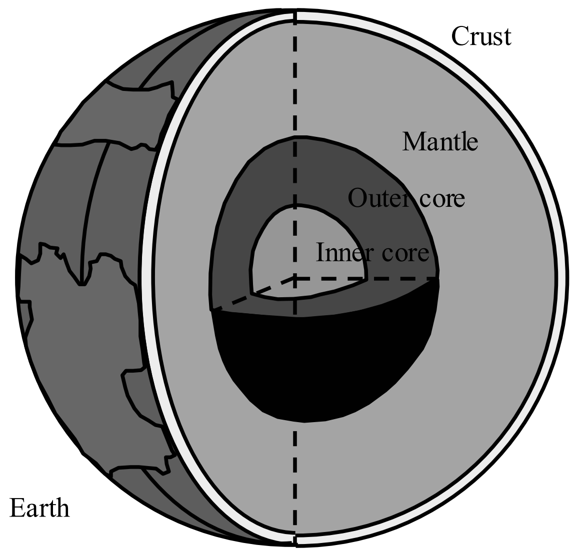


Figure 0.1 The earth and its interior (Ref. 0.16)

The differences in temperature and pressure across the interior of the earth results in continuous dynamic activity, although this cannot be easily sensed from the surface where we live. The activity makes its presence felt periodically at different locations on the surface crust through various types of disruptions (volcanoes, earthquakes, tsunamis).

The crust is fractured into many pieces, called tectonic plates, which tend to move continually on the mantle (which is a hot fluid that sets up 'convection' currents). The various plates move in different directions and at different speeds. When two adjoining plates collide against each other, they often end up forming mountains on the earth's surface over a period. Continued movement will cause these mountains to increase in height, as in the case of the Himalayas. Other types of relative motions at the plate boundaries are also possible, causing earthquakes.

Owing to the frictions generated at the plate boundaries, the free relative movement between plates is prevented and a large amount of energy (called strain energy) gets built up in local regions of the earth's rocky crust. The resulting high stress eventually exceeds the strength of the rock along weak regions (called faults). Such local failures result in a sudden release of a tremendous amount of the stored energy, and this manifests in the form of a series of sudden and violent movements in a very short time (often less than a minute). The earth quakes!

Quaking implies vibrations (to-and-fro movements). This happens because the release of the large strain energy generates seismic waves (such as 'body waves' and 'surface waves') in all directions, reflecting and refracting, as they travel across various strata (Figure 0.2). The shaking (or quaking) is usually most severe at the surface of the earth near the epicentre, where we and our buildings are located.

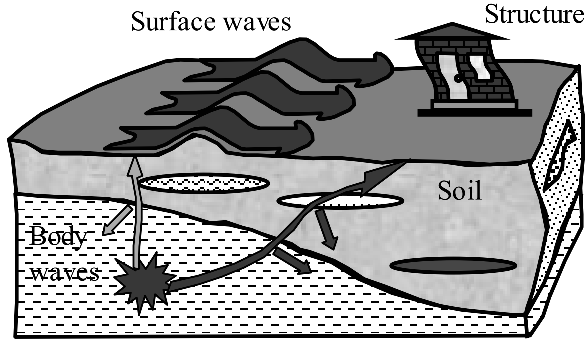


Figure 0.2 Movement of waves during an earthquake (Ref. 0.16)

0.2.2 When and Where Does the Earth Quake?

When and where exactly does the release of the built-up energy take place? If only we had some fore-warning about this, we could do much to save lives, if not buildings. Unfortunately, this cannot be predicted scientifically, as of now.

The exact timing of this natural phenomenon is something that eludes the scientist, unlike other natural events like eclipses, whose occurrence can be predicted with pin-point accuracy. In the case of the earthquake, we have no idea when and where the next one will occur. When, for example, will the next major earthquake hit an urban centre like Delhi (or Mumbai)? It could happen today, or after a month, or a year, or several decades later. We must be prepared for the worst, especially in view of the fact that the last big quake to hit Delhi happened long, long ago.

Historical records reveal the tendency of earthquakes to revisit regions after an interval of time (called *return period*), which is variable (random). That is why there is concern when several decades (or, sometimes, a century or more) have elapsed since the last big quake has hit a particular region. Conversely, once a major earthquake has occurred at a particular place (sometimes followed immediately by a series of low-magnitude tremors), there is less danger of another major earthquake (of similar intensity) occurring at the same location in the immediate future. For this to occur, the strain energy has to build up again over many more years.

We know, from historical records that some regions of the earth are more vulnerable to earthquakes than others. This is the basis of *seismic zonation*. However, with the passage of time, the earth can throw up new surprises (such as dormant faults becoming active). For example, the Deccan plateau of the Indian sub-continent was considered to be relatively earthquake-free, until the occurrence of the Latur earthquake in 1994. We need to keep monitoring and updating the various zones and their relative seismic intensities.

The latest seismic zones in India and codified in the Indian Standard (IS:1893 - 2002, published by the Bureau of Indian Standards) are as given in the map in Figure 0.3. The higher the zone, the more vulnerable is that region to a severe earthquake.

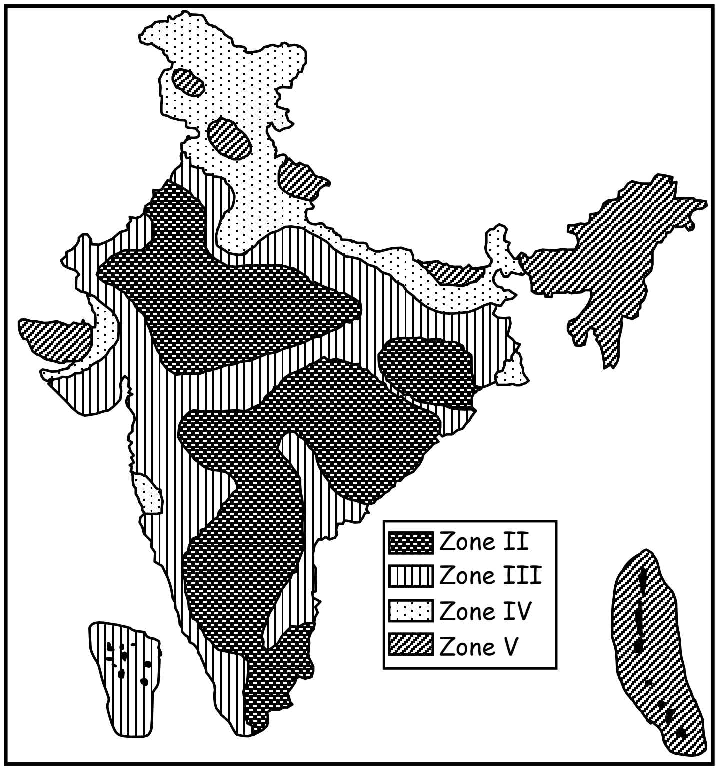


Figure 0.3 Seismic zones in India (IS:1893 - 2002)

0.2.3 Size of the Earthquake

A typical earthquake lasts for less than a minute, but can destroy a city built over a period of centuries. The time is too short for people to escape from most buildings. Therefore, the only way to ensure survival is by ensuring that buildings do not collapse during earthquakes, even though some damage may be unavoidable. The extent of damage depends not only on the structure but also on the size of the earthquake.

The size of the earthquake is best described by its *magnitude*, as measured at the epicentre of the quake. As illustrated in Figure 0.4, the epicentre is the location on the surface of the earth, exactly above a point called focus (at a focal depth, which could extend up to about 70 km), from which the seismic waves are generated.

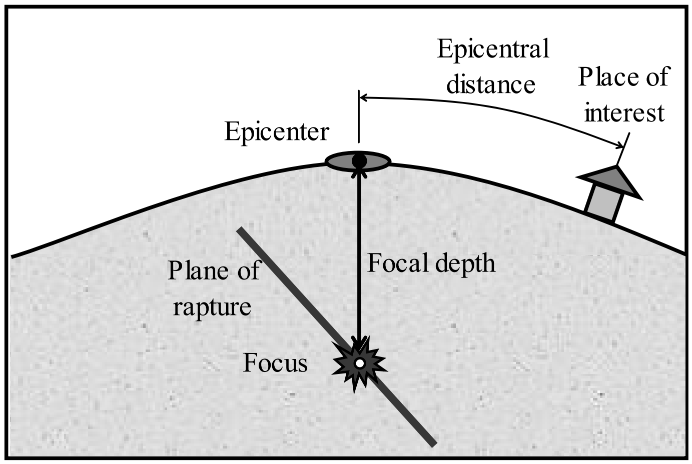


Figure 0.4 Basic terminology of earthquake (Ref. 0.16)

With the help of seismograms, it is possible to locate the epicentre and arrive at a numerical measure of the strain energy released. It is convenient to express this energy released on a relative scale. The commonly used scale for this purpose is the Richter scale. For example, an earthquake with a magnitude of 8.3 (close to the 2001 Bhuj earthquake), is expressed as M8.3. Every unit increase in magnitude implies an increase of about 31 times higher the energy. Thus, the energy released during the M8.3 earthquake is 31 times that of an M7.3 earthquake, and $(31)^2$ times (close to 1000 times) the energy of an M6.3 earthquake. The energy release of such an earthquake works out to roughly 1000 times the energy released by the atom bomb dropped at Hiroshima in 1945!

Buildings located on the surface of the earth get affected by the earthquake, depending on various factors, including how close they are with respect to the epicentre (epicentral distance is shown in Figure 0.4). The magnitude of an earthquake does not give an indication of the size of the earthquake as experienced at different locations in the neighbourhood, having different epicentral distances. For this, the appropriate measure is *intensity*, which is a qualitative description of the effects of the earthquake at a particular location, as evidenced by observed damage and human reactions at that location. The Modified Mercalli Intensity Scale, ranging from I to XII, is commonly used to measure intensity. The Indian standard (IS: 1893) refers to the “MSK 64” intensity scale with 12 categories; 1) Not noticeable, 2) Scarcely noticeable, 3) Weak, partially observed, 4) Largely observed, 5) Awakening, 6) Frightening, 7) Damage of building, 8) Destruction of building, 9) General damage, 10) General destruction, 11) Destruction, 12) Landscape changing.

0.2.4 Damage Caused by an Earthquake

The extent of damage depends not only on the size of an earthquake (either in terms of magnitude or intensity), but also on the type of construction practice followed in a particular region or country. For example, an earthquake in Tokyo or Los Angeles may result in damage of only a few buildings because strict construction regulations are adopted, but the same earthquake may be catastrophic in Mumbai or Delhi, in terms of buildings damaged and lives lost, because the building design and construction practice are not adequately regulated.

Let us not forget: earthquakes do not kill; unsafe buildings do.

Our efforts should be directed to mitigating this potential disaster, by ensuring that buildings have the desired seismic resistance, to the extent possible. Seismic retrofit of existing buildings is probably the surest way to mitigate the disaster.

0.2.5 Do's and Dont's During an Earthquake

The earthquake, when it occurs, catches human beings by surprise. Sometimes, the earthquake announces its imminent arrival with a rumbling noise. It makes its presence felt by shaking, which may be mild or violent, but not lasting for more than a minute. In those precious moments, we must try to remain calm and take appropriate action. What action to take depends on whether we are located indoors or outdoors. The following are some simple and practical suggestions to follow.

If indoors, and if we suspect that the building is structurally weak, our safety lies in making an exit from the building by the shortest and fastest route. Do not use the lift. On the other hand, if the building is structurally safe, remaining indoor is the best option. Do not try to exit the building during the shaking. Avoid doorways, as doors may slam and cause injuries. For safety, drop to the floor, get underneath a sturdy cot or table and hold on to its base (Figure 0.5). Keep away from possible falling objects. If the tremor is violent, then after the shaking is over, it is wise to exit the building, in view of possible after-shocks and damage to the structure.

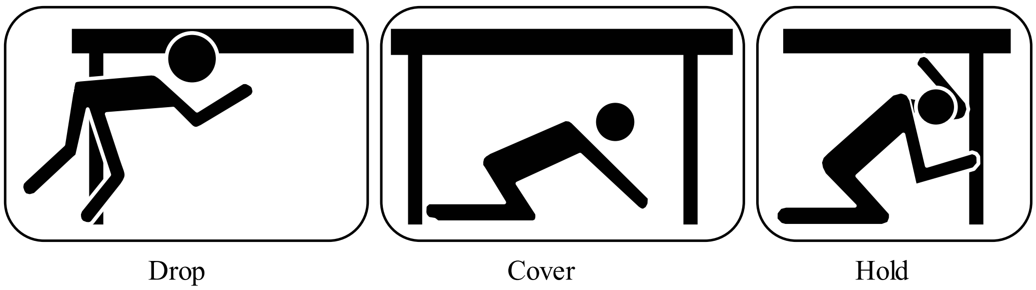


Figure 0.5 Safety measures while indoors

If outdoors, move to an open area, away from buildings and objects that could possibly fall (such as hanging sign boards and overhead power lines). If driving, pull over to the side of the road and park at a safe place, not blocking the road.

0.3 HOW BUILDINGS RESPOND TO EARTHQUAKES

0.3.1 Effect of Mass and Height

The earthquake as well as wind load acting on the buildings are termed as 'lateral loads' since their effect is felt mainly in the horizontal direction. This is in contrast to the weights of the building (and occupants), which act vertically down due to gravity. Forces due to earthquake, called *seismic forces*, are induced in a building because of the heavy masses present at various floor levels. Such forces are called inertial forces, is calculated by the products of the masses and their respective accelerations. If there is no mass, there is no inertial force. Accelerations generated by the seismic waves in the ground get transmitted through the vibrating structure to the masses at various levels, thereby generating the so-called horizontal seismic forces. The building behaves like a vertical cantilever, and swings horizontally almost like an inverted pendulum, with masses at higher levels swinging more (Figure 0.6). Hence, the generated seismic forces are higher at the higher floor levels. Because of the cantilever action of the building (fixed to the ground and free at the top), the forces accumulate from top to bottom. The total horizontal force acting on the ground storey columns is a sum of the forces (seismic loads) acting at all the levels above. This is termed as the *base shear* and it leads to highest stresses in the lowermost columns.

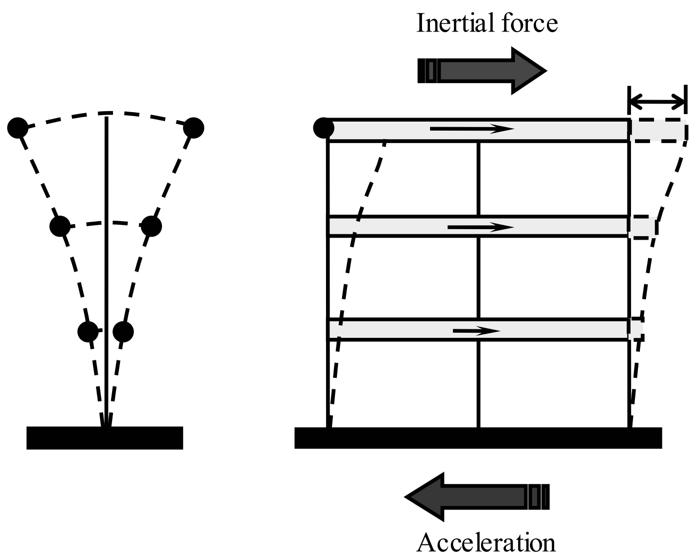


Figure 0.6 Seismic forces generated by masses vibrating

Heavier buildings attract larger seismic forces. On the other hand, lighter buildings are affected less. This was an important lesson learned from the great Assam earthquake of 1897 (magnitude M8.1), which destroyed almost all buildings (up to 3 storeys) built in British India. Following this disaster, constructions were limited to single and double storeyed "Assam type" dwellings with light roofing (Figure 0.7) as ideal earthquake-resistant construction in North-East India, which falls under the highest seismic zone (zone V) in the country.

Figure 0.7 “Assam type” low-rise and light building

Over the years, these basic lessons have been forgotten, and numerous high-rise buildings have mushroomed, especially in recent times in the urban centres of the country. Many of these buildings are seismically deficient. Figure 0.8 shows an old building in Guwahati, originally 4-storeyed, to which three additional storeys were added recently — an example of a potential man-made disaster, waiting to happen, in a highly congested area. To avert such disasters, local building authorities must strictly ensure that all new constructions should comply with design standards. Existing buildings that are highly unsafe must be declared unfit for occupation (and, if located in congested areas, must be demolished), unless they are retrofitted appropriately.

Figure 0.8 A 4-storeyed old building (in zone V) extended to 7-storey!

0.3.2 Effect of Stiffness

Buildings are expected to behave elastically under service loads. Elasticity is that property by which a body or a structure, displaced by a load, regains its original shape upon unloading. It is by virtue of this property, that buildings that are pushed horizontally by wind or mild earthquake loads, return to the original vertical configuration after the wind or the tremor has passed (Figure 0.9).

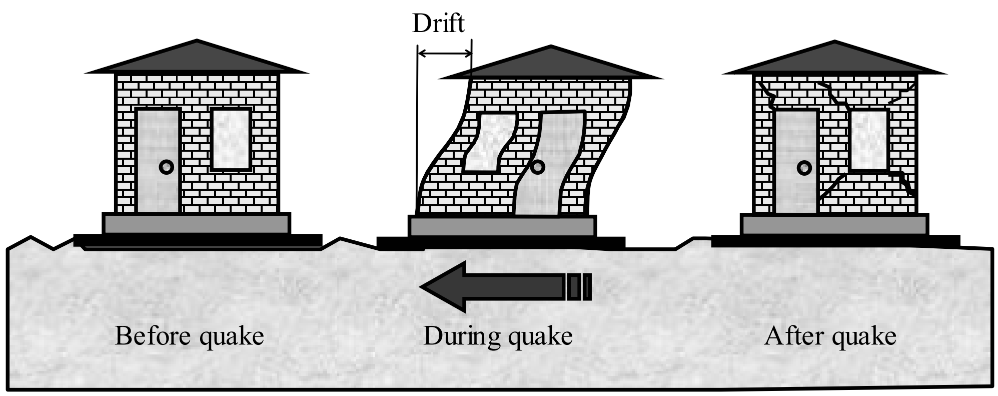


Figure 0.9 Elastic behaviour of a building (Ref. 0.16)

How much the building deflects under a given load is measured by a property called stiffness, which may be defined as the force required to cause unit deflection. The stiffness required to resist lateral forces is termed as lateral stiffness. The stiffer the building, the less it will deflect ('drift', as indicated in Figure 0.6 and 0.9)

Should our buildings be relatively stiff or flexible?

Certainly, it is desirable for the building to behave elastically under lateral loads, including forces under low earthquake levels that are likely to occur occasionally during the life of the building. But, it would be highly uneconomical to design ordinary buildings to behave elastically under high level earthquakes, which are rarely expected to occur during the design lifetime of the building. Because of the limitation of resources, the design standards allow us to take some risk of damage in the event of a rare severe earthquake. What we need to ensure is that the building, although likely to be severely damaged in the event of the rare earthquake, does not collapse, so that lives are not lost. How do structural engineers achieve this? They allow the building to behave inelastically (that is, the building does not regain its original shape after the earthquake) at such high load levels and thus dissipate energy. The building's original stiffness gets degraded, and it becomes flexible.

Of course, there must be adequate stiffness in buildings. This can be achieved by providing adequate lateral load resisting systems (such as masonry walls with bands in small buildings, frames, braces or shear walls in large buildings). Otherwise, they will get severely damaged and may even collapse under low level earthquakes. In the case of exceptionally important buildings (such as nuclear reactors), it is even desirable to have sufficient stiffness to ensure elastic behaviour even under rare earthquakes. They must survive at all cost, and that too without damage.

The mass and lateral stiffness of the building contribute to another important structural property, called the natural period of vibration. It is the time taken by the building to undergo a cycle of to-and-fro movement (like a pendulum). Buildings with high stiffness and low mass have low time period, whereas buildings with low stiffness and high mass have high natural period. The value of this natural period also governs the magnitude of seismic force that the building will attract. This is similar to the effect of resonance in a vibrating system. The conventional buildings of a few storeys, common in urban areas, have low natural period and hence attract higher seismic forces as a fraction of their weight.

0.3.3 Effect of Ductility

The ability of a structure to deform with damage, without breaking suddenly (without warning), is termed as *ductility*. With ductility, a building can continue to resist seismic forces without collapsing. There is a story of a proud tree teasing a blade of grass for not being able to stand erect in the wind. But, when a very severe storm came, it was the tree which fell down and the blade of grass survived (Figure 0.10).

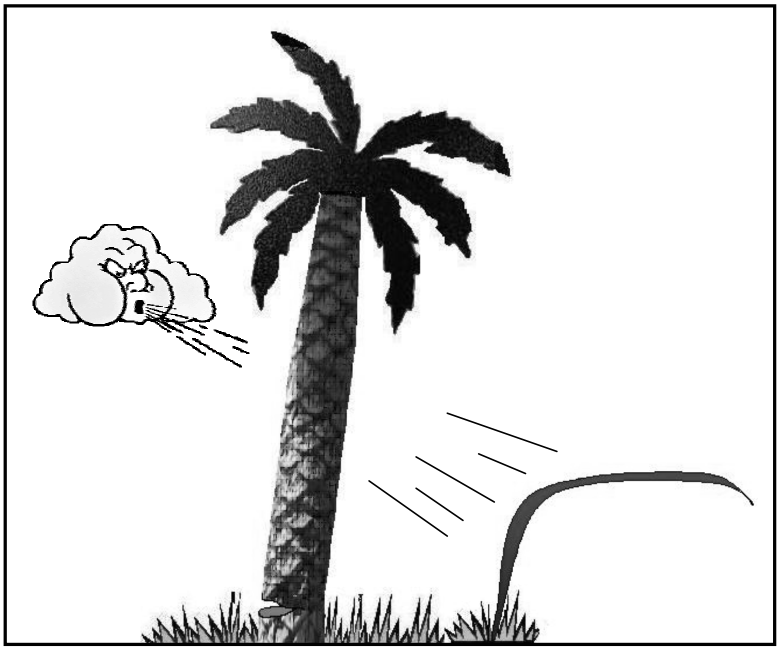


Figure 0.10 Blade of grass survives, but the tree does not!

This is because the blade of grass was able to undergo very large deformation without breaking down, unlike the big tree which snapped suddenly at its base, when its strength was exceeded[†].

In a similar way, buildings too can exhibit either ductile or brittle (non-ductile) behaviour, depending on the structural material, design and detailing. Generally, conventional masonry buildings exhibit brittle behaviour (Figure 0.11a), when provided with earthquake resistant features. On the other hand, well-designed buildings made with reinforced concrete or structural steel can exhibit ductile behaviour (Figure 0.11b). Concrete is made of stone chips, sand, cement and water. When steel bars are placed in the concrete to strengthen it for tension, it is termed as reinforced concrete. Otherwise the concrete is termed as plain concrete.

[†] Masonry buildings, although brittle, can be designed and detailed to have earthquake resistant features, as described in the Section 0.4.

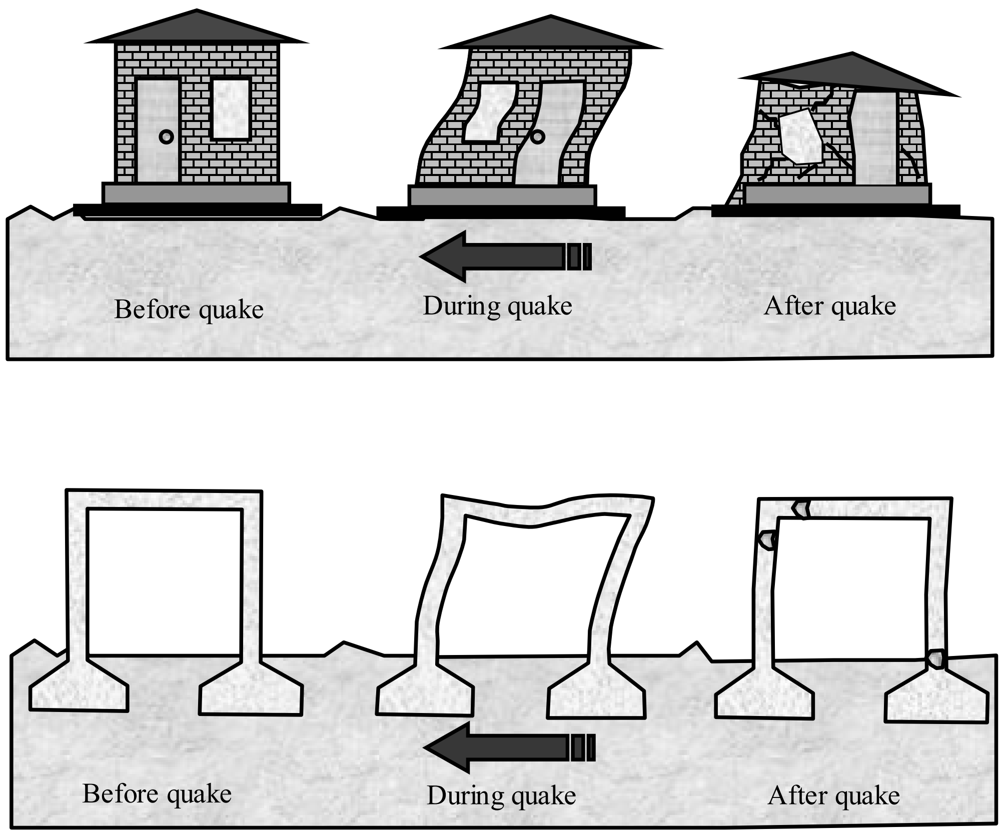


Figure 0.11 Behaviour of brittle and ductile buildings under a major earthquake (Ref. 0.16)

Materials like brick, stone and plain concrete are relatively brittle. When bricks or stones are used in masonry wall construction without adequate bond, they can fall apart suddenly, even if the walls are relatively thick. This indeed is how the failure of many buildings occurred during the Latur earthquake (Figure 0.12). Many such buildings have weak mud mortar and absence of bond stones.

Figure 0.12 Collapse of brittle and poorly bonded stone masonry walls (Latur earthquake, 1993) (Ref: www.nicee.org)

Ordinary buildings are commonly classified as either *load bearing* or *framed* structures. Most low-rise buildings are load bearing buildings made of masonry walls, which resist both gravity (vertical) loads and lateral loads due to wind and earthquake. As the building height increases, and in high seismic zones, lateral loads tend to govern the design. In such cases, it is structurally efficient and economical to adopt framed buildings. In such buildings, it is the framework (skeleton) of the building, comprising beams, columns and footings, made usually of reinforced concrete, which mainly resist both vertical and lateral loads. The walls in the framed buildings are treated as non-structural elements (cladding and partitions), whose locations may be shifted without affecting the structural stability. In high-rise buildings, reinforced concrete shear walls are often introduced to enhance the lateral load resisting capacity.

Unlike brick and stone, materials like steel and reinforced concrete (if properly designed and detailed) possess considerable ductility. Ductility is required at locations of very high stress, such as the beam-column joint (Figure 0.13). The horizontal bars in the beam should be anchored well in the joint and the vertical ties spaced closely near the joint. The vertical bars and the closely spaced horizontal ties in the column should be continuous throughout the joint. These important details are often overlooked in reinforced concrete construction, whereby the desired ductility is not achieved.

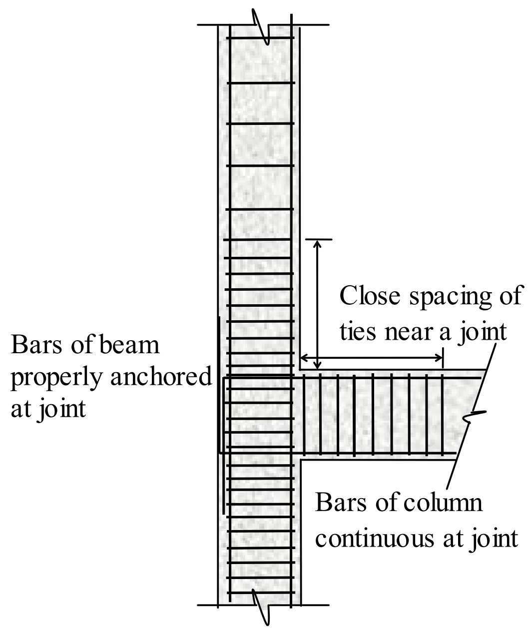


Figure 0.13 Ductile detailing of reinforcing bars at a beam-column joint

If the structural components (walls, beams, columns etc.) in a building can "hang on" through ductile behaviour, without breaking down during the brief period of the major earthquake, the building will not collapse, even though it may get damaged. Such ductile buildings attract lesser load with increasing deformation (since stiffness gets reduced) than the buildings which remain stiff (like the tree in our story). Also, a significant amount of the input energy due to the earthquake in the building gets dissipated through the yielding (deformation without increase in stress) of the ductile materials (Figure 0.14a). Otherwise, the entire input energy needs to be stored as elastic strain energy (Figure 0.14b). The strain energy and the dissipated energy are shown by the shaded areas in the base shear versus drift graphs. However, as the drift is generally much less in the latter type of buildings, the total seismic force to be resisted by such a building is very high to handle the same input energy. This is not practical for ordinary buildings, because it calls for very large member sizes and will be prohibitively expensive. If the desired ductility can be provided in the building, the design seismic force ($V_{inelastic}$ in Figure 0.14a) can be much lower (up to 20%) than the corresponding force in an elastic building ($V_{elastic}$ in Figure 0.14b).

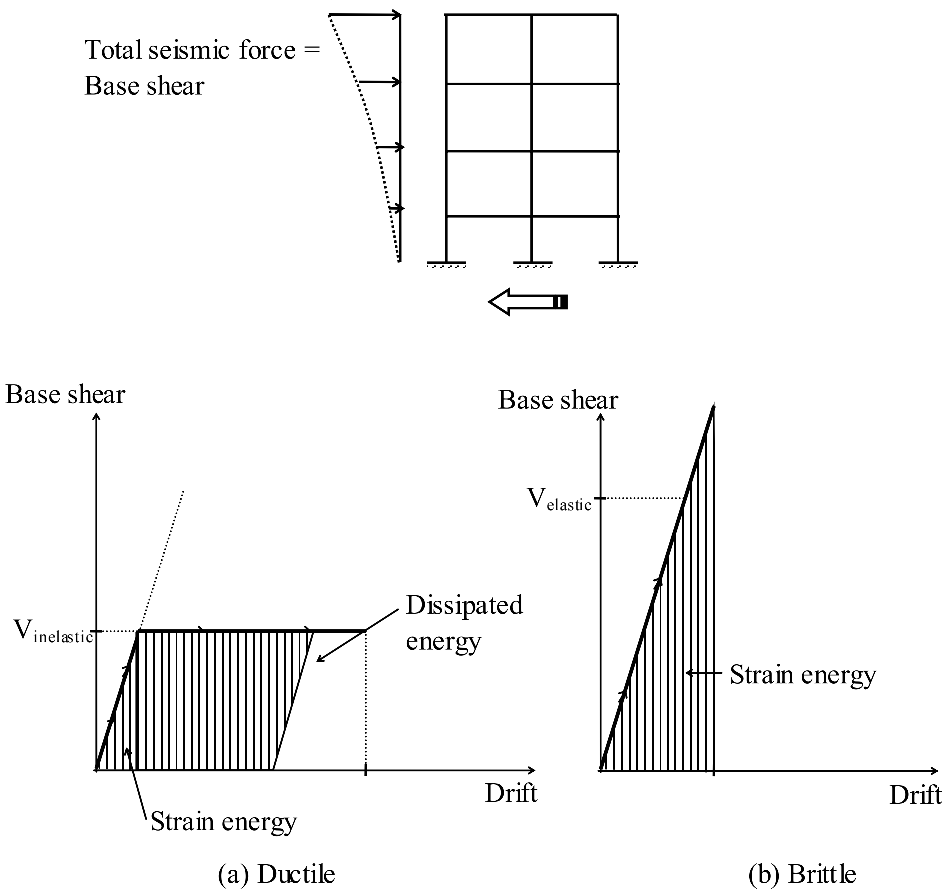


Figure 0.14 Reduction in design seismic force (base shear) in ductile buildings

Sadly, most of the existing buildings in our country seem to lack adequate ductility – even the so called "engineered" buildings, primarily due to lack of attention paid to seismic design and detailing.

0.3.4 Effect of Strength and Integrity

Every structural component has strength, which is the magnitude of the maximum internal force (such as axial force or bending moment) it can resist under a certain type of loading. When this strength is exceeded by the applied load, the material fails (or collapses). The strength depends not only on the type of material (for example, steel is stronger than brick), but also on other factors, such as the size of the cross-section (for example, a 230 mm thick brick wall is twice as strong as a 115 mm thick wall). However, the thicker wall will attract higher earthquake force.

As mentioned earlier, the load attracted by a structural component during an earthquake depends on the mass and lateral stiffness of the building. If the building is not designed to "yield", and behave in a ductile manner, it will be required to resist higher load during an earthquake to prevent a sudden failure (refer Figure 0.14). A structural component should be designed to have a strength that is not less than the maximum internal force, associated with the overall seismic load on the building, and the associated ductility.

If the strength is not adequate, the building component will fail. The failure of the vertical components in a building (such as load bearing walls, columns or footings) is more critical than that of the horizontal components (such as beams and slabs), because the former type of failure is likely to trigger a possible collapse of the entire building. The failure of a beam may cause only a local distress; but the failure of a ground storey column or a footing can trigger overall building (global) collapse. In the recent Gujarat earthquake, many multi-storeyed buildings collapsed because of the failure of the columns in the ground storey (Figure 0.15).

Figure 0.15 Collapse of the ground storey of a building
(Bhuj earthquake, 2001) (Ref: www.nicee.org)

Thus, an important principle adopted in the seismic resistant design of framed buildings is this:

Soil must be stronger than foundations;
foundations must be stronger than columns;
columns must be stronger than beams.

To ensure that forces are safely transmitted from beams to columns, from columns to foundations, and from foundation to soil, the connections at the beam-column joints, column-foundation joints and foundation-soil interface should have the required strength (and ductility). This requirement is part of ensuring the *integrity* of the building. Retaining the stiffness in the building is the other aspect of integrity. Consider, for example, a single room, bounded by four masonry walls and covered by a roof slab. If the connections between the walls and between the slab and the walls are not effective, the building has limited strength and stiffness to resist seismic forces, and can collapse even under a minor earthquake shaking. Indeed, this is precisely what happened to a large number of dwelling units during the Latur earthquake (Figure 0.16).

Figure 0.16 Poor integrity in a masonry building
(Latur earthquake, 1993) (Ref: www.nicee.org)

In a masonry building, if reinforced concrete bands are provided in walls at plinth, lintel and roof (for pitched roofs) levels and the wall-to-wall connections provide good bond (that is, do not "open out"), the walls will act together like a box. This will enable the building to resist even a major earthquake effectively (Figure 0.17).

Figure 0.17 A masonry building with lintel band after 1993 Killari Earthquake: good integral action (Ref: www.nicee.org)

The more the number of lateral load-resisting systems in a building, the less is the potential for global collapse. This is because when one of them fails, the loads get redistributed, and other systems take an increasing share of the load. This potential for having multiple load paths in a building is called redundancy and it makes the building as a whole stronger.

0.3.5 Effect of Layout and Configuration

Providing earthquake resistance to buildings is primarily the responsibility of civil engineers. But architects also have a major role to play. Some architectural features, relating to overall size and shape, are unfavourable and invite potential seismic disaster. It is desirable that the client also knows something of these features, if the building is located in a high seismic zone. Prevention is always better than cure. In such situations, safety is preferable to fancy looks. Otherwise, structural design has to be done carefully and competently. In general, if the basic architectural features favouring good seismic resistance are adopted, the cost of making the building earthquake proof is less and even an incompetent engineer cannot do much damage.

The, building should have a simple geometrical shape in plan, such as rectangular or circular. All rectangular shapes are not uniformly good. If the building is too long (in one direction) or too large in plan, it is likely to be damaged during earthquakes. Buildings with large cut-outs in the walls, floors and roofs are undesirable, as this affects their integrity. Similarly, buildings which have 'L', 'U', 'V', 'Y' or 'H' shapes in plan are also undesirable, inviting severe stresses at the interior corners called re-entrant corners (Figure 0.18). Each wing of the building tends to vibrate separately in the event of an earthquake, causing serious problems at the common core region, leading to potential collapse (Figure 0.19).

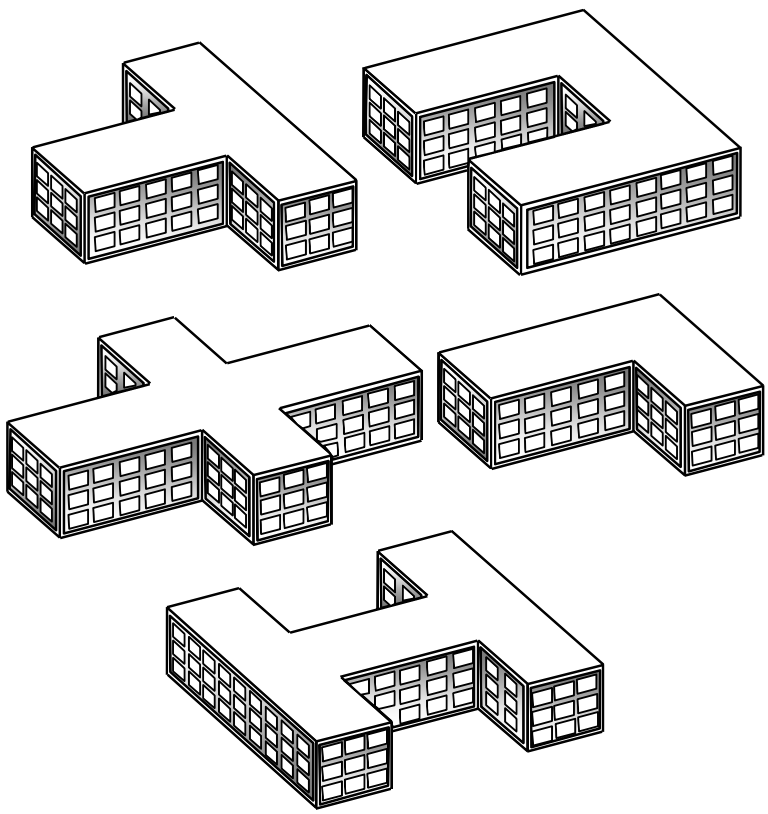


Figure 0.18 Buildings with plans that lead to large earthquake damage

Figure 0.19 Collapse of an “L” shaped building (Macedonia earthquake, 1963)

Buildings with asymmetry in plan are bound to twist under an earthquake, inviting further damaging effects. A general rule of thumb is to make the building as simple, solid and symmetric in plan as possible. If complex geometries are absolutely required, then it is desirable to break up the building plan into separate simple rectangular segments with proper separation joints, so that they behave as individual units under an earthquake (Figure 0.20).

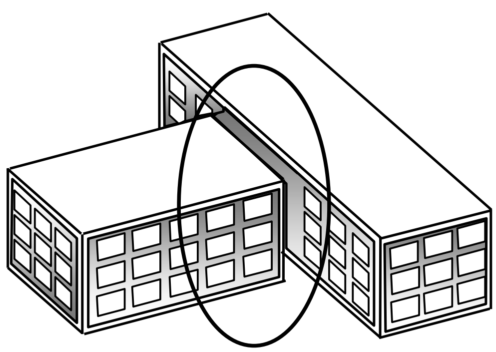


Figure 0.20 Building with separation joint between its two distinct “wings”

Buildings should not only be simple in plan but also in elevation. The walls and columns should continue uninterrupted from top to bottom, to ensure transmission of forces to the supporting ground through the shortest and simplest path. If there is any discontinuity in this path of load transmission, there is a danger of potential damage to the building in the event of an earthquake. Hence, hanging or floating columns (columns which begin in an upper storey from a beam) and discontinuity of walls in the ground or other storey (open ground storey) should be avoided (Figure 0.21). Figure 0.22 shows a residential multi-storeyed building with open ground storey for car parking.

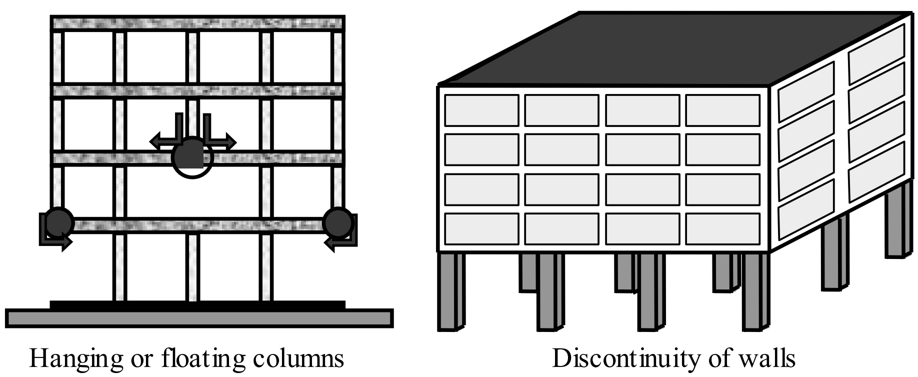


Figure 0.21 Buildings with discontinuities in columns or walls (Ref. 0.16)

Figure 0.22 Building with open ground storey (Ref: www.nicee.org)

In the 2001 Gujarat earthquake, many buildings with open ground storey collapsed and the parked cars were flattened (Figure 0.15). This is because the sudden change in stiffness at the ground storey induces extreme high stresses in the ground storey columns, which were not accounted for in the design.

Buildings with vertical setbacks (either plaza type buildings, as shown in Figure 0.23 or with cantilever projection at the top) or many overhanging projections (Figure 0.24) perform poorly in terms of seismic resistance. In fact, all overhanging projections including balconies should be avoided, as these are the first elements to collapse in the event of an earthquake (Figure 0.25).

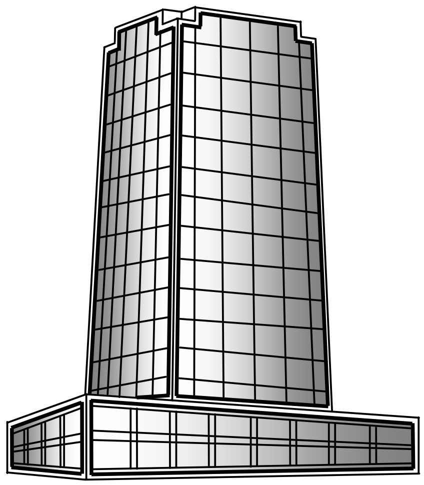


Figure 0.23 A building with vertical setbacks (plaza type building)

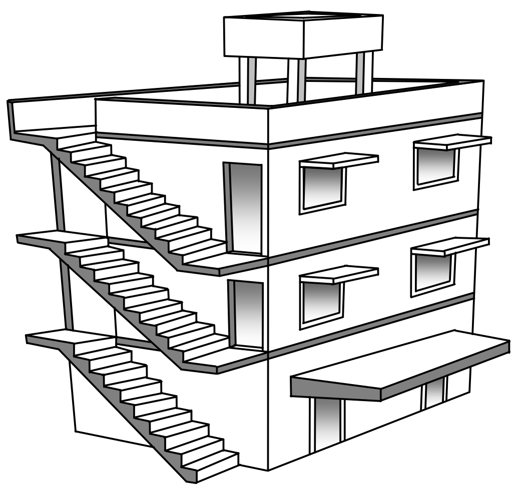


Figure 0.24 A building with overhanging projections

Figure 0.25 Collapse of overhanging projections (Bhuj earthquake, 2001)
(Ref: www.nicee.org)

Special attention should be given to the planning of the staircases, as these vital escape routes are at times the first ones to attract damage in a major earthquake. To ensure their survival, the slabs should ideally be supported on inclined beams (stringer beams) which connect integrally with the main frame in the building (Figure 0.26). This provision is usually neglected by architects. Since structural safety is paramount, the clients should insist on this provision.

Figure 0.26 Stairs with stringer beams connected to the building frame

Two buildings or two structurally isolated sections of a single building should not be too close to each other, as there is a danger of possible collision against each other (Figure 0.27). This effect is called pounding and is usually not considered in design. The effect is more severe for tall buildings. It is recommended that a calculated gap subject to a minimum value be maintained between the buildings to avoid pounding. Otherwise, there can be serious damage to both the buildings or building sections, even if they are designed adequately individually.

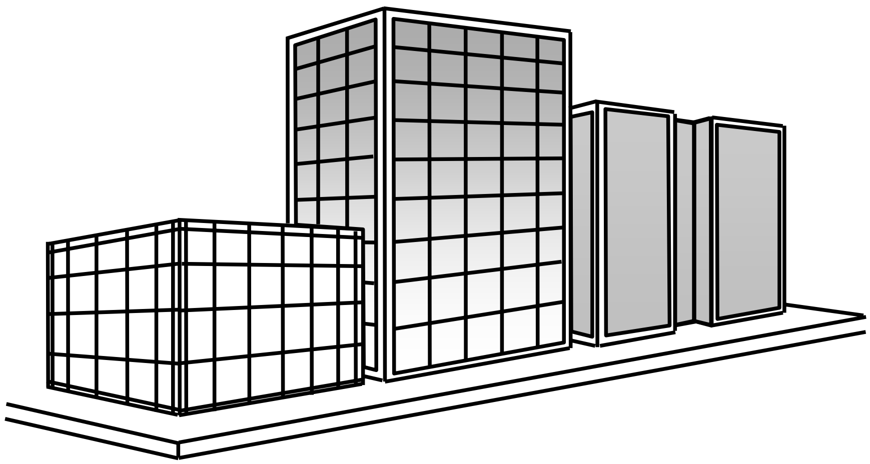


Figure 0.27 Closely spaced buildings can be subjected to pounding

0.3.6 Effect of Soil

The accelerations that occur in the rock layer of the crust during an earthquake get transmitted to the building through the soil over the rock layer. When the soil is relatively soft, the accelerations tend to get magnified, resulting in the structure attracting higher seismic loads. This must be taken into account during structural design. Knowledge of the soil strata is also essential for designing the foundations of the building.

Buildings located in loose granular soils (sands), in the presence of subsoil water, have another serious and potential danger that can occur during an earthquake. The soil can behave like quicksand through a phenomenon called liquefaction. This happens because of a sudden increase in pore water pressure on account of seismic shear waves, causing the water-sand mixture to flow upwards and practically convert the soil behaviour to that of a liquid. Buildings located in such soils may sink or go afloat, and tilt significantly and collapse (Figure 0.28).

Figure 0.28 Buildings tilted due to liquefaction of soil (Nigata earthquake, Japan, 1964)

0.4 SEISMIC BEHAVIOUR OF MASONRY BUILDINGS

0.4.1 Basic Components

The basic components of masonry buildings are foundations (spread wall footings), walls, floor slabs and roofs.

The walls and footings are mainly made of bricks or stones, laid in horizontal courses, with mortar filling up the gaps and providing the required bond between the units. The mortar is usually made with sand, mixed with cement, lime and/or mud, and water. Modern buildings are usually built using cement mortar, while traditional building have been built using lime mortar or mud mortar. Mud mortar, which continues to be used in low-cost construction, is relatively weak and unsafe.

The floor and roof slabs in modern buildings are made in reinforced concrete. In traditional buildings, these have been made using other materials such as timber, Madras terrace, jack arch, tin sheets, thatch and tiles.

0.4.2 Transmission of Gravity Loads

The main load on such structures is the self-weight ("dead load") due to gravity and the weight due to human occupancy as well as furniture and storage ("live load"). Under these gravity loads which act vertically downwards, the slabs undergo bending and transmit the forces through the walls to the foundation below. This structural action induces compression in bricks and stones. The forces are eventually transmitted to the soil below through bearing. As the soil bearing capacity is low compared to the compressive strength of masonry, the width of the wall footing is increased (in steps), to provide for larger bearing width, and hence lesser bearing stresses, at the level of contact with soil (Figure 0.29).

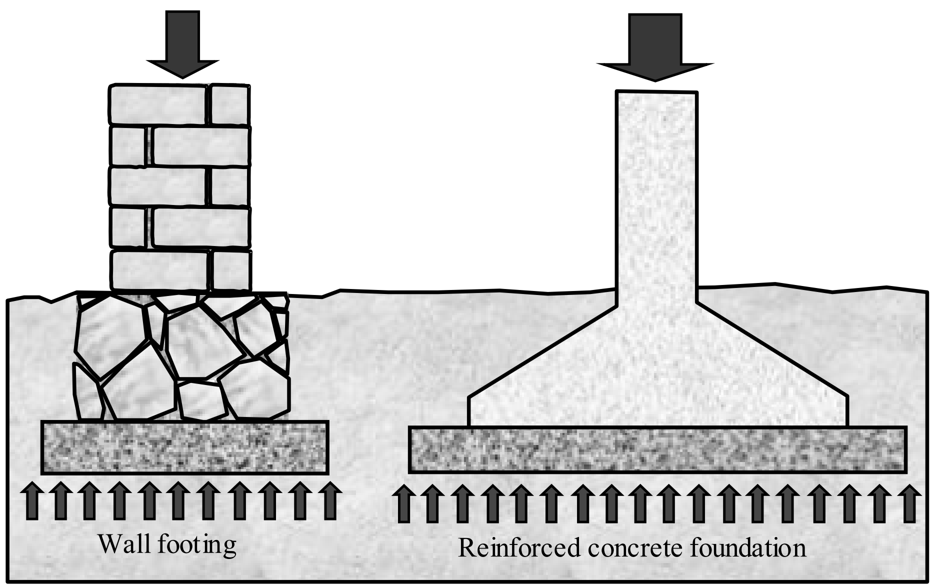


Figure 0.29 Increased width of footing under walls and columns

0.4.3 Transmission of Seismic Loads

The buildings also have to resist lateral loads induced by wind and earthquake. As discussed earlier, seismic loads are generated due to the various masses in the building undergoing accelerations. Eventually, these seismic loads are transmitted to the soil below (Figure 0.30), as in the case of gravity loads, but with a major difference.

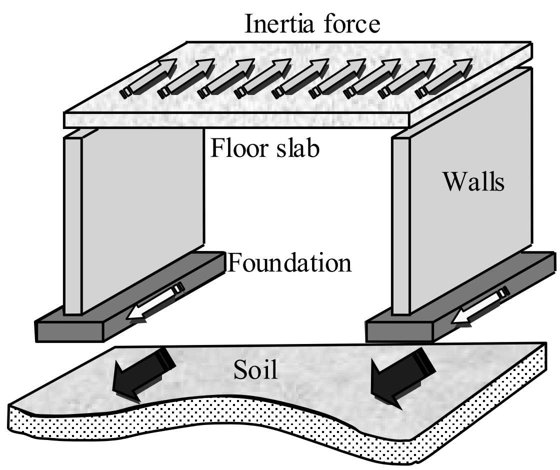


Figure 0.30 Seismic forces transmitted to the soil through walls and footings (Ref. 0.16)

The floor slabs tend to move rigidly in the horizontal plane, transmitting their inertial forces to the vertical masonry walls, inducing overturning moment and shear forces in these walls. The overturning moment causes one part of the wall (roughly one-half) to undergo tension and the other part to undergo compression. This tension can exceed the compression induced due to the dead loads. As masonry is weak in tension, this can cause cracking (Figure 0.31).

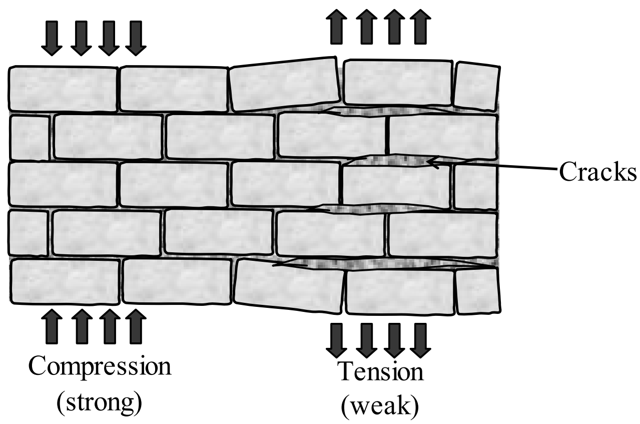


Figure 0.31 A masonry wall under compression and tension

The same wall, which is subject to bending in its own plane, is simultaneously subject to shear. The combined effect of this is to cause the rectangular wall to take the shape of a parallelogram, in which one diagonal gets contracted (compressed) and the other gets extended (stretched). Cracking is likely to occur in a direction perpendicular to the diagonal tension (Figure 0.32). As the seismic loads are reversible, this cracking occurs along both diagonals. Such diagonal ‘X-cracks’ are characteristic tell-tale signs left behind by an earthquake (Figure 0.33).

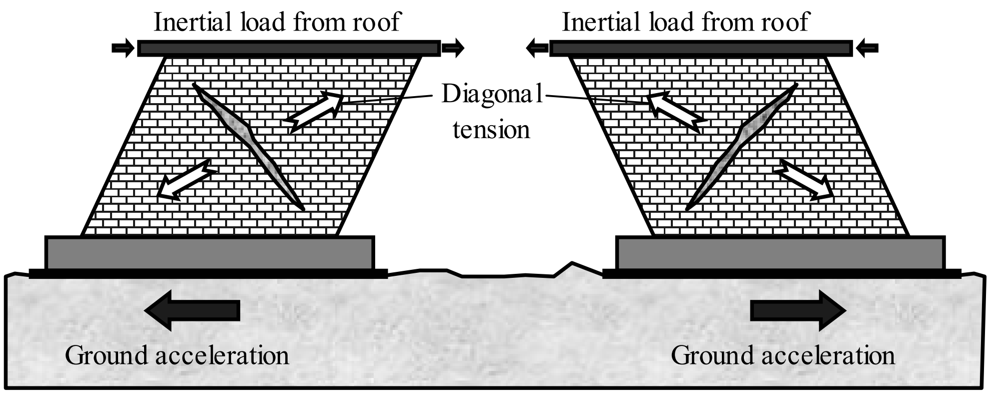


Figure 0.32 Formation of diagonal cracks in a masonry wall due to earthquake

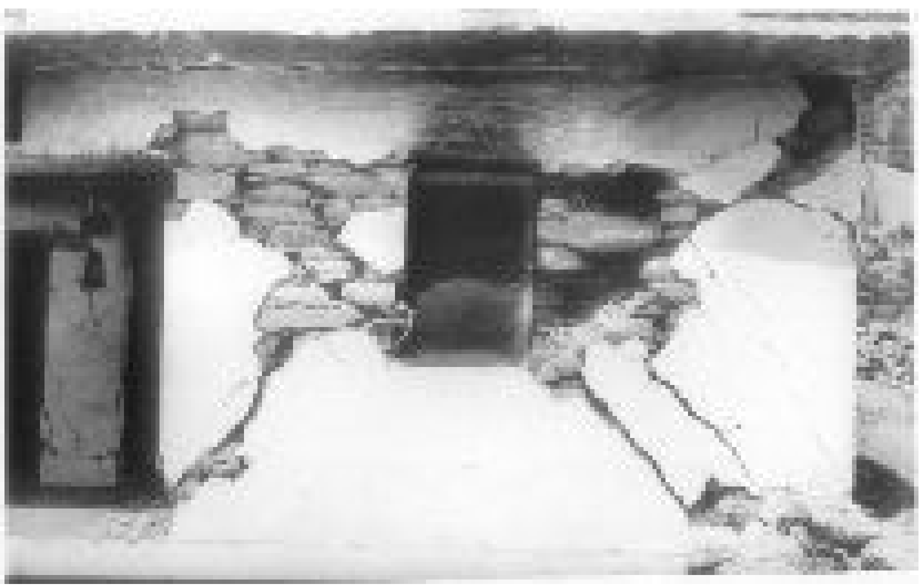


Figure 0.33 Extensive diagonal cracks in masonry walls (Ref: www.nicee.org)

0.4.4 Need for Cross Walls

A wall is subjected to seismic forces not only in its own plane but also perpendicular to the surface of the wall (out of plane). The bending and shear resistances of a wall increase with increase in the wall thickness and with increase in compression due to gravity load. However, unless the wall is extremely thick and well bonded, it will lack the stiffness and the strength to resist the forces acting perpendicular to it (Figure 0.34). Such thin long walls will easily collapse under an earthquake, unless they are braced by cross walls. A “cross wall” resists the horizontal forces acting in its own plane. The longer the cross wall, the stronger and stiffer it is to resist such forces. In summary, a masonry wall has good resistance to seismic forces acting in the direction of the wall, but it is not effective in resisting loads acting perpendicular to its plane and requires the assistance of cross walls for this purpose.

Fortunately, the total seismic force acting on the building is distributed to the various wall components in proportion to their lateral stiffness along any given direction. Thus, it is the cross walls that bear the brunt of the earthquake loading in any direction. However, this is possible only if they are adequate in number and well bonded to the perpendicular walls and the floor/roof slabs. Therefore, there must be enough number of walls, which should neither be too thin nor with too many openings, and reasonably closely spaced, in two perpendicular directions, and tied together properly, for a masonry building to have the desired seismic resistance.

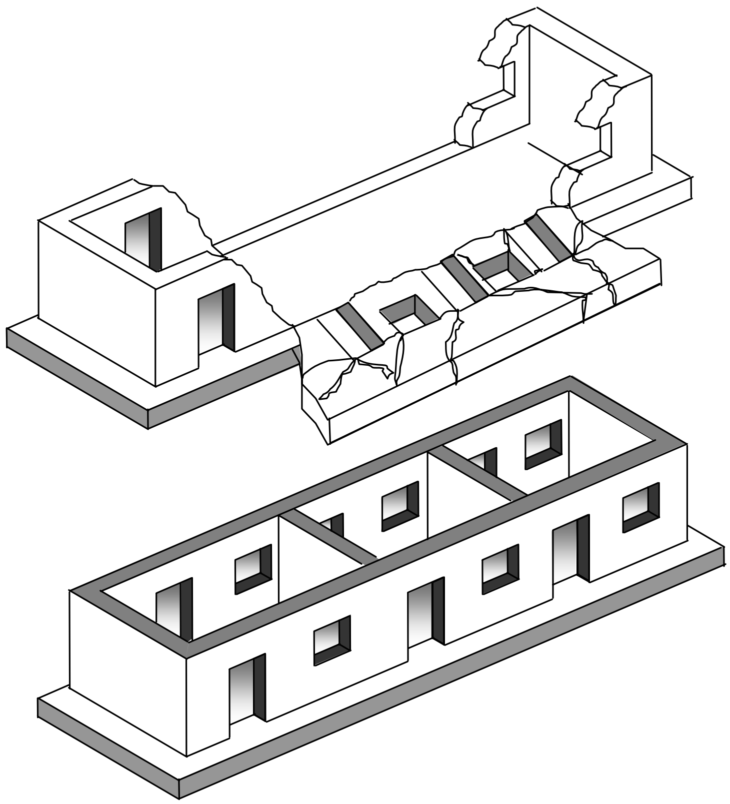


Figure 0.34 Cross walls protect a long wall from failing

0.4.5 Need for Horizontal Bands and Vertical Bars

In order to ensure integral action of the masonry walls in the load bearing construction, horizontal bands (preferably made of reinforced concrete) at the plinth, sill and lintel levels are provided. The absence of such bands, combined with poor bonding of walls at corner junctions, is a primary cause for collapse of many masonry buildings during an earthquake (Figure 0.35).

Figure 0.35 Lack of integrity due to absence of bands (Ref: www.nicee.org)

Also, roof and gable bands are required in the case of sloped roofs, along with truss-wall connections, to provide integral action between the roofs and walls. The complete set of horizontal bands and connections required for such buildings is illustrated in Figure 0.36.

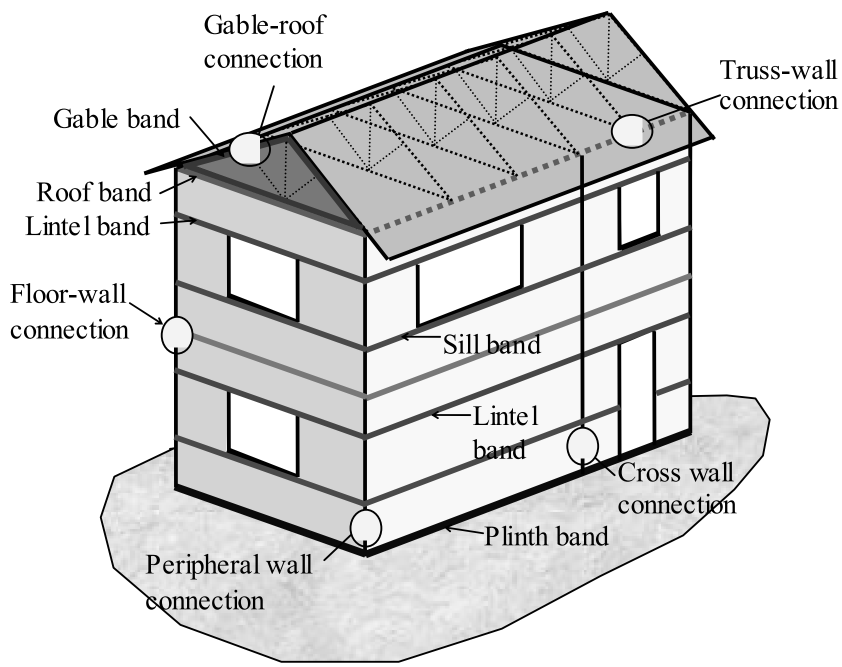


Figure 0.36 Bands at plinth, sill, lintel, roof and gable levels and other connections (Ref. 0.16)

The steel bars in the plinth, lintel and sill bands provide the desired tensile resistance to horizontal load by tying up elements together and strengthen the corner around doors and window openings, during bending in the horizontal plane. It is also desirable to have steel bars in the vertical direction at the edges of all walls segments. These bars provide the required tensile resistance due to in-plane bending in the wall segments (called wall piers) adjoining the door and window openings. The vertical steel bars should be anchored to the foundation and the roof bands. These details are illustrated in Figure 0.37. The horizontal bands build integrally in the building (making the walls act together) and hence reduce the tendency to collapse out of plane (Figure 0.38).

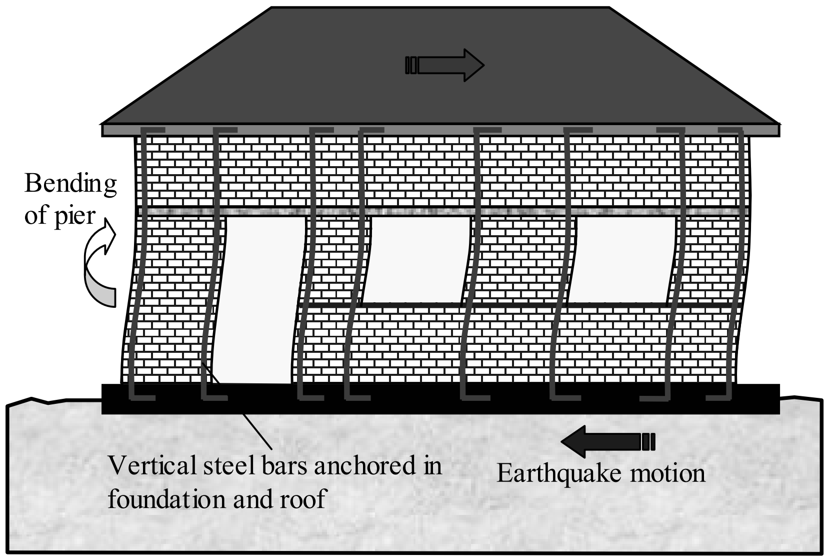


Figure 0.37 Provision of horizontal and vertical bars during earthquake (Ref. 0.16)

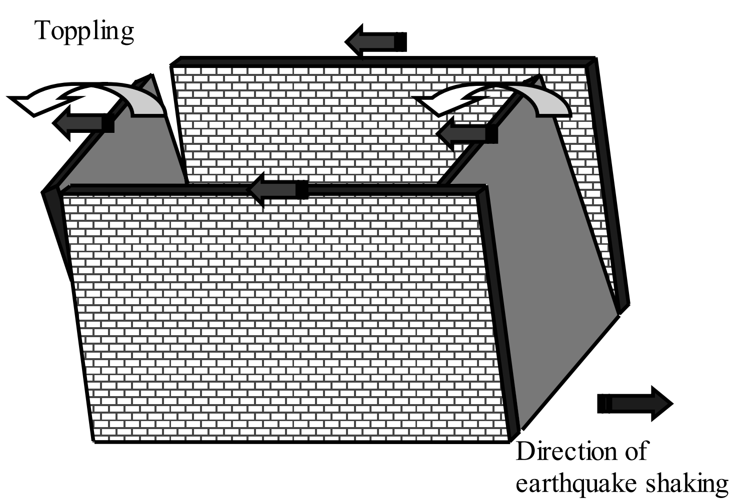


Figure 0.38 possible out-of-plane collapse of unconnected walls during an earthquake (Ref. 0.16)

In the absence of the above earthquake-resistance features in a masonry building, seismic vulnerability is indicated. Based on an assessment of this vulnerability, various retrofit means are possible, as described in the Section 0.6.

0.5 SEISMIC BEHAVIOUR OF REINFORCED CONCRETE BUILDINGS

0.5.1 Framed Buildings

As the building height increases, it attracts more and more lateral forces due to wind and earthquake. Conventional load-bearing walls become unviable in such buildings, as the masonry wall thickness required in the lower storeys is large. The provision of thick masonry walls is not only expensive, it also reduces the carpet area in the rooms, and increases the mass, thereby attracting higher seismic forces. In such situations, frames (a skeleton of beams, columns and foundation) are better suited than load-bearing walls as the structural system. In very high seismic zones (such as Zone V), it may be desirable to go for a framed construction even in single or two-storeyed buildings. In lower seismic zones (such as Zones II and III), load-bearing walls can be used up to two or three storeys, but they should be provided with the earthquake resistant-features as described in Section 4.

Framed construction in India is done commonly using reinforced concrete (rather than structural steel). In reinforced concrete, concrete which is strong in compression and weak and brittle in tension, is reinforced with steel bars, giving the composite material tensile strength and ductility. In beams and columns, which together make up a frame, transverse ties (with hooks) around the main reinforcement are also provided, to enhance strength in shear and ductility.

In typical multi-storeyed building construction, the frames along with the floor slabs, are constructed first. Walls made of bricks or other masonry units are introduced later. The main purpose of these walls, called infill walls, is to provide for enclosed spaces. The exterior walls also provide resistance to wind pressure. All infill walls contribute to increasing the lateral stiffness of the frames. However, they are liable to undergo diagonal cracks under a severe earthquake; this cracking is beneficial in dissipating some of the input energy due to the action of the earthquake on the building. However such infill walls, unless designed and constructed properly in the frame, may fail under seismic loads acting normal to the wall.

The seismic forces, acting effectively at the centres of mass of each floor of the building, cause the building to deflect laterally. Although the frame in a building is three-dimensional, it is only the parallel frames aligned in one direction that contribute to resisting lateral loads acting in that direction. Hence, it is important to ensure that there are adequate frames in the two perpendicular directions of a multi-storeyed building, more-or-less symmetrically distributed (to avoid twisting of the building). The horizontal floor slabs (made of reinforced concrete), being integrally connected with the frames, ensure that all the frames in any one direction effectively participate in resisting the seismic load in that direction. Similarly, the beams and columns in the frame should be properly tied together (Figure 0.13).

0.5.2 Buildings with Shear Walls

To enhance the lateral load resisting capacity, shear walls made of reinforced concrete are also provided, extending from the foundation level to the top. These shear walls contribute significantly to the lateral stiffness and the strength of the building. Reinforced concrete walls are far more superior in their performance compared to masonry walls because of their ability to resist large tensile forces without cracking significantly. In view of this, tall buildings with shear walls are preferred in high seismic zone regions. They act in combination with the frames in resisting lateral loads, but they attract much of the total seismic force by virtue of their higher stiffness. They should be provided in both directions and should be located close to the perimeter of the building, more-or-less symmetrically in plan, in order to avoid twisting of the building (Figure 0.39).

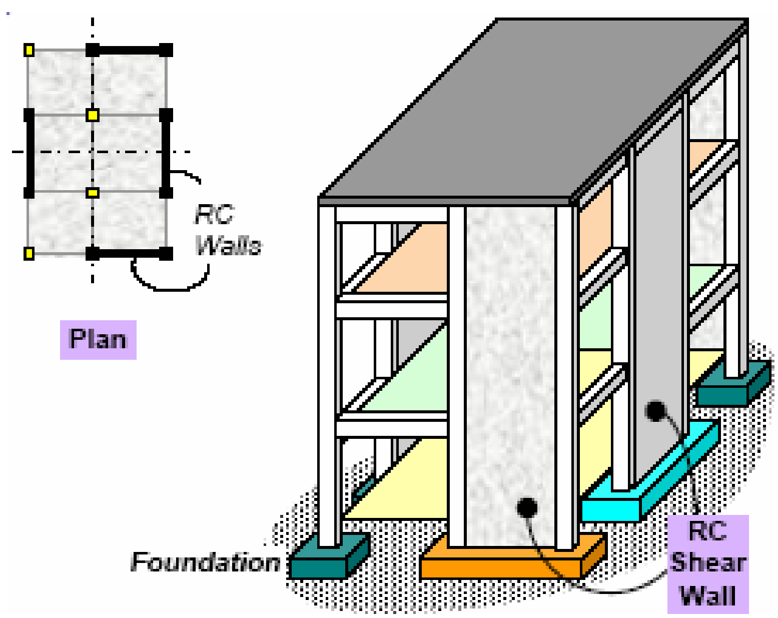


Figure 0.39 Reinforced concrete building with frames and shear walls (Ref. 0.16)

0.5.3 Failure of RC Framed Buildings

Many low-rise and medium-rise framed buildings have been constructed in the recent past, without proper attention paid in their design for wind or earthquake loads. This serious shortcoming in structural design and detailing has been exposed by the recent earthquakes in various parts of the country. And, there is now an increasing awareness about the need to design explicitly for seismic loads, in conformity with the prevailing codes of practices.

As mentioned earlier, foundations have to be stronger than the columns and the columns need to be stronger than the beams to avoid the global collapse of the building. Usually, the main cause of failures is the failure of the columns, particularly in the lower storeys. Columns and beams have to be adequately reinforced and detailed to prevent shear or bending failure. Buildings with open ground storey are particularly unsafe, as the stresses in the ground floor columns are very severe (Figure 0.15).

0.5.4 Failure of RC Columns

Columns in RC framed buildings may fail under an earthquake, either in shear or in bending. Shear failures occur mainly because the column sizes provided are inadequate to resist the seismic loads and also because of inadequate lateral ties provided. Bending (flexural) failures occur because of inadequate amount of steel bars provided vertically in the columns, particularly near the beam-column joints or column-foundation junctions, and may also occur due to poor quality of concrete.

A typical shear failure in a column, accompanied by wide diagonal cracks in both directions, is shown in Figure 0.40.

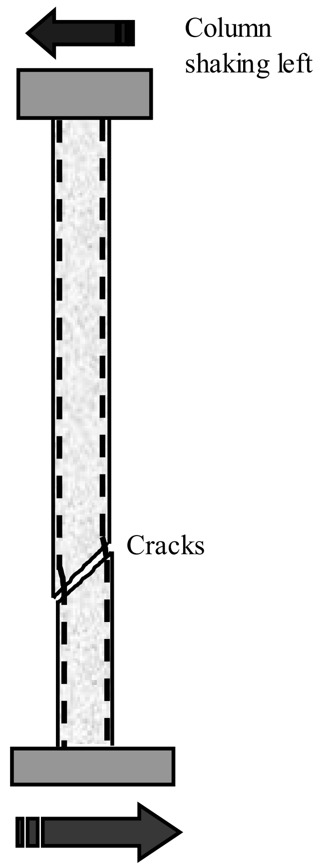


Figure 0.40 A typical shear failure in a column

The column shear failure, with diagonal cracking, can occur at any location along the height of the column. Figure 0.41 shows how devastating such a failure can be, accompanied by loss of concrete and buckling of reinforcement.

Figure 0.41 Shear failure in a reinforced concrete column (Ref: www.nicee.org)

Bending failure may occur either due to premature crushing of concrete in compression (which is undesirable, as shown in Figure 0.42a), or due to yielding of steel, accompanied by tensile cracking of concrete. A typical bending failure caused by yielding of bars on the tension faces near the top and bottom joints of the column, is shown in Figure 0.42. Usually, cracks will appear symmetrically on both sides because of the reversible cyclic nature of earthquake.

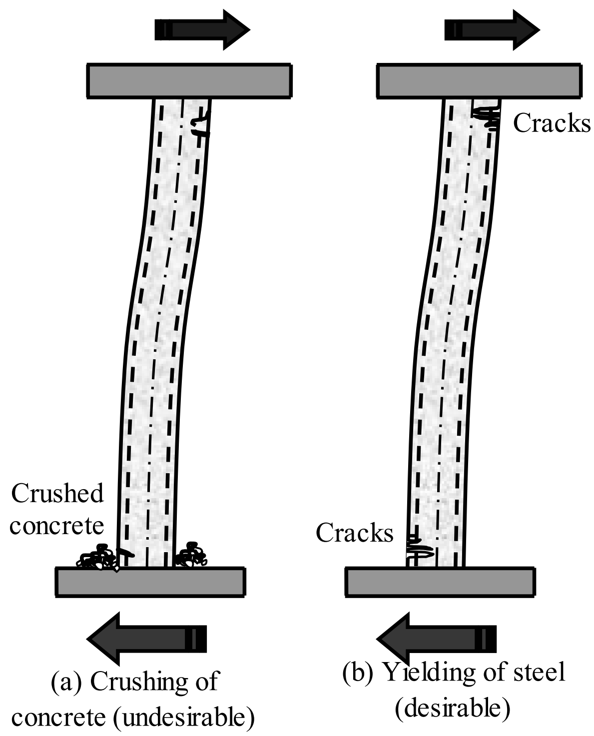


Figure 0.42 A typical bending failure in a column

0.5.5 Importance of Adequate Size, Reinforcement and Detailing of RC Columns

In order to avoid shear and bending failures, the columns should have adequate size and reinforcement, with proper detailing and closely spaced ties. The common practice of providing only 230 mm thick columns (no more than the thickness of a brick wall) is unsuitable for seismic resistance, especially in shear. The minimum size recommended is 400mm in one direction and about 600 mm in the perpendicular direction. Long and narrow columns (length greater than 2.5 times the width) should be avoided.

There is an Indian Standard Code of practice (IS: 13920 - 1993), which gives clear recommendations on how the reinforcements in beams and columns need to be tied with transverse steel ties, closely spaced, in order to provide ductility (Figure 0.13). Such detailing should be strictly followed in earthquake-resistant framed constructions, especially when they are designed for reduced seismic forces based on the assumed ductile behaviour (Figure 0.14). In particular, it is necessary to provide ties inside the column at the beam-column joint for ductility. Also, splicing of the vertical bars should be avoided at the floor levels, and should ideally be at the column mid-height location.

0.5.6 Design and Detailing of RC Beams

Beams may also fail either in shear or in bending. Shear failure is undesirable, as it limits the load resisting capacity and prevents the yielding of the longitudinal steel (ductile behaviour). Shear failures occur mainly because of inadequate lateral ties provided. Bending or flexural failures occur because of inadequate amount of horizontal steel bars, or inadequate anchorage of the bars, particularly at the bottom near the beam-column joints, or poor quality of concrete. A typical flexural failure in a beam and the failure of the supporting column are shown in Figure 0.43.

Figure 0.43 Failures in beam and column (Andaman earthquake, 2004)

Reinforced concrete beams should be carefully designed, with adequate reinforcement at top and bottom (to resist bending action) and stirrups (to resist shear action) that are closely spaced near the supports and provided with proper hooks. The horizontal bars should be well anchored into the columns; otherwise, they will slip leading to failure. Under gravity loads, typically, the beams sag in the middle (requiring steel bars at the bottom) and hog near the column supports (requiring steel bars at the top). Under an earthquake, this hogging action increases at one end, but decreases and sometimes reverses to sagging at the other end. This possibility of reversal of stresses at the supports of beams must be accounted for in design and detailing.

Since the failure of a beam is less catastrophic than the failure of a column, the design should be such so as to have the supporting column stronger than the beam. Otherwise, in frames where the beams are strong and the columns weak, failure of the columns can lead to collapse of the storeys one over the other (pancaking effect), as shown in Figure 0.44.

Figure 0.44 'Pancaking' failure due to failure in columns (Bhuj earthquake, 2001) (Ref: www.nicee.org)

0.5.7 Integration of Shear Walls and Staircases

Reinforced concrete shear walls, commonly provided around lift cores, are usually very strong and stiff. However, unless these shear walls are integrally connected to the rest of the building through proper framing, they will not serve the desired purpose. Figure 0.45 illustrates such a situation where the lift core alone is found to survive with relatively little damage, but the rest of the building separated out due to lack of integrity and collapsed. Staircases should be well framed and integrated to the rest of the building, in view of their extreme importance during an earthquake (Figure 0.26), and preferably be located symmetrically in a building.

Figure 0.45 Lack of integrity between lift core and rest of the building (Bhuj earthquake, 2001) (Ref: www.nicee.org)

0.5.8 Columns with Unequal Heights in a Storey

When columns with unequal heights are provided in a storey, such as in the ground storey of a building located on a sloping site (Figure 0.46), then the shorter columns being stiffer attract more horizontal forces and are liable to fail in shear, unless they are specially designed for these effects.

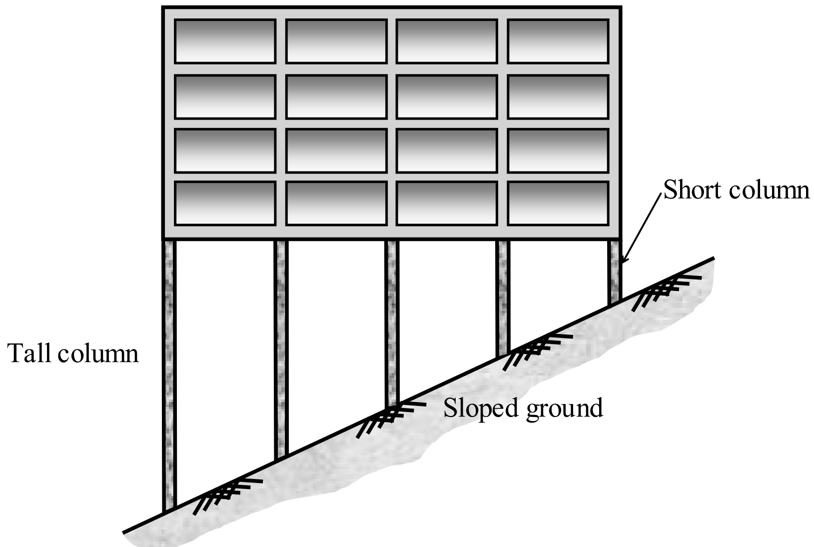


Figure 0.47 "Short column" effect due to sloping site (Ref. 0.16)

This "short column" effect is also encountered when the effective height of some columns in a particular storey is reduced on account of provision of an intermediate (mezzanine) floor (Figure 0.47). This can also happen when masonry infill walls are provided with openings adjoining columns at some locations (Figure 0.48). This effect is often not anticipated in design, and can result in a column shear failure. It is best to avoid such construction practices.

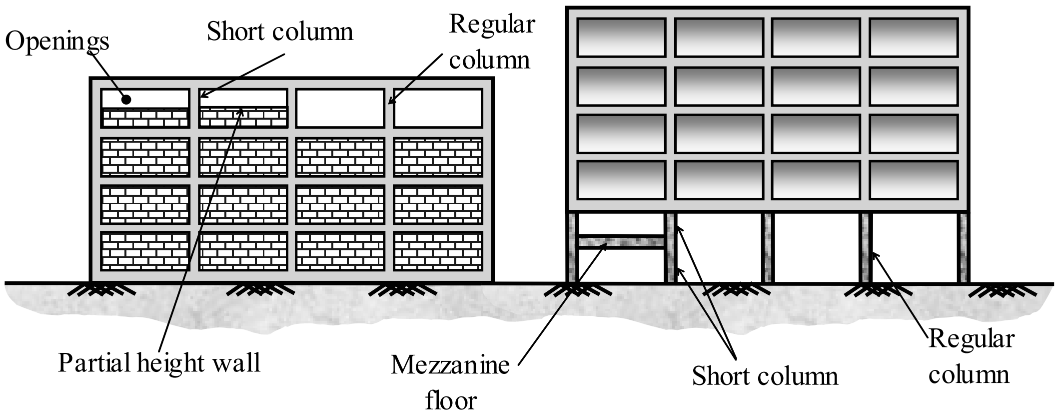


Figure 0.48 "Short column" effect due to openings in masonry infill walls or mezzanine floor (Ref. 0.16)

0.5.9 Torsional Effects

In the case of buildings with irregularities in plan and elevation, the resultant line of action of the seismic force is likely to be eccentric to that of the resisting force (i.e., the *centre of mass* is eccentric to the *centre of rigidity*). In such cases, the building will not only sway to-and-fro sideways, but also twist in plan. This twisting action introduces torsional effects, that must be specially accounted for in the structural analysis and design.

As mentioned earlier, it is best to avoid such problem, by proper planning of the layout and configuration of the building. Otherwise, it is extremely important that such buildings should be designed for the additional torsional effects. Often this is neglected in practise, and this makes such buildings vulnerable.

0.5.10 Connections in Pre-cast Concrete Buildings

Pre-cast concrete buildings are suitable for rapid and economic construction of buildings in a mass scale. All the main components (columns, beams, slabs and wall panels) are factory-made and installed at site, with in-situ concreting done only at the beam-column and roof-wall panel junctions. Sometimes bolts and other fastening devices are used to provide connections.

In pre-cast concrete buildings, special attention should be given to the design and detailing of joints inter-connecting the various components. When the building undergoes severe shaking during an earthquake, all these connections will be put to test. If they fail, the building will simply fall apart (Figure 0.49).

Figure 0.49 Lack of integrity in pre-cast construction (Bhuj earthquake, 2001) (Ref: www.nicee.org)

0.6 HOW EARTHQUAKE-SAFE IS YOUR BUILDING?

The descriptions and glimpses provided by pictures of destruction of ordinary buildings shown in the previous sections should serve as a wake-up call to pose this important question:

How earthquake-safe is your building?

If we do not address this question, and take appropriate action to retrofit, if necessary, we have only ourselves to blame, if things go wrong, when the next earthquake hits our region. We may face two different situations:

1. Safety of an existing building in which we live;
2. Safety of a new building that we are planning to invest in.

These are discussed in the sections to follow. But before that, we need to understand that no building can ever be 100 percent safe against earthquake loading, and get built economically. There is always some risk of failure involved. Based on past experience and understanding of the structural behaviour of buildings, certain "acceptable" levels of risk in structural design have been recommended.

0.6.1 How Much Risk is "Acceptable"?

The risk involved depends on the "level" of the earthquake. The seismic zone and site characteristics will govern the prescribed maximum level of the earthquake. But the chance of such an extreme earthquake occurring during the life of the building can be very low. There are lower levels of earthquake that are more likely to occur.

Figure 0.50 illustrates an approximate relationship between expected damage and the earthquake level for ordinary buildings as per standard practice recommended in design codes. We should expect the building to experience no damage when it is subject to tremors that have a good chance of occurring during the "service life" of the building. The term, serviceability earthquake, is sometimes used to refer to an earthquake level that has only a 50% chance of being exceeded during a period of 50 years. For earthquake levels that are rare (with lower probability of occurrence), we can accept the risk of some damage. In an extreme event, called maximum considered earthquake (MCE), which has only a 2% chance of being exceeded in 50 years, we could even expect very serious damage to the building, but not complete collapse, so that lives are not lost. We must be prepared to expect some nominal damage to the building at earthquake levels slightly higher than the serviceability earthquake and increasingly more serious damage at levels approaching MCE.

Of course, we can have our buildings designed for "no damage" even at MCE, but this would be prohibitively expensive! We contemplate doing this only in the case of lifeline buildings or major structures such as hospitals and nuclear reactors.

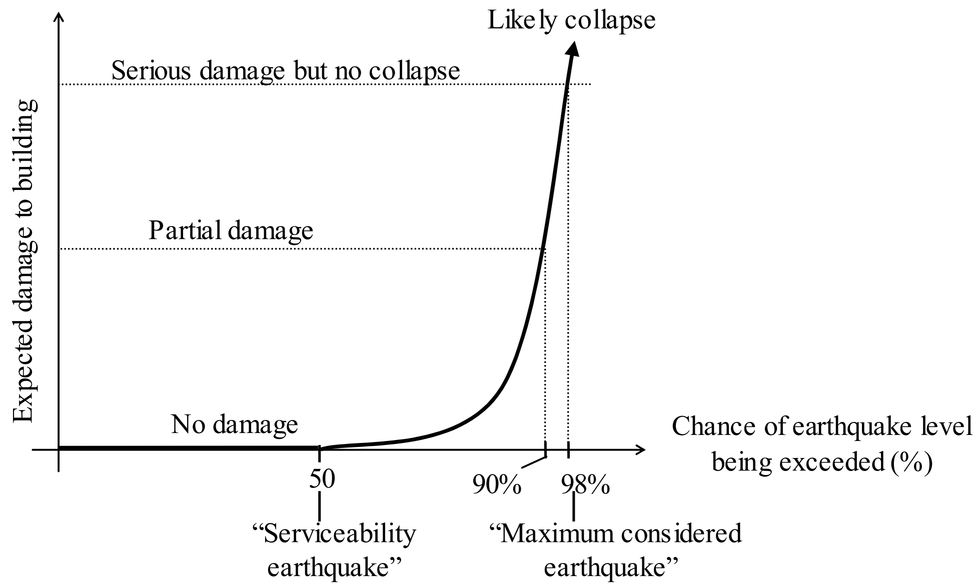


Figure 0.50 Relationship between expected damage to building and earthquake level for ordinary buildings designed to accepted standards

0.6.2 Assessment of Existing Buildings

Existing buildings may not comply with requirements of the prevailing standards due to various reasons. The condition of the buildings may also have deteriorated (cracking, corrosion, etc.) over the years, in the absence of repair and maintenance. The buildings may not have been designed specifically to resist earthquake loads. Even if designed, the seismic resistance provided may not measure up to the current code requirements (which are periodically upgraded). For example, many old buildings in Chennai, classified as Zone II until 2002, are likely to be found to be deficient after the recent upgradation to Zone III.

It is possible today to have our building assessed properly to see whether or not they meet the acceptable seismic risk, and if they do not, to have the buildings retrofitted to the level required. The *Handbook for Seismic Retrofit* provides the necessary resources to facilitate not only risk assessment (in terms of code compliance), but also various methods of retrofit in order to minimise the risk of failure.

There are several steps involved in the evaluation of the seismic resistance of an existing building. First, a quick assessment can be carried out by a procedure called rapid visual screening. This procedure helps the inspectors to identify and inventory the vulnerable buildings. When a building is identified as vulnerable by rapid visual screening, relevant data is acquired by filling up the data collection forms. Next, the preliminary evaluation is conducted. It involves a set of initial calculations to identify the potential weaknesses in the building. If this preliminary evaluation indicates the need, a more detailed structural evaluation is carried out by a structural engineer to quantify the extent of inadequacy, with respect to the prescribed standards (design codes). For detailed structural evaluation, it is necessary to have an assessment of the existing condition (condition assessment), including all aspects of observed deterioration of the building.

After the evaluation, a decision has to be taken on whether or not to retrofit. Some guidelines are given in the following sections on the desired level of retrofit and the various options of retrofit (retrofit strategies). The decision to retrofit and selection of a retrofit scheme (combination of retrofit strategies), of course, will be governed by the importance of the building, estimated cost of retrofit, disruption to the use of building and available technology. If the cost is affordable and the other considerations are viable, then it would be wise to implement the retrofit scheme immediately. If the existing building is found to be very unsafe, and the cost of retrofit is prohibitively high, retrofit may not be a viable option. Of course, continued occupancy of the building should not be encouraged. In the interest of safety to human lives, such buildings should be abandoned, if not demolished and rebuilt.

0.6.3 Making New Buildings Earthquake-Safe

We have no excuse when it comes to new buildings that we plan to invest in. We simply must ensure that they are properly designed and constructed, meeting the prevailing seismic code requirements. How do we ensure this? We must demand this of the builder. Indeed, this is our legal right as consumers, and the responsibility of the builder. Builders usually give all kinds of assurances about safety, as do advertisements for some building materials, such as bricks and cements that are claimed to be "earthquake-proof". The reality is that materials by themselves cannot be safe against earthquakes; they get damaged when their strength is exceeded, and the building may collapse when this happens, if not properly designed, detailed and built.

In this introductory chapter, many suggestions have been given regarding architectural layout and structural design and detailing for earthquake resistance. The users will do well to verify that these features have been incorporated in the building. If not, they must not hesitate to ask the builder pointed questions on these aspects. It is always desirable to have proper documentation of drawings and design calculations, and the builder must be asked to furnish these. In the case of multi-storeyed buildings, it is also desirable to have the structural design proof-checked by a competent third party for code compliance. Finally, abundant care must be taken at the time of construction to ensure that the detailing is implemented as indicated in the drawings, that the materials used are of good quality, and that all the construction activities are carried out under competent supervision.

0.7 INTRODUCTION TO SEISMIC RETROFIT

0.7.1 To Retrofit or Not?

The purpose of seismic retrofit is to enhance the structural capacities (strength, stiffness, ductility, stability and integrity), so that the performance of the building can be raised to the desired level to withstand the design earthquake. A decision on whether or not to retrofit an unsafe building depends on many factors. Lifeline buildings, such as hospitals must necessarily be retrofitted, in view of their extreme importance. Otherwise, they may meet the tragic fate of the Bhuj District Hospital complex (Figure 0.51).

Figure 0.51 Ruins of the Bhuj District Hospital (Bhuj earthquake, 2001) (Ref: www.nicee.org)

It is likely that many ordinary buildings will be found seismically unsafe. Retrofitting all such buildings is a major task that many building corporations and government have to grapple with. The financial implication of such a mammoth task can be mind boggling. Who will bear the cost?

Usually, in the case of private buildings, the owners of the building will have to bear the cost of seismic retrofit. This cost must be weighed against the cost of not retrofitting. What is the risk involved? Studies have shown that the risk of collapse of an existing building whose strength is less than that of a new building (designed to current standard) can be correlated to that of the new building.

As illustrated in Figure 0.52, if the seismic strength of an existing building (or structural component) is only 33% of that required by the current standard for a new building, the risk involved is as high as about 20 times that of the new building. If the strength is two-thirds that required by the current standard, the risk reduces to three times the standard risk: this level of risk is generally considered as the limit of acceptable risk. Hence, it is recommended that seismic retrofit be necessarily undertaken when the strength of an existing building (in term of the total seismic load that it can resist without collapsing) drops below about 70% of the capacity required by the current standard.

If it is not viable to carry out seismic retrofit for financial reasons or other considerations, then we must accept and live with the risk involved (Figure 0.52). When the risk is too high (more than 10 times the standard risk), building authorities should prohibit human occupancy of the building in the interest of public safety. The building should be demolished and reconstructed when financial resources become available.

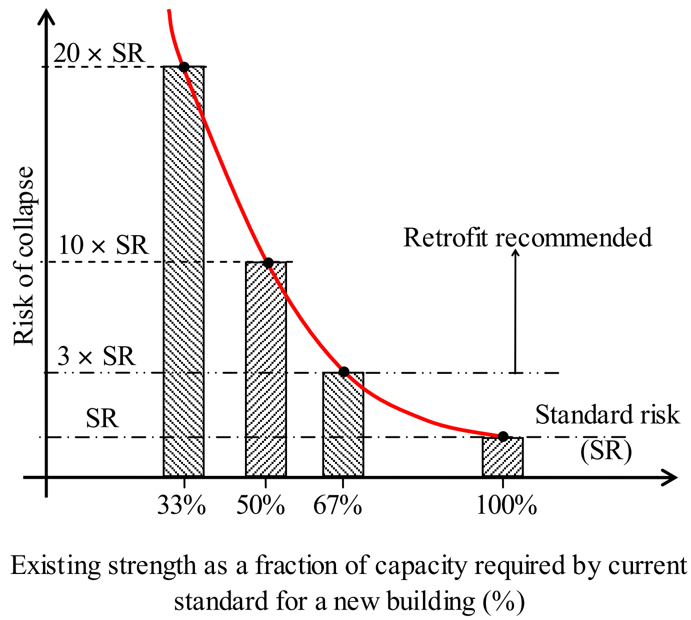


Figure 0.52 Seismic load capacity versus risk of building collapse

0.7.2 Extent of Seismic Retrofit

As discussed earlier, the strength of an old building cannot be reasonably expected to be the same as that of a new building designed to current standards. It is recommended that seismic retrofit be taken up if the seismic load resisting capacity of the building is less than about 70 percent of that required by the current standard.

But should the aim of retrofit be to raise this capacity to 100 percent (that is, equal to that required by the current standard)?

The answer to this question depends on the importance of the building. For lifeline buildings such as fire stations, hospitals, power stations, telephone exchanges, television stations, radio stations, railway stations, stations for rapid transit system, air ports; important service and community buildings such as schools, cinema halls, multiplexes, marriage and other assembly halls, the capacity should be raised to 100 percent. For other buildings, it can be decided based on the expected remaining usable life (RUL) of a building, expressed in terms of the original design life (ODL) of the building. As indicated in Figure 0.53, if the building is relatively old and has lived more than 50 percent of its design life, it should be retrofitted to resist at least 70 percent of the total design seismic load as per the current standards. This minimum level of retrofit should be raised to at least 77, 84, 89 and 95 percents of the total design seismic load if the remaining usable life is 60, 70, 80 and 90 percents of the original design life, respectively.

The expected design life of any building depends on its type and quality of construction and can vary from about 10 years for some types of non-engineered construction to about 100 years for high quality engineered construction. For a typical reinforced concrete building, the design life may be taken as 50 to 60 years.

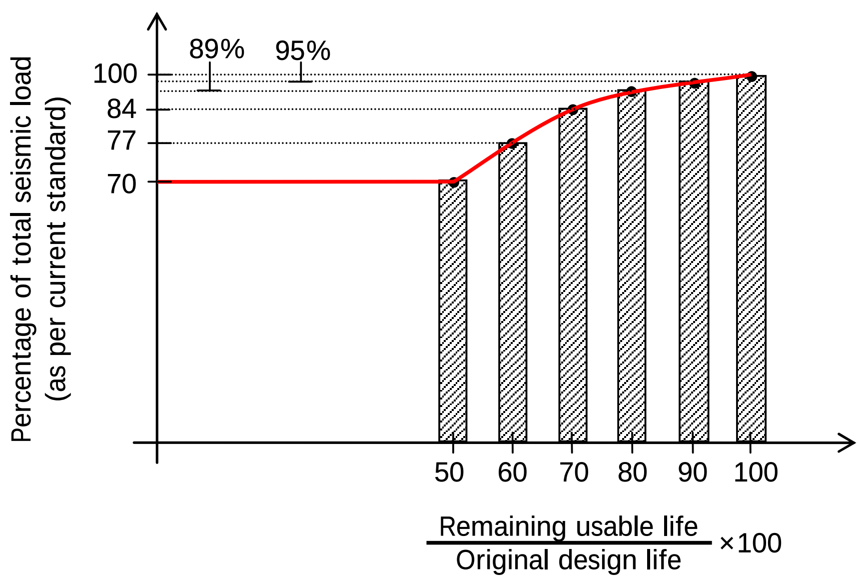


Figure 0.53 Design seismic load as a function of remaining usable life

0.7.3 Local versus Global Retrofit

In order to provide increased lateral stiffness and strength to the building as a whole, global retrofit strategies have to be planned, wherever required. This is done in order to ensure that a total collapse of the building does not occur, even if some structural components in the building may get damaged.

In order to avoid failure of the components, and also thereby enhance the overall performance of the structure, local retrofit strategies also need to be planned.

More than one combination of local and global retrofit strategies is possible, having different cost implications. The structural engineer must work out an appropriate, practically viable and economical solution. The *Handbook of Seismic Retrofit* provides specific guidelines on these local and global retrofit strategies, as applicable to non-engineered buildings, masonry buildings, historical and heritage structures, reinforced concrete buildings and steel buildings.

Some of the important strategies relevant to non-engineered buildings, masonry buildings and reinforced concrete buildings are highlighted in the following two sections.

0.8 RETROFIT OF NON-ENGINEERED AND MASONRY BUILDINGS

0.8.1 Strengthening of Walls, Floors and Roofs

The seismic resistance of the walls of small buildings such as dwelling units can be enhanced by providing containment reinforcement as shown in Figure 0.51. Horizontal, vertical and cross bars are inserted in grooves which are subsequently covered by mortar. Perpendicular walls can be stitched to enhance the integrity. Figure 0.52 shows the stitching by steel bars inserted at an angle of 45°. The drilled holes are subsequently filled with cement grout. An arch can be relieved of the compressive stress by inserting steel beams above it, as shown in Figure 0.53.

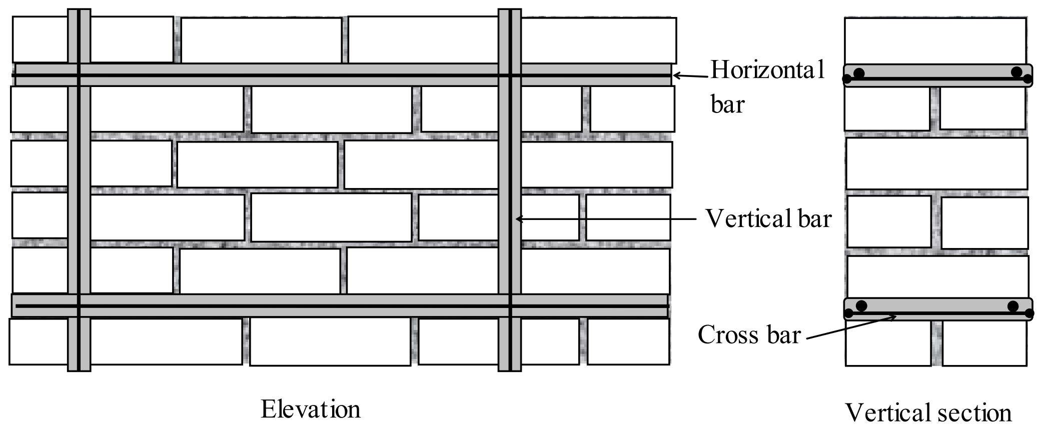
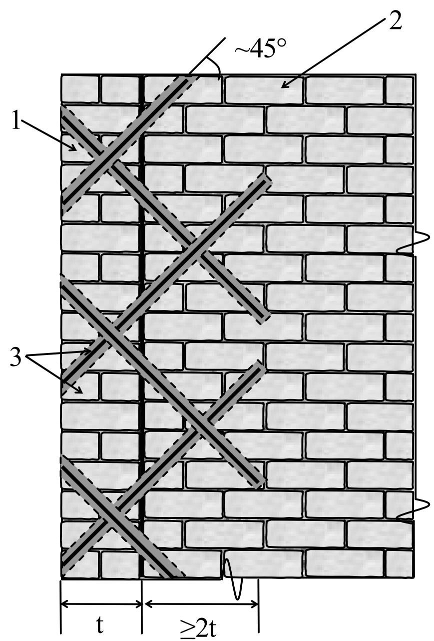


Figure 0.51 Containment reinforcement for strengthening wall



1. Transverse wall
2. Longitudinal wall
3. 10 mm diameter bars in 20 mm holes filled with cement grout

Figure 0.52 Connecting perpendicular walls with inclined bars

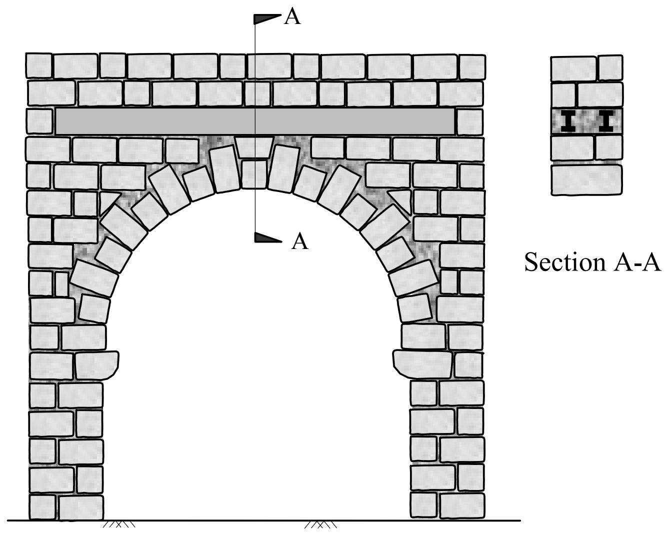
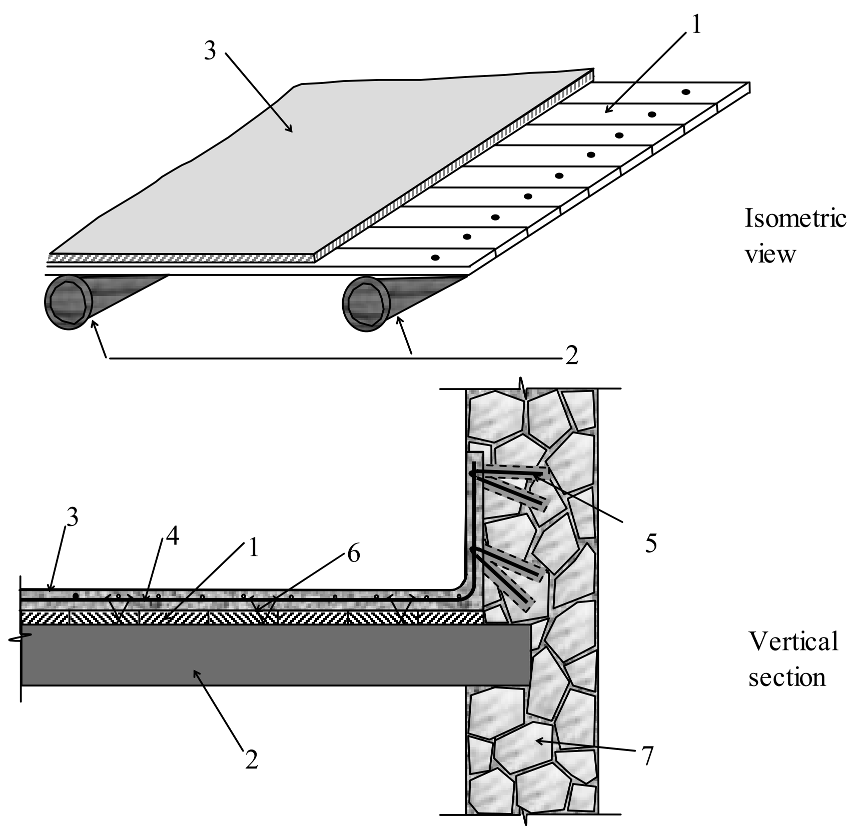


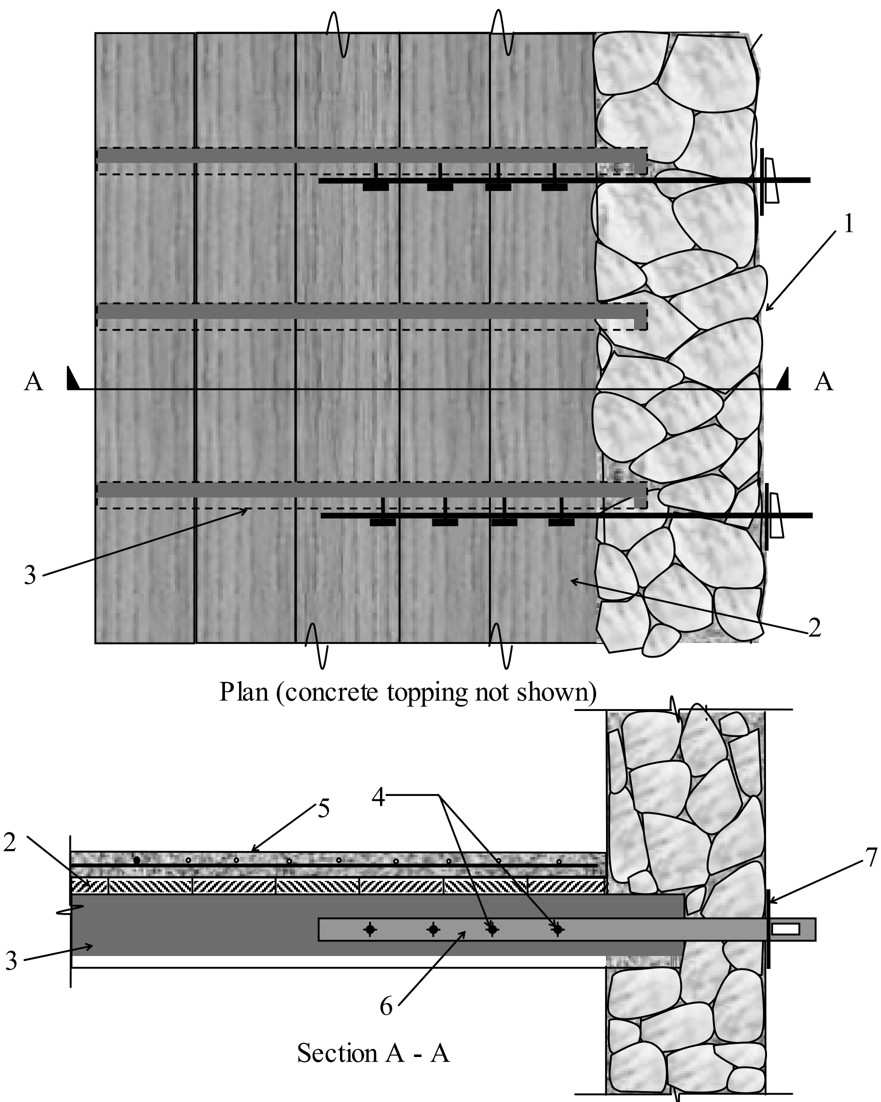
Figure 0.53 Stress relieving of arch by inserting steel beams

The connection of an intermediate wooden floor or roof with the wall can be strengthened to enhance the integrity (Figure 0.54). A concrete overlay with mesh reinforcement is cast on the wooden floor. The overlay is extended as a skirting on the wall, where the mesh is anchored. Nails are provided to attach the mesh with the wooden floor. An alternative strategy of strengthening the connection is shown in Figure 0.55. Steel flats are nailed on the existing rafters and anchored to the wall.



1. Existing wooden planks
2. Existing wooden rafters
3. New concrete topping
4. Steel mesh reinforcement
5. Steel anchors grouted in drilled holes
6. Nails
7. Existing wall

Figure 0.54 Strengthening of wooden floor



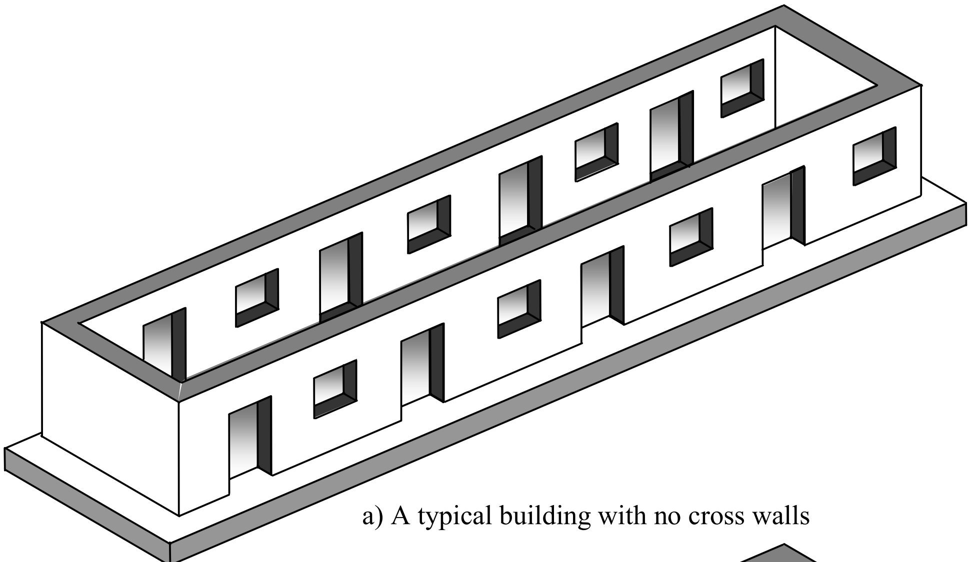
1. Existing wall
2. Existing wooden planks
3. Existing wooden rafters
4. Nails
5. New concrete topping
6. Steel flats
7. Anchor plates

Figure 0.55 Strengthening of wooden floor (alternate)

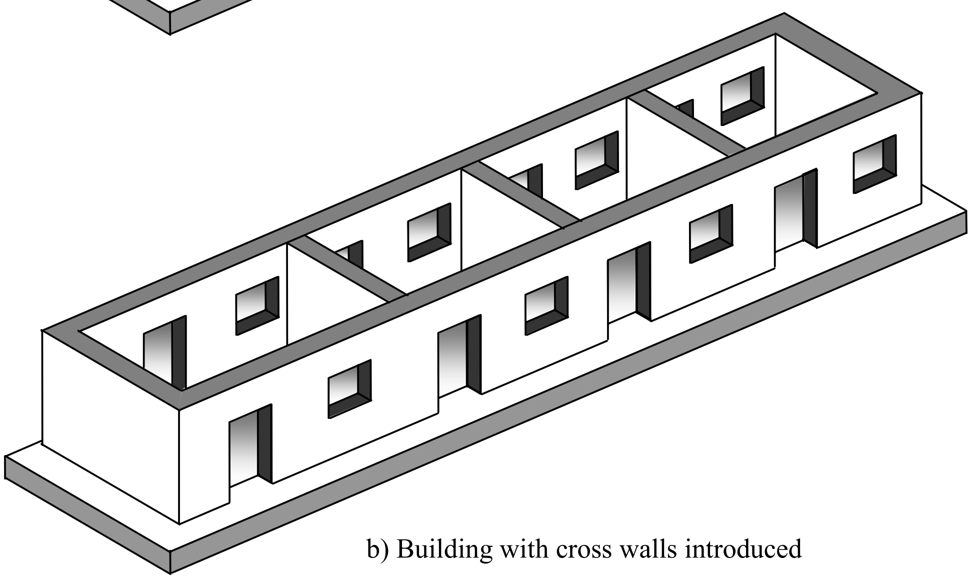
0.8.2 Techniques for Global Strengthening

The layout and configuration of a masonry building can be improved by introducing joints between perpendicular blocks, similar to that shown in Figure 0.20. The span of long walls can be reduced by introducing cross walls or buttresses. Figure 0.56 shows the improvement of a “barrack” type of building with sufficient cross walls. For huts, the lateral strength can be significantly improved by providing cross braces (Figure 0.57).

For integrity of the building, the provision of bands at plinth, lintel or roof levels is illustrated in Figure 0.36. The requirement of vertical steel bars at the corners of the building and adjacent to the openings is shown in Figure 0.37. Similar provisions can be introduced in an existing building by the “splint and bandage strengthening technique”. Strips of steel mesh are attached to the exterior surfaces of the walls and subsequently covered by mortar (Figure 0.58). The vertical and horizontal strips are termed as splints and bandages, respectively. There should be continuity of the horizontal strips at the corners.



a) A typical building with no cross walls



b) Building with cross walls introduced

Figure 0.56 Introduction of cross walls

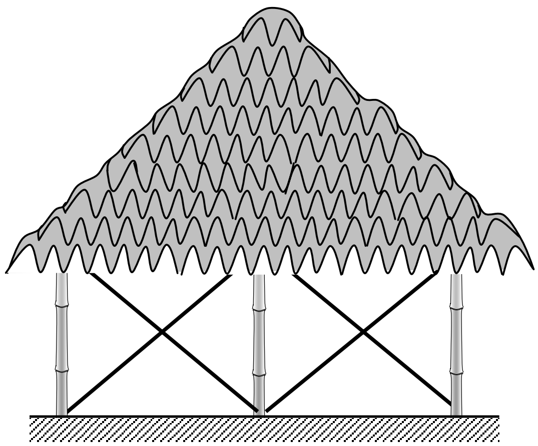


Figure 0.57 Introduction of braces in a hut

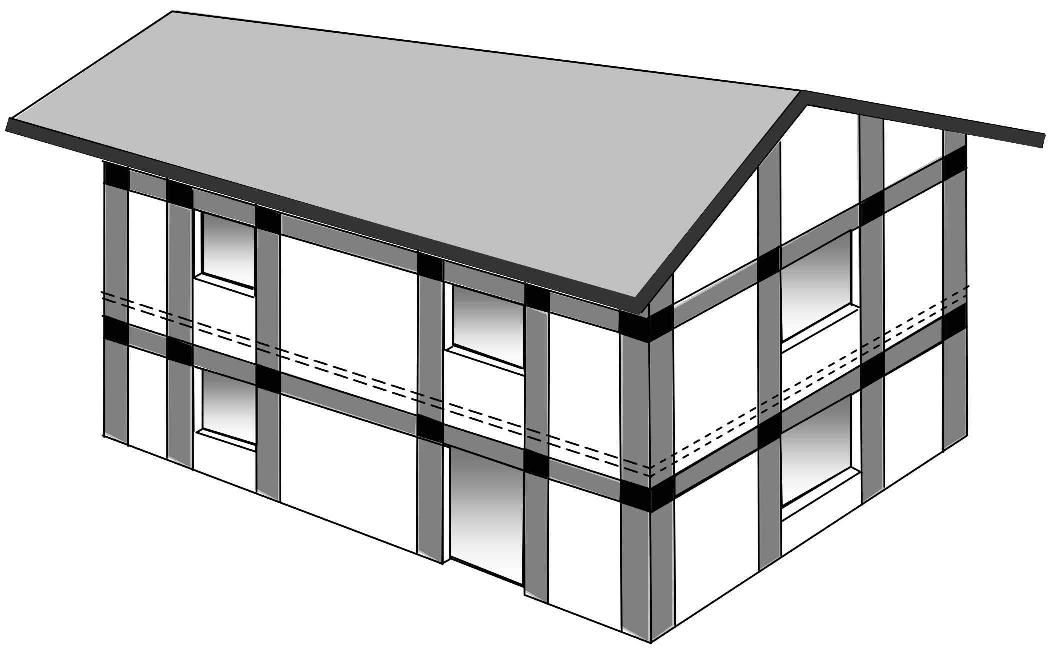


Figure 0.58 Splint and bandage strengthening technique

0.9 RETROFIT OF REINFORCED CONCRETE BUILDINGS

The retrofit strategies can be grouped under global and local strategies. A global retrofit strategy targets the seismic resistance of the building. A local retrofit strategy targets the seismic resistance of a member, without significantly affecting the overall resistance of the building. It may be necessary to combine both local and global retrofit strategies under a feasible and economical retrofit scheme.

0.9.1 Global Retrofit Strategies

When a building is severely deficient under the design seismic forces, the first step of seismic retrofit is to strengthen and stiffen the structure by providing additional lateral load resisting elements. Additions of infill walls, shear walls or braces are grouped under global retrofit strategies. A reduction of an irregularity or of the mass of a building can also be considered under global retrofit strategies.

Addition of infill walls in the ground storey is a viable option to retrofit buildings with open ground storeys. In absence of plinth beam, the new foundation of the infill wall should be tied to the existing footings of the adjacent columns (Figure 0.59). Else, a plinth beam can be introduced to support the wall.

Shear walls can be introduced in buildings with frames or in buildings with flat slabs or flat plates. In the latter type of buildings, since there are no conventional frames, the lateral strength and stiffness can be substantially inadequate. A new shear wall should be provided with an adequate foundation. The reinforcing bars of the wall should be properly anchored to the bounding frame (Figure 0.60).

Steel braces can be inserted in a frame to provide lateral stiffness, strength, ductility, energy dissipation, or any combination of these (Figure 0.61). The braces can be added at the exterior frames with least disruption of the building use. For an open ground storey, the braces can be placed in appropriate bays to retain the functional use. The connection between the braces and the existing frame is an important consideration. One technique of installing braces is to provide a steel frame within the designated bay. Else, the braces can be connected directly to the frame with plates and bolts.

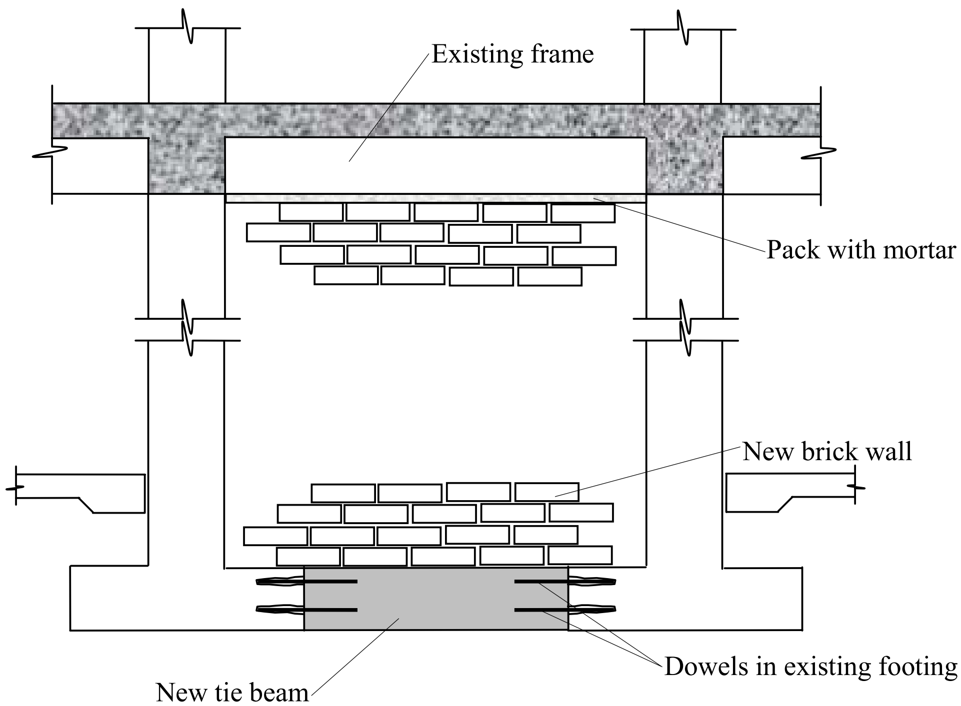


Figure 0.59 Addition of a masonry infill wall

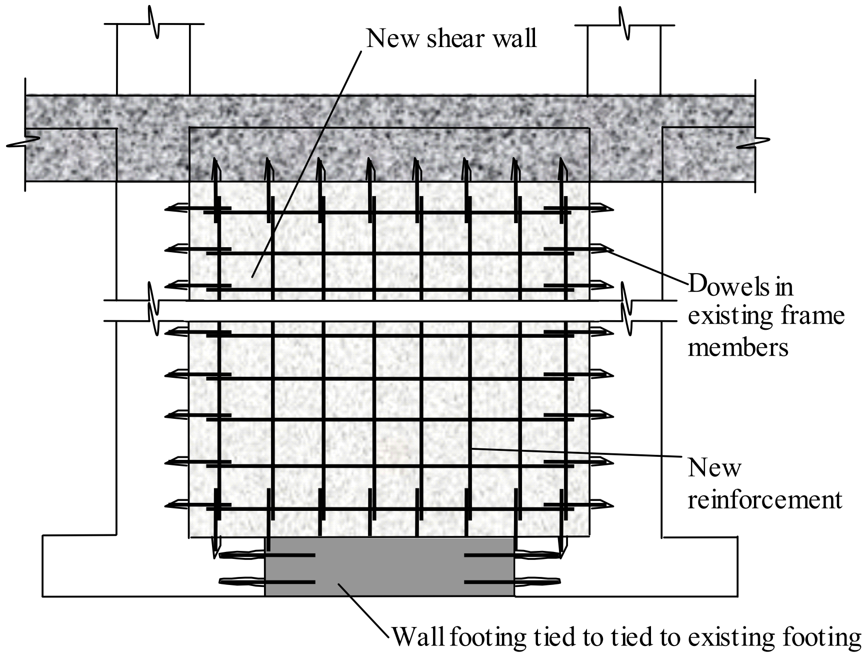


Figure 0.60 Addition of a shear wall

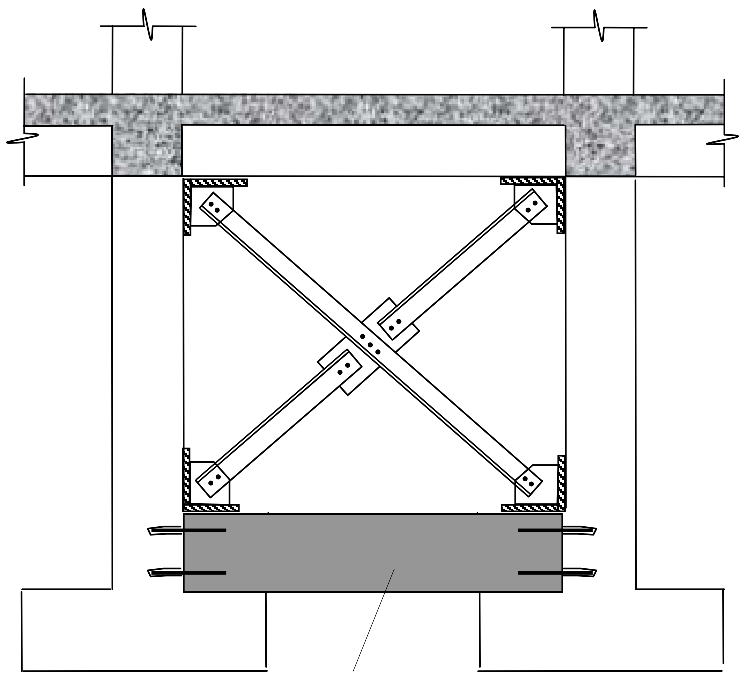


Figure 0.61 Addition of steel braces

0.9.2 Local Retrofit Strategies

Local retrofit strategies refer to retrofitting of columns, beams, joints, slabs, walls and foundations. The local retrofit strategies fall under three types: concrete jacketing, steel jacketing (or use of steel plates) and fibre-reinforced polymer (FRP) sheet wrapping.

Concrete jacketing involves addition of a layer of concrete, longitudinal bars and closely spaced ties (Figure 0.62 and 0.63). The jacket increases both the flexural strength and shear strength of the column or beam.

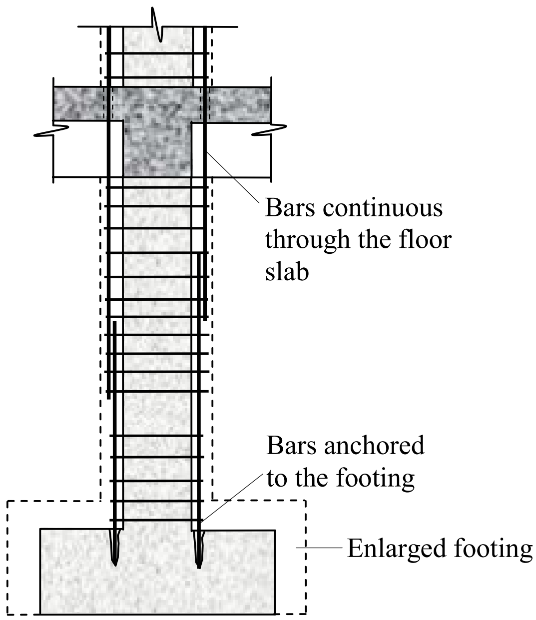


Figure 0.62 Concrete jacketing of a column

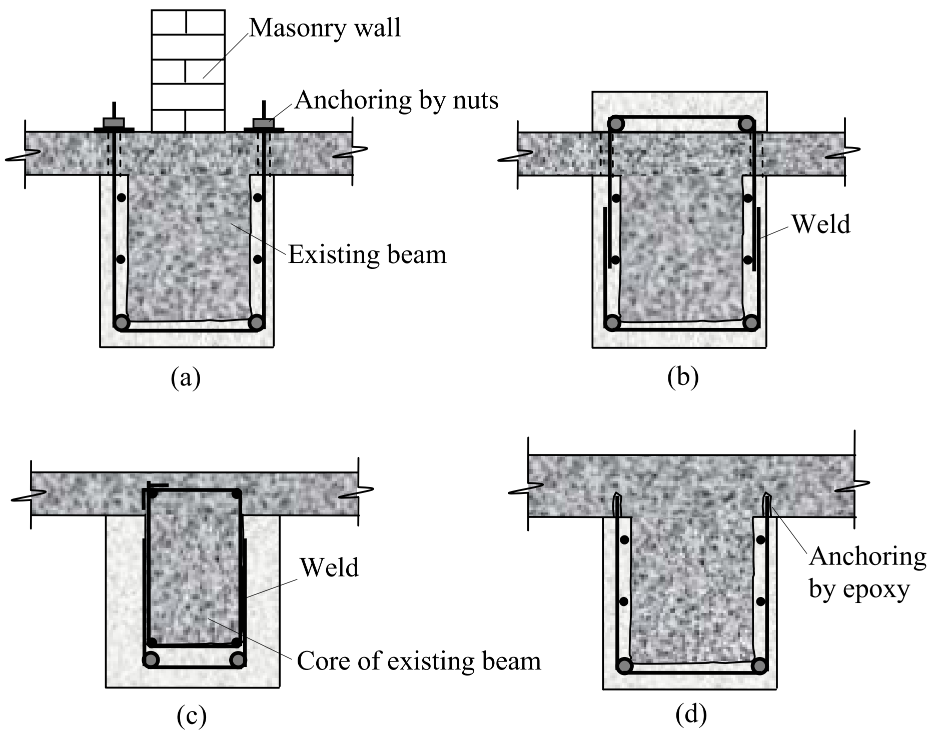


Figure 0.63 Concrete jacketing of beams

Steel jacketing of column refers to encasing the column with steel plates and filling the gap with non-shrink grout. The jacket is effective to remedy inadequate shear strength and provide confinement to the column. Since the plates cannot be anchored to the foundation and made continuous through the floor slab, steel jacketing is not used for enhancement of flexural strength. As a temporary measure after an earthquake, a steel jacket can be placed before an engineered scheme is implemented. Different types of steel jacketing are illustrated in Figure 0.64.

Steel sheets are used in beams to enhance their flexural or shear strengths. The enhancement of flexural strength is possible for the sagging behaviour at the central region of a beam. A steel sheet is bonded or bolted at the bottom face of the beam. This is considered for strengthening a beam for gravity loads. For seismic load, the shear strength can be enhanced by bonding or bolting sheets on the side faces near the two ends of the beam (Figure 0.65).

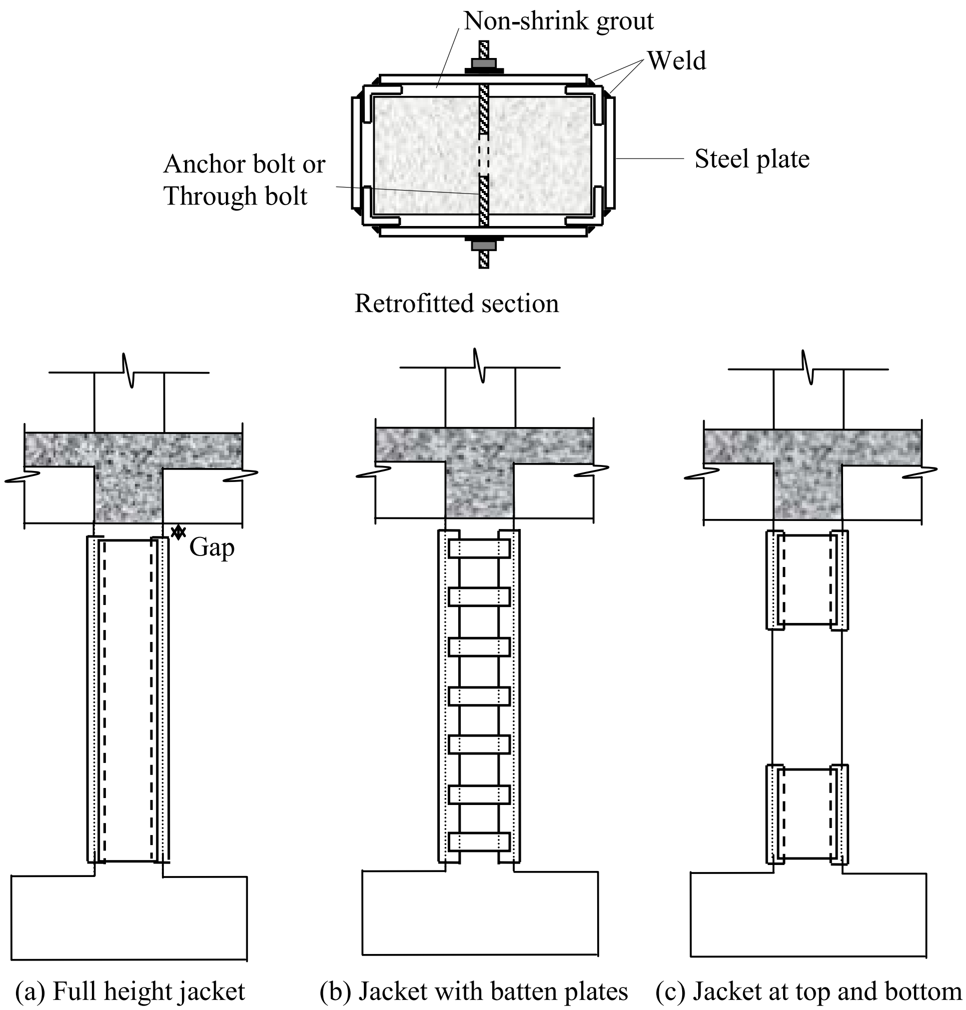


Figure 0.64 Steel jacketing of columns

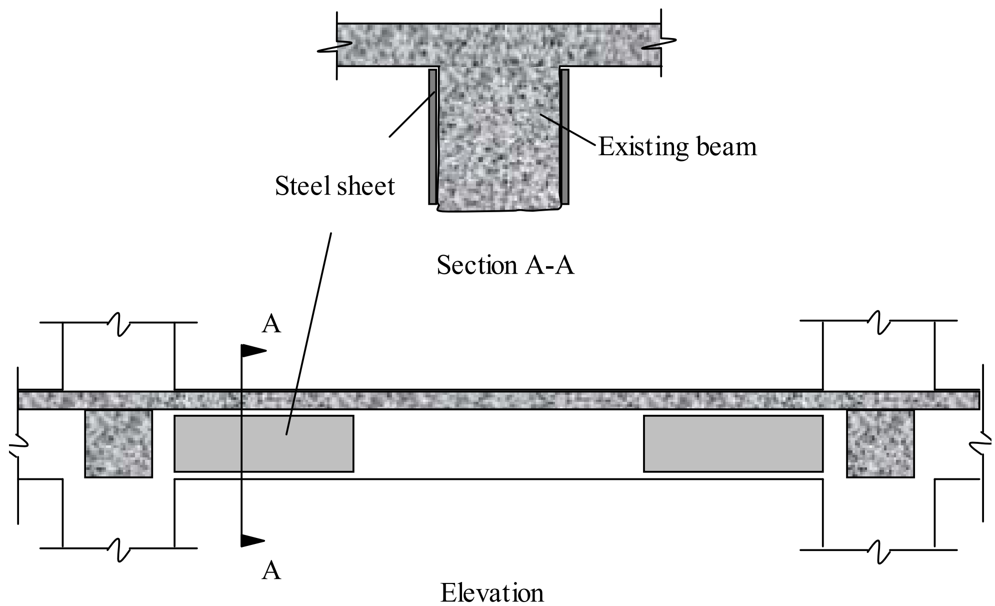


Figure 0.65 Use of steel sheets in beams

Fibre reinforced polymer (FRP) sheets are used in a similar manner to steel plates or sheets. The shear strength of a column can be enhanced by wrapping FRP sheets around it. The wrapping also enhances the behaviour under flexure due to the confinement of the concrete. The confinement refers to the enclosing of concrete which has a beneficial effect in terms of increase in compressive strength and ductility. For a beam, the sheets cannot be wrapped all around if there is a slab. Nevertheless, the shear strength can be increased by bonding sheets at the sides.

0.10 SUMMARY

Some of the key points identified in this write-up are summarised below:

- Earthquakes do not kill; unsafe buildings do.
- Earthquakes are natural phenomena. But when and where exactly the next earthquake will occur cannot be predicted by modern science.
- The Indian sub-continent has been divided into four seismic zones (zones II, III, IV and V), zone V being the most hazardous.
- If a building has been properly designed and constructed in accordance to the prevailing codes of practice, it is expected to survive the expected design earthquake.
- Most buildings (including the so-called "engineered" buildings) in India, however, are vulnerable, as revealed by the devastating earthquakes of the recent past.
- Get your building assessed for its seismic vulnerability.
- If the building is found unsafe, get it suitably retrofitted. Prevention is better than cure.
- In the case of an old building, the retrofit should be carried out so that the building has at least 70 percent of the seismic resistance capacity required of a new building as per the prevailing design code.
- If you are planning to invest in a new building, make sure that the builder provides the required seismic resistance. Use the information in this introductory chapter to ask pointed questions. In the case of multi-storeyed buildings, get the design proof-checked by a competent structural engineer to ensure that the design is code compliant.
- Ensure that the building architecture is simple in plan and elevation, with good seismic resistance features, especially if the building is located in a high seismic zone. Safety is more important than fancy looks. Avoid overhanging projections and any kind of irregularity in plan or elevation. Ensure that the staircases are well framed and will not collapse during the earthquake.
- Heavier buildings attract larger seismic forces. Low-rise buildings with light-weight materials are best suited in high seismic zone areas.
- Buildings must have adequate lateral stiffness, strength and integrity to remain elastic and undamaged, with low drift, during nominal ("serviceability") earthquakes.
- Under more severe and rarer earthquake levels, some damage is to be expected in ordinary buildings (other than lifeline buildings); otherwise the building would not be financially viable. However, the building must have sufficient ductility to withstand even the severe earthquake, without collapsing. It is possible to ensure this reasonably through proper design, detailing and construction.
- In framed buildings, foundations must be stronger than columns, and columns stronger than beams. Local distress is preferred to global collapse.

- Masonry walls that are long and thick act like shear walls, with good stiffness and strength, to resist lateral seismic forces that act along their length. However, they are weak against loads that act in a direction perpendicular to their length. To prevent possible collapse, adequate cross walls need to be provided, with proper bonding at the junctions. This can also be done as part of a seismic retrofit scheme.

- Masonry walls in load-bearing construction should be provided with reinforced concrete bands at plinth, sill, lintel and roof levels. It is also desirable to provide vertical steel bars at the edges of wall piers, ensuring that these are well anchored to the foundation and roof bands. Provision of vertical and horizontal reinforcement can also be part of a seismic retrofit scheme.

- The floor and roof slabs in a load-bearing construction need to be integrally connected to the supporting walls for effective transfer of forces. This can also be taken up as part of seismic retrofit.

- In reinforced concrete (RC) framed buildings, columns should have adequate cross-sectional size and reinforcement (both vertical bars and closely spaced lateral ties) for adequate strength (in shear and bending) and ductility. The common practice of providing only 230 mm thick columns is unsuitable for providing seismic resistance, especially in shear. Weak columns in existing buildings can be strengthened by various ways, such as jacketing (using RC, steel or FRP).

- The ground floor columns in buildings with open ground storey (for car parking) are particularly vulnerable under an earthquake, unless they have been properly designed for the additional stresses. There are many options of seismic retrofit, both global (providing walls and braces) and local (column jacketing) possible.

- RC beams also should have adequate reinforcement bars at top and bottom, which are well anchored at the beam-column joints. Closely spaced stirrups are also required, with proper hooks. Various options of seismic retrofit involving jacketing with RC, steel and FRP are possible.

- RC shear walls, often provided around lift cores, should be well integrated to the rest of the building; otherwise proper connections should be introduced through seismic retrofit.

This introductory chapter will have served its purpose if it has sensitised the reader, the common man, on the vital importance of making his buildings safe against earthquakes. More detailed technical information is provided in the *Handbook of Seismic Retrofit of Buildings.*

0.11 REFERENCES

3. IS 1893: 2002, "Criteria for Earthquake Resistant Design of Structures", Bureau of Indian Standards, New Delhi.
4. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings", Bureau of Indian Standards, New Delhi.
5. IS 13827: 1993, "Improving Earthquake Resistance of Earthen Buildings", Bureau of Indian Standards, New Delhi.
6. IS 13828: 1993, "Improving Earthquake Resistance of Low-Strength Masonary Buildings", Bureau of Indian Standards, New Delhi.
7. IS 13920: 1993, "Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces", Bureau of Indian Standards, New Delhi.
8. IS 13935 (Draft version, 2005), Seismic Evaluation, Repair and Strengthening of Masonry Buildings – Guidelines, Bureau of Indian Standards.
9. "An Earthquake Preparedness Guide", National Disaster Management Division, Ministry of Home Affairs, Government of India.
10. "Guidelines for Earthquake Resistant Buildings", (2002), Indian Institute of Technology Roorkee. Project sponsored by The Tata Iron and Steel Company Ltd.
11. "Guidelines for Earthquake Resistant Non-engineered Construction", (2004), Published by the International Association for Earthquake Engineering and National Information Centre of Earthquake Engineering.
12. "IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings", (2005), Indian Institute of Technology Kanpur. Project sponsored by the Gujarat State Disaster Mitigation Authority.
13. "Manual for Seismic Evaluation and Retrofit of Multi-storeyed RC Buildings", (2005), Indian Institute of Technology Madras and Structural Engineering Research Centre, Chennai. Project sponsored by Department of Science and Technology, Government of India.
14. "Technologies for Retrofitting of Existing Buildings and Structures to make them Earthquake Resistant", (2006), Indian Institute of Technology Roorkee. Project sponsored by Technology Information, Forecasting and Assessment Council.
15. Baker, L., "Earthquake", "Chamoli Earthquake Handbook", Published by Centre of Science and Technology For Rural Development. Project sponsored by Department of Science and Technology, Government of India.

16. Basu, P. C., (2002), “Seismic Upgradation of Buildings: An Overview”, The Indian Concrete Journal, The Associated Cement Companies Ltd., August, pp. 461-475.
17. Jain, S. K., (2001), “Seismic Strengthening of Existing Reinforced Concrete Buildings”, Repair and Rehabilitation, Compilation from The Indian Concrete Journal, The Associated Cement Companies Ltd., pp. 45 – 51.
18. Murty, C. V. R., (2005), “Earthquake Tips”, Indian Institute of Technology Kanpur. Project sponsored by Building Material and Technology Promotion Council.
19. Seth, A., (2002), “Seismic Retrofitting by Conventional Methods”, The Indian Concrete Journal, The Associated Cement Companies Ltd., August, pp. 489-495.

1

INTRODUCTION AND TERMINOLOGY

1.1 INTRODUCTION

Damages caused by recent earthquakes have exposed the vulnerability of buildings in India. Many of the non-engineered and semi-engineered constructions lack the basic features required for seismic resistance. Many of the so-called 'engineered' constructions, such as multi-storeyed apartments, are not adequately designed, detailed and constructed to provide the desired resistance against seismic forces. This may be attributed largely to a lack of awareness of seismic resistant design and code requirements. In recent years, particularly after the devastating Gujarat earthquake in 2001, there has been a concerted effort nation-wide to provide for increased awareness, in education and practice, with regard to seismic resistant design of buildings.

There is now a greater awareness and insistence on adherence to design code requirements, with regard to new buildings, especially those constructed by major organisations in the public and private sectors. It is hoped that this healthy practice becomes mandatory and adopted by all builders. Mechanisms need to be evolved by local approving bodies (corporations, municipalities, development authorities) to ensure that the buildings conform to the *National Building Code*. In particular, the structural design has to be proof-checked by a competent third party for code compliance. Ordinary people, investing their life savings in buildings such as apartment complexes, should also insist on this, in their own interest. All this is possible with buildings to be built in the future.

But, what is to be done about *existing* buildings? Many of these will be found to lack compliance with the current codes of practice, especially in terms of earthquake resistance. This is partly attributable to the increased seismic demand and up-gradation of some seismic zones in the country, as reflected in the recently revised code of practice for seismic analysis (IS 1893 Part 1: 2002). Even 'engineered' buildings built in the past are likely to lack the seismic strength and detailing requirements of the current design codes, such as IS 1893: 2002, IS 4326: 1993, IS 800 (Draft) and IS 13920: 1993, because they were built prior to the implementation of these codes.

Thus, many of the existing buildings, whether 'engineered' or 'non-engineered', old or recent, may be vulnerable. The degree of seismic vulnerability (risk of failure) can be ascertained only after a proper structural evaluation is undertaken. Based on this assessment, proposals can be worked out to *retrofit* the vulnerable buildings. The retrofit is required in order to avert potential disaster in the event of an earthquake. Seismic retrofit can be done in various ways and to various levels, depending on several factors. The aim, generally, is to ensure that building collapse does not occur in the event of the design earthquake, and not necessarily to achieve complete compliance with the prevailing codes. Of course, cost of retrofit is a major consideration in arriving at a decision.

This Handbook is an effort to compile the available information on seismic retrofit of buildings. The Handbook begins with a primer on seismic retrofit of buildings (Chapter 0), written in a simple style, and addressed to the lay reader, giving a broad overview of fundamental aspects. The subsequent chapters deal in detail with issues related to seismic evaluation, condition assessment and retrofit of a wide range of buildings, such as non-engineered buildings, masonry buildings, historical and heritage structure, reinforced concrete buildings and steel buildings. The various steps of seismic retrofit related with the engineering of the building are discussed.

The coverage of the Handbook includes structural, material, geotechnical and quality control aspects. An overview of each chapter is provided for quick perusal. Recent advances in the material and technology for seismic retrofit are included. A few case studies are provided as guidelines for undertaking seismic retrofit projects. The Handbook is intended for practising engineers in public sector and private organizations, educators, decision making officials, professionals in non-governmental organisations and students. Emphasis is laid on the fundamental concepts and principles related to seismic retrofit.

Seismic retrofit of an existing building will generally involve intervention that affects the use and functionality of the building during the retrofit operations. The process of convincing the tenants or users on the need for seismic retrofit, decision making, as well as financing the consultancy and construction are difficult as compared to new construction. Hence, the steps, goals and objectives of seismic retrofit have to clear before a project is undertaken. This chapter provides an overview of these aspects.

1.2 NEED FOR SEISMIC EVALUATION OF EXISTING BUILDINGS

On a priority basis, seismic evaluation and retrofit are undertaken for the life-line buildings, such as hospitals, police stations, fire stations, telephone exchanges, broad casting stations, television stations, railway stations, bus stations, airports (including control towers), major administrative buildings, relief co-ordination centres and other buildings for emergency operations. The next set of important buildings includes schools, educational institutions, places of worship, stadia, auditoria, shopping complexes and any other place of mass congregation. High rise multi-storeyed buildings, major industrial and commercial buildings, historical and heritage buildings are also among the important buildings.

Seismic vulnerability of an existing building is indicated under the following situations.

1. The building may not have been designed and detailed to resist seismic forces.
2. The building may have been designed for seismic forces, but before the publication of the current seismic codes. The lateral strength of the building does not satisfy the seismic forces as per the revised seismic zones or the increased design base shear. The detailing does not satisfy the requirements of the current codes to ensure ductility and integral action of the components.
3. The construction is apparently of poor quality.
4. The condition of the building has visibly deteriorated with time.
5. There have been additions or modifications or change of use of the building, which has increased the vulnerability. For example, additional storeys have been built.
6. The soil has a high liquefaction potential.

Especially under such situations, and specifically, in the case of life-line and other important buildings, there is a clear need for a structural assessment of the existing buildings for their adequacy to withstand the design seismic loads.

In this context, the changes in the seismic design code IS 1893 are noteworthy. There has been a significant increase in the magnitude of design base shear (which is a measure of the total lateral load on a building). Figure 1.1 illustrates this by providing a comparison of typical values of seismic coefficient (ratio of base shear to seismic weight) for the five different seismic zones in the earlier 1984 version and the revised 2002 version of IS 1893. In this illustration, the building considered is an ordinary framed building (with about 5 storeys) located in 'medium' soil. It is apparent that there is a substantial increase in the seismic coefficient for each zone. The minimum seismic zone in the country is now Zone II (as against Zone I earlier). Many areas have been upgraded to higher seismic zones (Zone II to Zone III, for example).

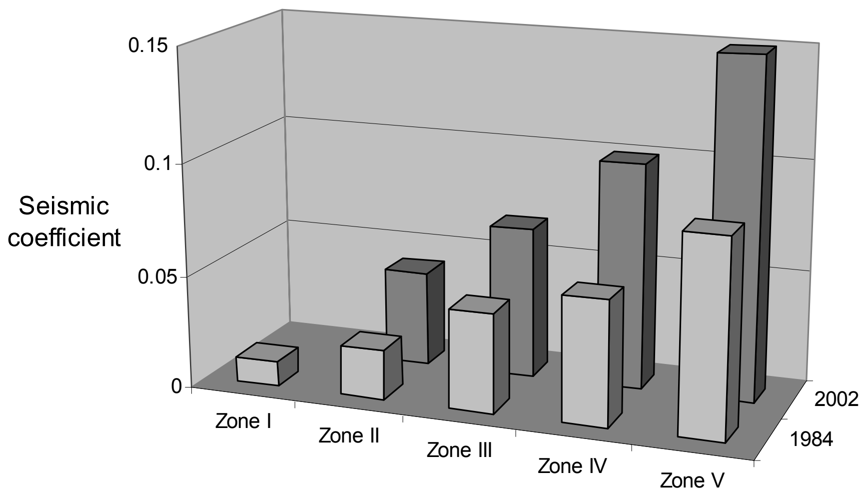


Figure 1.1 Variations of seismic coefficient in different zones

Before the methodology of seismic retrofit is introduced, it is necessary to understand the special attributes of seismic design of a building, as compared to the usual design for gravity loads.

1.3 ATTRIBUTES OF SEISMIC DESIGN

The design of buildings for seismic loads is special, when compared with the design for gravity loads (dead loads and live loads). Gravity loads are relatively constant, in terms of their magnitude and direction (always vertical) and are treated as 'static' loads. In contrast, seismic loads are predominantly lateral, reversible (the forces change direction), dynamic (the forces vary with time) and of very short duration. The seismic loads are more uncertain than the conventional gravity loads in terms of magnitude, variation with time and instance of occurrence. The variations of the forces with time affect the resistance of the building. The maximum magnitudes of the internal forces and their locations in the structural members are different from those due to gravity loads. Moreover, some features of the building essential to resist seismic loads are not considered in the conventional design for gravity loads. These special features are referred to as the attributes for seismic design.

IS 1893: 2002 lists the following attributes for seismic design of a building.

1. Simple and regular configuration
2. Lateral strength
3. Lateral stiffness
4. Ductility
5. Stability

In addition, integral action of the various members is essential.

1.3.1 Simple and Regular Configuration

In a building with a simple and regular configuration, the internal forces are more uniformly generated and the seismic loads are transmitted to the ground efficiently. The behaviour of such a building under seismic loads can be studied by the conventional methods of analysis. On the other hand, some building configurations that are satisfactory to resist gravity loads do not perform well under seismic loads. These configurations are termed as *irregular* configurations. For a building with an irregular configuration, there are concentrations of internal forces in some members which lead to damage. The conventional methods of analysis may not be suitable, depending upon the type and extent of irregularity. IS 1893: 2002 and IS 4326: 1993 list the different types of irregular configuration. These aspects are further explained in Chapter 2.

1.3.2 Lateral Strength

The 'lateral strength' is the capacity of a building to resist horizontal loads. A building should have adequate lateral strength to resist seismic loads. This is the primary requirement in a seismic design. A building may be adequate to resist static gravity loads, but may not be adequate to resist dynamic seismic loads. This leads to collapse of the building and injury to or death of the occupants. Of primary concern is the total lateral force resisting capability (lateral strength) of the building, which behaves essentially like a vertical cantilever subject to lateral loads acting on the masses at different levels. This lateral strength must not fall short of the total seismic load demand at any level in the building. The demand is largest at the base of the building, where there must be adequately strong lateral load resisting elements (walls, columns), and below, at the foundations also, the footings or piles must have the required strength to resist the horizontal base shear. Even in non-engineered buildings, these essential requirements of lateral load resisting elements with adequate strength have to be satisfied.

1.3.3 Lateral Stiffness

The second important aspect of design of a building is its lateral stiffness. Under gravity loads, the vertical deflection of a floor should not be excessive. Similarly, for moderate values of seismic loads, the lateral deflection of a building should not be excessive. The resistance of a building to lateral deflection is termed as *lateral stiffness*. If the lateral stiffness is not adequate, there will be a relatively large deflection in the event of the moderate earthquake, as a result of which the members may not be able to resist the simultaneous action of gravity and seismic loads satisfactorily. For a building with non-structural members like infill walls, facades, cladding, utility ducts, etc., there is likely to be heavy damage and injury or death, even if the lateral strength is adequate. The overall stiffness of a building is generally measured by the ratio of the relative lateral displacement of the roof (with respect to the base) to the height of the building. The stiffness of each storey is measured by the ratio of the inter-storey drift (relative lateral displacement of the upper floor with respect to the lower floor) to the height of the storey.

1.3.4 Ductility

Designing a building to withstand a severe earthquake such that there will be no significant damage or residual deformation, although desirable, will turn out to be an extremely costly proposition. As the probability of occurrence of such an extreme earthquake during the design life of a building is generally low, and resources are limited, seismic design codes world-wide allow for damage and residual deformations in buildings if such an event were to occur. This has the advantage of substantially reducing the design seismic loads, as compared to elastic design, by allowing the input energy to be dissipated through cracking and yielding (which are irreversible phenomena), rather than through elastic strain energy. The internal forces built up in the various members result in inelastic deformations, and the design and detailing must facilitate this, so that the members 'hang on' and remain intact (although damaged) during the extreme earthquake. The ability to sustain these internal forces through large deformations, without breaking, is termed as *ductility*.

In a properly designed building, if the members have this property of ductility, then the overall behaviour of the building will also be ductile. Otherwise, in the absence of ductility, the behaviour will be brittle, and the structure is likely to collapse without undergoing significant deformations. Non-engineered buildings typically exhibit such brittle behaviour, whereas well-designed framed buildings exhibit ductile behaviour. This is illustrated in Figure 1.2, which shows typical (static) base shear versus roof displacement curves for these two cases. The area under each curve is a measure of the total energy absorbed. Clearly, the property of ductility greatly enhances the ability of the structure to withstand larger input energy. In the absence of ductility, if the building is to survive, the input energy has to be absorbed as elastic strain energy, requiring the initial straight line portion of the graphs in Figure 1.2 to extend upwards, implying the need for a much higher design seismic load.

1.3.5 Stability

An earthquake induces vibrations in a building in all directions. The building must retain stability under these conditions. The action of lateral seismic loads on a building is to push the building horizontally and cause it to bend. In some cases, the building may also twist about its base. There must be adequate fixity at the base (foundation) to prevent translations and rotations, causing instability. In particular, there should be sufficient margin of safety against the possibilities of instability due to overturning or sliding.

1.3.6 Integral Action

The various structural components of the building should be properly inter-connected in order to generate 'unity' and 'integral action' or 'integrity' in the behaviour. For example, the floors and roof in a load-bearing masonry building need to be properly connected to the supporting masonry walls or pillars so that they act integrally and facilitate transmission of forces to the foundations. Similarly, the cross-walls in a building need to be inter-connected through proper joints at the corners and with horizontal bands or ties to ensure combined action. In a framed building, the slabs need to be properly connected to the frames to mobilise the lateral load resisting capacity of the frames. The beams and columns are to be properly connected at the joints to mobilise the frame action.

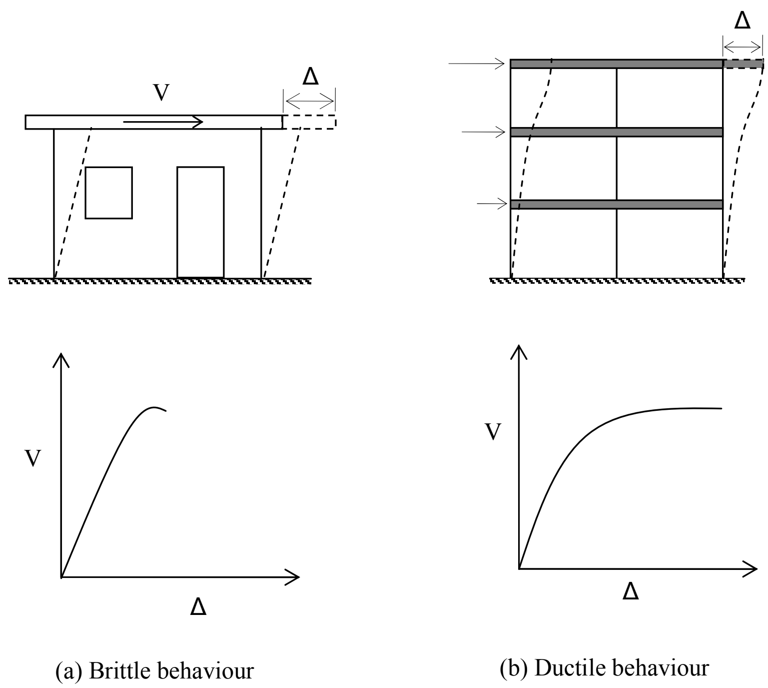


Figure 1.2 Typical base shear (V) versus roof displacement (Δ) curves

1.4 RETROFIT VERSUS REPAIR AND REHABILITATION

It is important to distinguish, at the very outset, between the terms *retrofit* and its counterparts, *repair* and *rehabilitation* of a building. All three terms refer to modifications carried out on a building, but in different contexts. While 'repair' is a generic term that is loosely used to describe such interventions, strictly, it is used to refer to minor interventions that are non-structural in nature. On the other hand, both 'retrofit' and 'rehabilitation' refer to structural interventions aimed at strengthening the building. The difference between these two terms is simply this: retrofit is undertaken before the occurrence of the earthquake (as a preventive measure) in an undamaged building, while rehabilitation is done in a building that is already damaged (either due to a disaster such as earthquake or due to some serious deterioration such as excessive corrosion). The implications of retrofit and rehabilitation, in terms of the typical base shear versus roof displacement curves for a building, are as depicted in Figure 1.3. For adequate strengthening of existing buildings against earthquakes, it is seismic retrofit that is required.

Retrofit and rehabilitation may involve addition of new structural elements or change in the structural system. But repair is restricted to the as-built system.

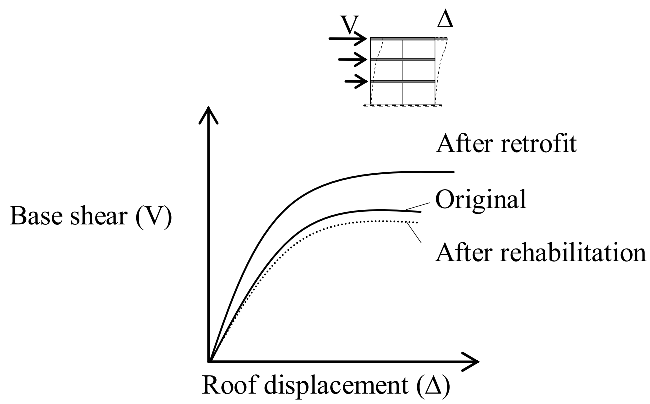


Figure 1.3 Expected base shear versus roof displacement curves

1.4.1 Repair

Repair refers to the actions that improve the functionality of components in a building that have been rendered defective, deteriorated or damaged due to some cause. The purpose of repair is to rectify the observed defects and bring the building to its original architectural shape and its intended purpose. Repairs are generally non-structural in nature, and if carried out on structural members (such as patching of structural cracks caused by tension), are unlikely to enhance structural strength significantly. In fact, a repaired building may be deceptive in that it will appear good and give the occupants a false sense of safety; it does not guarantee structural safety, particularly against earthquakes.

Repair works typically involve actions such as patching cracks and falling plaster, fixing doors, windows, broken glass panes, setting right wiring and other electrical installations, fixing plumbing services (water pipes and sewage lines) and other services such as gas lines, refurbishing non-structural walls and partition walls, re-affixing roof tiles and false ceilings, repairs to flooring, drainage, architectural finishes, tiles, etc.

1.4.2 Rehabilitation

Rehabilitation refers to structural interventions that improve the strength of the components in a building that are either deteriorated or damaged. The rehabilitation is intended to regain the original strength of these structural members. For example, in the event of a fire in a building, rehabilitation works are undertaken to replace or strengthen the damaged structural members. Such intervention cannot provide more than the original strength of the building, and is appropriate if this original strength provides an adequate level of safety. The term *restoration* is sometimes interchangeably used with rehabilitation. However, in this Handbook, for clarity, the term 'restoration' will be used only for historical structures.

Some of the common rehabilitation actions include removal and rebuilding of deteriorated walls, grouting of cracks in the structural members, guniting damaged concrete surfaces, treatment against corrosion, providing additional steel reinforcement to restore tensile strength affected by corrosion, underpinning of foundations, etc. Sometimes, the scope of the term rehabilitation includes repair of services and architectural finishes.

1.4.3 Retrofit

Retrofit specifically aims to enhance the structural capacities (strength, stiffness, ductility, stability and integrity) of a building that is found to be deficient or vulnerable. In the specific context of enhancing the resistance of a vulnerable building to earthquakes, the term *seismic retrofit* is used. Sometimes, the terms 'seismic rehabilitation', 'seismic upgradation' and 'seismic strengthening' are used in lieu of 'seismic retrofit'. The building need not be deteriorated or damaged. The retrofit is intended to mitigate the effect of a future earthquake.

Seismic retrofit can effectively raise the performance of a building against earthquakes to a desired level, and even to satisfy the requirements of an upgraded seismic design code. To what extent the retrofit has to be carried out is an important decision that also has cost implications. In the case of old buildings, it is generally not necessary to raise the structural capacities (attributes for seismic design) to the level expected of a new building as per the current code of practice. Some recommendations on this are given in the following sections.

1.5 GOALS OF SEISMIC RETROFIT

The goals of seismic retrofit refer to the actions to be taken with reference to the attributes for seismic design, in qualitative terms. They can be summarised as follows (IS 13935: 1993).

1. To increase the lateral strength and stiffness of the building.

2. To increase the ductility in the behaviour of the building. This aims to avoid the brittle modes of failure.
3. To increase the integral action and continuity of the members in a building.
4. To eliminate or reduce the effects of irregularities.
5. To enhance redundancy in the lateral load resisting system. This aims to eliminate the possibility of progressive collapse.
6. To ensure adequate stability against overturning and sliding.

1.6 OBJECTIVES OF SEISMIC RETROFIT

The objectives of seismic retrofit are quantitative expressions to achieve the goals of retrofit. Of course for a non-engineered building, the objective may not be quantifiable. The implicit objective is to provide adequate lateral strength by strategies that have been tested or proved to be effective in past earthquakes. The retrofitted building should not collapse during a severe earthquake.

For an ‘engineered building’, the objectives are based on measurement of relevant quantities. The objectives need to be defined before designing for retrofit. Traditionally, the members of a building are designed based on the internal forces (demand) calculated from a linear elastic analysis of the building subjected to the design base shear. A member is designed to have a resistance (capacity) equal to or greater than the demand. This satisfies the requirement of strength against both vertical and lateral loads. The building should not collapse during a severe earthquake. To satisfy the requirement of stiffness, the drift in a storey is limited to a certain value. To achieve ductility and integral action, the detailing of reinforcing bars in concrete buildings and the connections in steel buildings are to be done properly. The objectives of retrofit, based on the traditional seismic design method, are referred to as the *conventional objectives*.

For seismic retrofit of buildings, a new approach based on quantifying the ‘performance’ of a building is gaining popularity. The decision to retrofit and the selection of retrofit strategies for an existing building are open-ended tasks, as compared to seismic design of a new building. The performance based approach aids the decision making and selection of retrofit strategies.

1.6.1 Conventional Objectives

Demand-to-capacity ratio for a member

Based on the traditional design method, a member of a building is considered to be adequate if the design capacity of the member for an internal force (such as bending moment or shear force) is not less than the demand (including appropriate load and material safety factors). The internal forces are first calculated for the gravity loads by structural analysis. Next, the internal forces are calculated for lateral loads, such as seismic loads and wind loads. The values of an internal force in a member due to gravity loads and lateral loads are combined, assuming linear elastic behaviour, based on certain load combinations. The design value of the internal force (demand) is the highest numerical value obtained after the combinations. The capacity is calculated from an analysis of the member properties. After the calculation of the demand and capacity of a member for an internal force, the demand-to-capacity ratio (DCR) is computed as follows:

$$\mathrm{DCR} = \frac{\text{Demand for an internal force}}{\text{Capacity for an internal force}} \qquad \textbf{(1.1)}$$

If this ratio exceeds 1.0, it implies that the member strength is deficient, with reference to the code requirement for a new building. High values of DCR indicate high risk of failure. The checking of DCR for each member and each mode of failure provides an assessment of the overall strength of the building and the risk of failure in the event of the design earthquake. In a limit states method of analysis, the checking of DCR satisfies the limit state of collapse under factored loads. When values of DCR less than unity are encountered, seismic retrofit can be employed to raise the DCR values suitably in order to reduce the risk of failure.

It may be noted that the DCR can also be computed without introducing the material safety factors for member strength, to arrive at a more realistic assessment of the actual strength of the building. The values of DCR thus computed will be higher than the corresponding values computed with partial safety factors. The decision regarding seismic retrofit can also be based on this approach.

Drift

The storey drift ratio is the ratio of the inter-storey drift, Δ_s, (relative lateral displacement of the upper floor of a storey with respect to the lower floor) to the height of the storey, h_s. As mentioned earlier, in addition to the lateral strength, the lateral stiffness of the building also should be adequate. This check is aimed at satisfying the limit state of serviceability under unfactored lateral loads (with load factor equal to 1.0). The drifts of the storeys can be calculated from the structural analysis of the building. The recommended limit of the storey drift ratio is 0.004 (Clause 7.11.1, IS 1893: 2002).

$$\frac{\Delta_s}{h_s} \leq 0.004 \qquad \textbf{(1.2)}$$

If this limit is not satisfied in an existing building, then there is a clear risk of violating the serviceability limit state. This risk can be reduced by suitably providing seismic retrofit measures to enhance the lateral stiffness and thereby reduce the storey drift ratio.

The calculation of lateral displacement is also used to check the possibility of *pounding*. When the gaps between two adjacent buildings or two adjacent blocks of the same building are not sufficient, the buildings may come in contact during vibration due to the ground motion. This is known as pounding, and it leads to transfer of force from one building to the other. In a conventional analysis, the pounding forces are not considered. Hence, it is recommended to have sufficient gap between two adjacent buildings or two adjacent blocks of the same building (Clause 7.11.3, IS 1893: 2002). This gap is called *seismic joint* and is larger than the traditional expansion joint. The required gap increases with height of the building.

<u>Role of Linear Elastic Analysis</u>

The calculations for conventional objectives of seismic retrofit can be performed by linear elastic methods of analysis. The equivalent static analysis based on application of static lateral loads at the floor levels, is the simplest to undertake. Since the equivalent static analysis deals with application of static forces, it is called a *static* method. On the contrary, the response spectrum method is based on dynamic characteristics, such as mode shapes. It is called a *dynamic* method and is recommended for certain types of buildings (Section 7.8, IS 1893: 2002). In both the equivalent static analysis and response spectrum method, the load and deformation of a member are considered to be proportional. Hence, both these methods are linear elastic methods of analysis. These methods have the following deficiencies.

1. The results from the linear elastic methods do not reflect the inelastic deformation capability or ductility of the members or of the building.
2. The sequence of yielding of sections and subsequent redistribution of loads in the building are not available from either of these methods.

Since the design for seismic forces is based on the accepted criteria that the members can undergo nonlinear inelastic deformation with increasing internal force, a nonlinear inelastic method of analysis is preferred. Moreover, the design for all types of buildings cannot target the same level of seismic loads. To address these requirements, a performance based approach is recommended whenever the necessary tools for the analysis are available.

1.6.2 Performance Based Approach

The traditional approach to seismic design of a building is a 'force based' analysis and design. The *performance based* approach is an alternative to that approach, which is based on quantifying the inelastic deformations of the members and the building as a whole, under the seismic loads. The deformations or strains are considered to be better measures than stresses or forces to assess damage. To quantify inelastic deformations, a performance based approach requires a nonlinear lateral load versus deformation analysis. Pushover analysis and nonlinear time history analysis are the static and dynamic methods of nonlinear analysis, respectively. The performance based approach gives the designer more choices of 'performance' of the building, as compared to the demand-to-capacity ratio and drift as mentioned under conventional objectives.

The performance of a building is measured by the state of damage under a certain level of earthquake. The state of damage is expressed as a 'performance level'. For the building as a whole, the performance level is quantified by the inelastic drift of the roof. For a member, the performance level is quantified by its deformation. The objective of seismic retrofit constitutes of targeting a performance level of the building and the members under a certain level of earthquake.

<u>Building Performance Levels</u>

The performance levels are discrete damage states identified from a continuous spectrum of possible damage states. A building performance level is a combination of the performance levels of the structure and the non-structural components. The structural performance levels are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). The three levels are based on the behaviour of the lateral load resisting systems under increasing base shear. Figure 1.4 shows the three levels in a base shear versus roof displacement curve for a building with adequate ductility. Detailed attributes of the performance levels are given in FEMA 356.

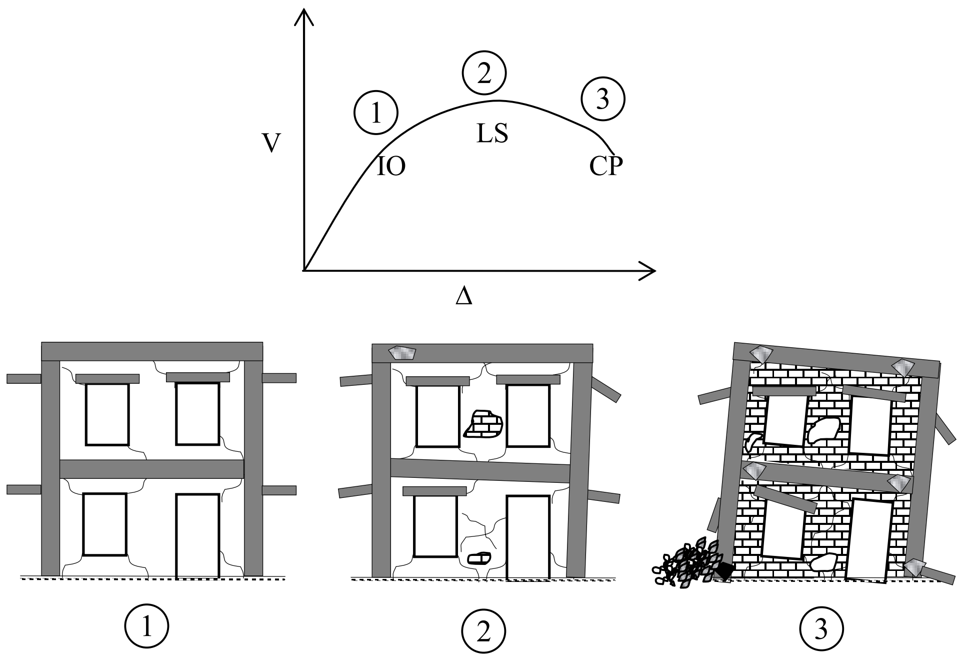


Figure 1.4 Performance levels in a base shear (V) versus roof displacement (Δ) curve

Similar to the structural performance levels, the member performance levels are discrete damage states in the load versus deformation behaviour of each member. The member performance levels are also Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). For the beams and columns of a lateral load resisting frame, the following curves relating the loads and deformations are necessary.

1. Moment versus rotation
2. Shear force versus shear deformation.

For a column, the moment versus rotation curve is calculated in presence of the axial load. For each member, the respective curve is assigned at each end where the deformation is largest.

For steel members, the moment versus rotation curves are available from the section properties. For reinforced concrete (RC) members, the moment versus rotation curves are calculated based on conventional analysis of sections (Park and Paulay, 1975). The shear force versus shear deformation behaviour is difficult to analyse. Since in beams and columns, the shear deformation is negligible as compared to flexural deformation, a complicated analysis is not warranted. A linear elastic behaviour up to the shear capacity and stiffness corresponding to the shear modulus of the material, can be assumed.

The performance levels are selected based on the limits on the deformation. Figure 1.5a shows a typical moment versus rotation curve for an RC beam. In Figure 1.5b, the behaviour is idealized to a piecewise linear curve and it is shown along with the performance levels. The discontinuity at cracking is ignored and the moment capacity is limited to the moment at yielding of reinforcing bars (M_y). In absence of reliable modelling of the post-peak (beyond ultimate) behaviour, a non-zero residual moment capacity of 20% of M_y is considered for numerical stability Typical load versus deformation curves for the members and the corresponding performance levels are provided in ATC-40.

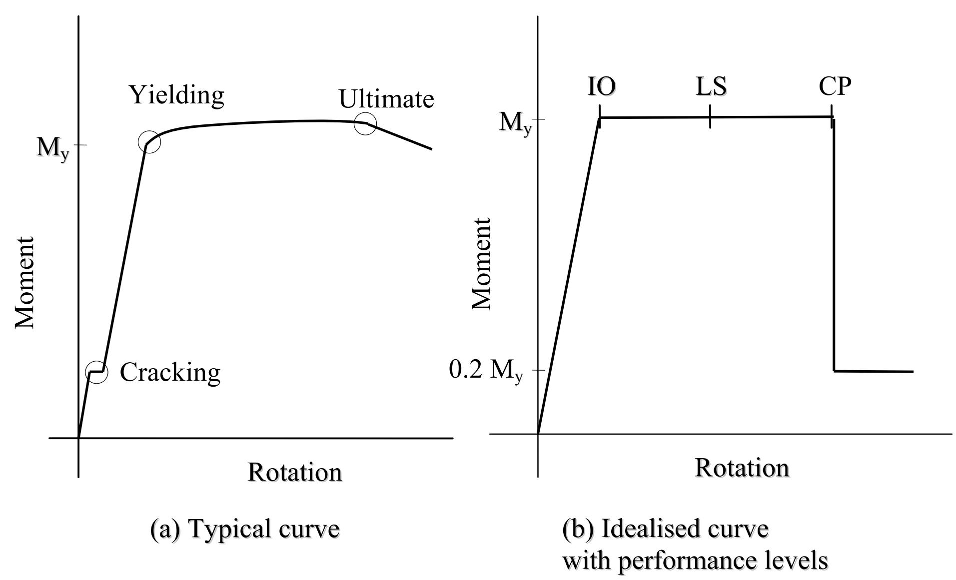


Figure 1.5 Typical moment versus rotation curves of a beam

Seismic Hazard Levels

In a performance based analysis, *seismic hazard level* (or earthquake hazard level or simply, earthquake level) refers to the level of ground motion. The earthquake level can be described by two types of methods. First, in the deterministic method, the engineering characteristics of the shaking at a site due to an earthquake are represented through zone factor and response spectra or ground motion time histories. The second type of method in describing the earthquake level is the probabilistic method. A probability of exceedance of an earthquake level in a specified period is used to define the earthquake levels. The *serviceability earthquake*, *design basis earthquake* (DBE) and the *maximum considered earthquake* (MCE) correspond to 50%, 10% and 2% probability of exceedance in 50 years, respectively (FEMA 356).

1.6.3 Performance Based Objectives

An objective of a performance based approach relates a target building performance level under a selected earthquake level. The selection of the two levels is based on recommended guidelines for the type of building, economic considerations and engineering judgment. For retrofit of ordinary buildings, a minimum performance of *Collapse Prevention* under MCE can be selected. For important buildings, a dual level performance objective that targets *Life Safety* under DBE and *Collapse Prevention* under MCE can be selected. The aim of such an objective is to have a low risk of life threatening injury during a moderate earthquake (as defined by DBE) and to check the collapse of the vertical load resisting system during a severe earthquake (as defined by MCE). To check Life Safety under DBE, performance of the non-structural components also needs to be investigated. For life-line buildings, enhanced performance can be targeted. A detailed discussion is provided in FEMA 356.

Role of Pushover Analysis

It was mentioned that a performance based approach requires a nonlinear lateral load versus deformation analysis. The pushover analysis is a static method of nonlinear analysis. The pushover analysis is an elegant method to observe the successive damage states of a building, both in the existing condition and under a proposed retrofit scheme. It addresses the deficiencies of an elastic analysis by the following features.

1. The analysis considers the inelastic deformation and ductility of the members.
2. The sequence of yielding of sections (formation of hinges) in members and subsequent redistribution of loads in the building are considered.

Of course there are deficiencies of push over analysis as compared to the more rigorous non-linear time history analysis. To perform a pushover analysis, reasonably accurate nonlinear load versus deformation curves for the members are required. The results from a pushover analysis will be erroneous if the load versus deformation curves are not properly calculated, or if the quality of construction is poor. Moreover, the pushover analysis gives only an envelop representation of the base shear versus roof displacement behaviour of a building. The actual performance of a building may significantly differ from the calculated performance, since the load versus deformation curves are not accurate and the earthquake levels used in the analysis are statistical estimates. A closer simulation of the actual behaviour under an earthquake requires a nonlinear time history analysis with appropriate hysteretic model for the load versus deformation behaviour of each member.

1.6.4 Guidelines for Level of Retrofit based on Conventional Objectives

Most engineers, unfamiliar with the performance based approach (which requires nonlinear analysis), will prefer to operate with the conventional objectives of satisfying strength and stiffness requirements. Decisions have to be made on whether or not to retrofit, and to what extent retrofit needs to be carried out when the conventional objectives are not satisfied. There are no explicit statements on this in the present codes of practice, and the decision is generally left to the judgement of the engineer.

It is well recognised that an old existing building cannot be expected to perform as well as a new one, especially if its original design pre-dates the requirements of the current seismic code. A guideline to reduce the design base shear is provided in the draft code "Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines". The reduction of the base shear is based on the remaining useable life of an existing building. The reduction factor (U) is given as follows.

$$U = \left(\frac{T_{rem}}{T_{des}}\right)^{0.5} \geq 0.7 \qquad \textbf{(1.3)}$$

Here,

T_{rem} = remaining useable life of the building (in years)

T_{des} = design life of the building (in years).

The above factor is expressed in a graphical form with respect to T_{rem} in Figure 1.6. It is to be noted that engineering judgement is required to determine T_{rem} and T_{des}. The calculation of base shear is explained in the chapter of Structural Analysis for Seismic Retrofit.

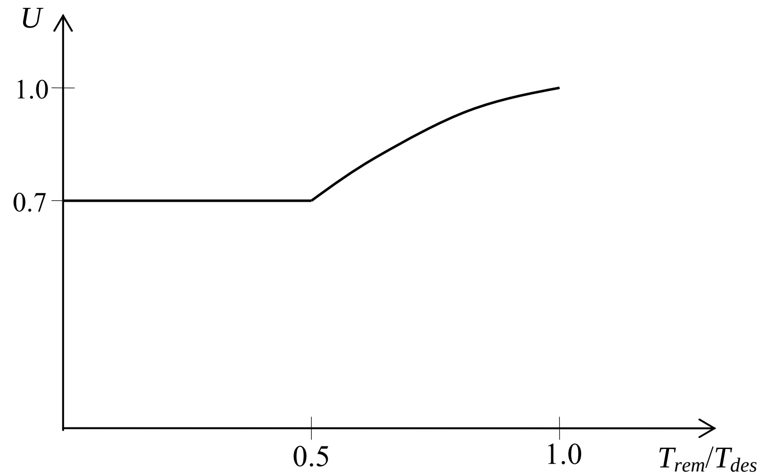


Figure 1.5 Useable life factor versus remaining useable life of a building

Of course reducing the base shear has an associated risk of failure. The demand-to-capacity ratio (DCR, Eqn. 1.1) is a simple measure of the deficiency at the member level, in relation to the expected capacity of a new member designed as per the current code. The DCR calculated based on the base shear as per the current code and the material safety factors can be used to determine the risk. If the value of DCR equal to unity is used as a reference point, then for every value of DCR greater than unity, there is an associated risk of failure that can be expressed as a factor, called *risk factor*. Figure 1.7 illustrates the relationship between DCR and risk factor (NZSEE Study, 2002). The risk factor increases almost exponentially with increase in DCR. For example, a DCR of 1.5 corresponds to a risk factor of 3.0. This implies that a structural member having demand 1.5 times the capacity (as per the current code requirement for new buildings) will have an associated seismic risk (probability of failure under a design earthquake) that is 3 times the accepted level of risk (set by the code for a new building). Similarly, a DCR of 3.0 is associated with a risk which is as high as 20 times the accepted standard. It is recommended that for seismic retrofit of ordinary buildings, the target DCR can exceed 1.0 but the risk factor should be limited to 3.0. In other words, the DCR should not be greater than 1.5. For retrofit of heritage or important buildings, the DCR should be limited to 1.0.

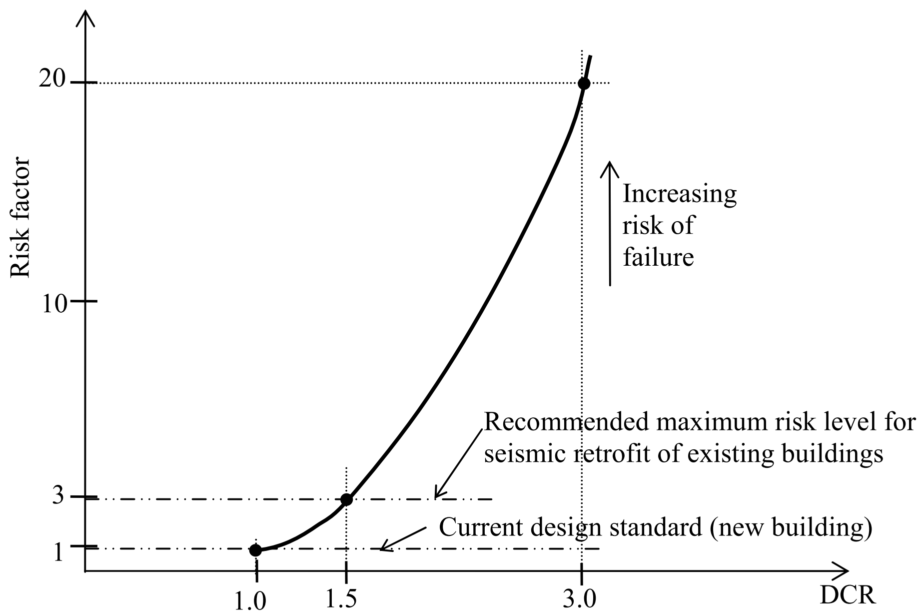


Figure 1.7 Relationship between DCR and risk factor

1.7 STEPS OF SEISMIC RETROFIT

A retrofit programme for a building refers to the complete process of retrofitting. For a systematic approach, it is necessary to be aware of the steps of a retrofit programme before undertaking the retrofit project. The implementation of each step requires a certain time schedule and finance. All the listed steps may not be applicable for all projects. Similarly, there may be detailed sub-divisions of one step for a particular project.

The suggested steps of a retrofit programme are explained with reference to the material in the Handbook. The steps are as follows (Basu, 2002, FEMA 356).

<u>i) Reviewing initial considerations</u>

The initial considerations relate to the type of structure, the seismic zone location of the structure, occupancy, economic considerations, historic status, local jurisdiction, social and administrative issues.

Given the requirement of vulnerability reduction for many buildings within a jurisdiction, the “Rapid Visual Screening” procedure is undertaken to identify the buildings which are expected to be more vulnerable under an earthquake. If the project targets one major building, the screening phase is skipped.

ii) Selection of the objective of retrofit

The objective of retrofit gives a quantitative target to achieve through retrofit. The objective can be drawn from the conventional objectives or performance based objectives, as per the guidelines given in Section 1.6.

iii) Obtaining information of the building

The construction documents, including the architectural and structural drawings, are to be collected from the archives. The collection of relevant data is explained in the chapter on Rapid Visual Screening, Data Collection and Preliminary Evaluation. For a proper evaluation, the actual condition of the building is to be assessed. The “Condition Assessment” describes the process of assessing the actual condition of the building in relation to its use.

iv) Seismic evaluation

The seismic evaluation identifies the deficiencies of a building. Broadly, the evaluation can be performed in the following phases.

a) Preliminary Evaluation

For an engineered building, the preliminary evaluation involves a set of initial calculations and identifies areas of potential weaknesses in the building.

b) Detailed Evaluation

The detailed evaluation refers to the structural analysis of the building. The method of analysis is to be finalised at this stage. The methods of structural analysis are briefly described in the chapter on Structural Analysis for Seismic Retrofit.

v) Decision to repair, retrofit or demolish

Based on the importance, target life, extent of deficiency of the building, the economic viability, the availability of the materials and technical resources, the expected life after retrofit, a decision is to be taken whether to repair, retrofit or demolish the building. The various stake-holders are responsible to take the decision. The action can be either one of the following.

a) The safety of the building is adequate. The building needs some repair and regular maintenance, ensuring adequate performance during a future earthquake.
b) The safety of the building is inadequate and hence, retrofit is necessary. The proposed retrofit scheme should be feasible and economically viable. The required retrofit is to be undertaken.
c) The safety of the building is grossly inadequate and the building is in imminent danger of collapse in the event of an earthquake. But the retrofit scheme is not economically viable or feasible. The building is to be declared unfit for use and demolished.

If the life-line and other important buildings are unsafe, either retrofitting or demolishing and rebuilding should be undertaken. For other types of building, intervention for preventing collapse during an earthquake should be undertaken. The options of intervention are discussed in the chapters to follow on different types of buildings.

vi) Selection and design of retrofit strategies

A 'retrofit strategy' or 'retrofit technique' refers to any option of increasing the strength, stiffness and/or ductility of the members or of the whole building. For a building, several retrofit strategies may be selected under a 'retrofit scheme'. The selection of retrofit strategies depends on the available technical expertise and inconvenience during the intervention. The retrofit strategies can be grouped under global and local strategies. A global retrofit strategy targets the seismic resistance of the building. A local retrofit strategy targets the seismic resistance of a member, without significantly affecting the overall resistance of the building.

The knowledge of the individual retrofit strategies is essential in the selection and design of the retrofit strategies. The design and the detailing should address the transfer of load and the compatibility of deformation among the existing members, modified members and the new members as per the assumptions in the analysis. Each retrofit strategy should consistently and reliably achieve the intended goals. The retrofit scheme should be cost effective. The retrofit strategies suitable for each type of building are covered in the subsequent chapters of the handbook.

vii) Verification of the retrofit scheme

In the case of non-engineered buildings, the proposed retrofit schemes should ideally be verified by experiments for their effectiveness. In the case of an engineered building, it should be ensured through structural analysis of the retrofitted building that the selected retrofit scheme satisfies the identified objective of retrofit. Alteration of the load path, redistribution of the member forces and the changes in the failure modes after retrofitting, need to be studied. The increase in strength at the cost of a ductile failure mode changing to a brittle mode, is not desirable. The retrofit scheme should be viable in terms of cost and execution. If the retrofit scheme is acceptable, then the construction documents are prepared. Else, the retrofit scheme has to be redesigned or the objective of retrofit has to be reselected.

viii) Construction

The effectiveness of the retrofit scheme greatly depends on the quality of construction. Hence, the construction as per the suggested details and specifications is imperative. The aspects of quality assurance and control are briefly covered in the chapter under the same title.

ix) Maintenance and monitoring

Although maintenance and monitoring of a retrofitted building are beyond the scope of a retrofit project, these two long-term activities should be undertaken. The maintenance of retrofitted buildings is necessary to achieve the performance during a future earthquake. Monitoring the performance of retrofitted buildings during an earthquake, wherever possible, is essential to detect any defect or deficiency in the retrofit strategies. This will lead to refinement of the design guidelines and specifications for future retrofit projects.

The steps of a retrofit programme are shown as a flow chart in Figure 1.8.

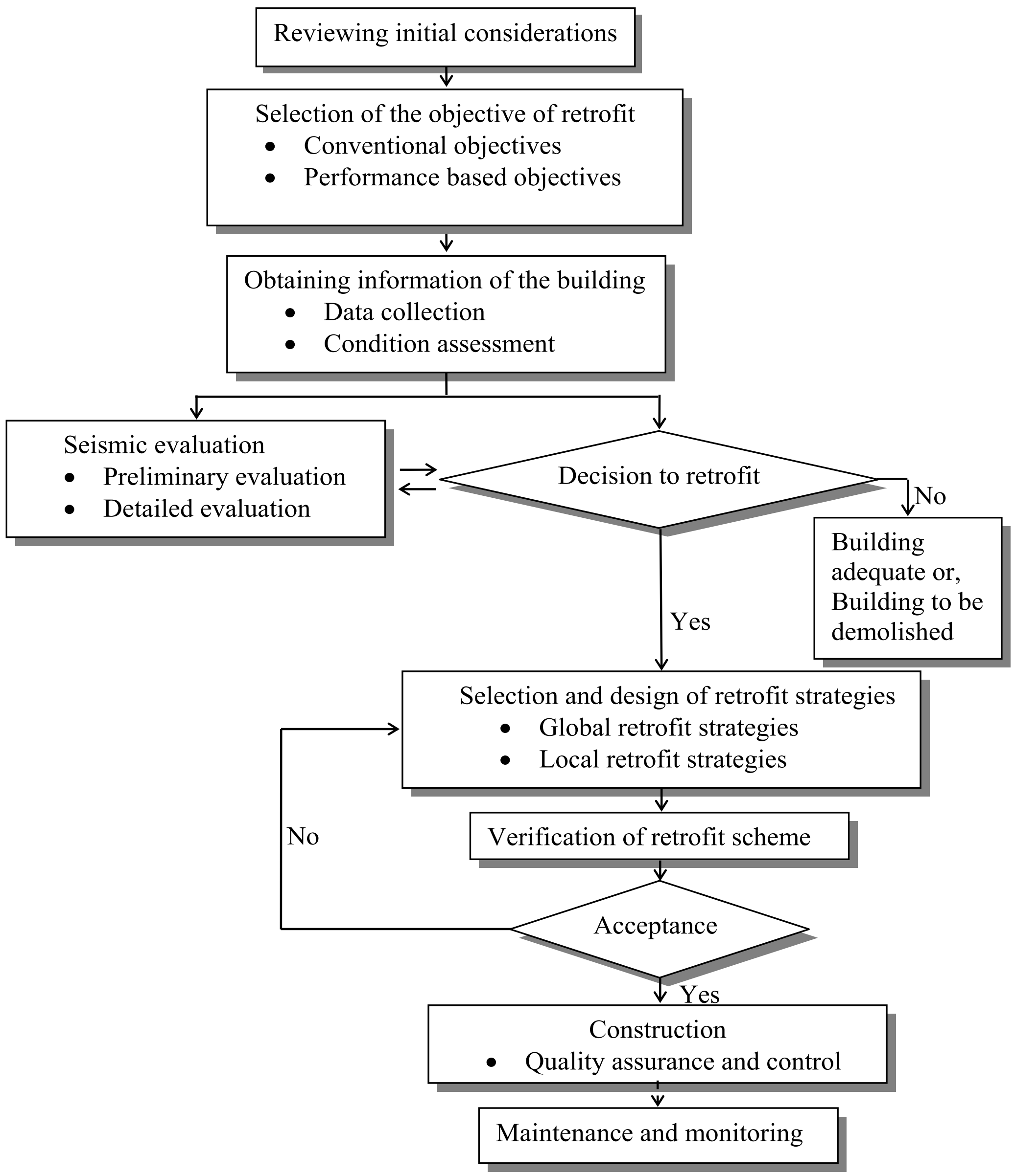


Figure 1.8 Flow chart of a retrofit programme

1.8 TERMINOLOGY

The words listed are categorised according to the context of usage. The General category lists the words which need proper engineering definition as per the parlance of earthquake engineering. Some of the words are used with different connotations in different countries, thus creating confusion among the professionals. Here, the words are defined based on the usage in the Handbook. In the categories of Earthquake and Geotechnical, a few words from seismology, soil mechanics and geology are selected. A familiarity with these words helps in understanding the aspects of earthquakes that are relevant in seismic design.

The words listed in the category of Building are necessary for analysing and understanding the behaviour of buildings / structures under seismic loads. Since a working knowledge of the materials is essential, the commonly used terms are listed in the category of Materials. The next three categories of Masonry Buildings, Reinforced Concrete Buildings and Steel Buildings list the materials, structural members and types of members that commonly exist in the three types of buildings

1.8.1 General

Acceptance criteria: The limiting values of quantities such as strength demand, inelastic deformation of a member or drift of a building that are used to determine the acceptability of the member or building.

Action: The internal axial force, moment, shear or torsion generated in a member due to the external loads.

Anchor: It is an object by which a component is attached to a medium such as concrete.

Base isolation system: The collection of units which are horizontally flexible and vertically stiff. It supports the weight of a building but reduces the transfer of ground motion.

Base shear: The summation of the lateral forces acting at the various levels of a building due to a base motion. It is equal to the shear acting at the lowest storey of the building.

Bonding agent: It is a medium capable of forming strong and tough adhesive between two components.

Boundary element (member/component): An element at the edge or opening of a shear wall or horizontal diaphragm that provides tensile and/or compressive strength.

Box system: A bearing wall structure without a space frame. The horizontal forces due to earthquake are resisted by the walls acting as shear walls.

Braced frame: It is a vertical lateral-load-resisting element consisting of columns (vertical), beams (horizontal) and braces (diagonal). A connection of a brace is not designed to resist moment.

Buttress: A wall-type structure built perpendicular to a wall or in the plane of a frame for support against lateral forces such as earthquake, wind, earth pressure or hydrostatic pressure.

Capacity: The permissible strength or deformation of a member or of the building calculated from the section and material properties.

Capacity (based) design: It is a type of design that targets a preferred type of failure of the building or of a component of the building by suitably adjusting the capacities to avoid other types of failure.

Centre-of-mass: The point at a floor of a building where the mass of the floor and tributary portions of the adjacent storeys can be considered to be lumped for the purpose of analysis.

Centre-of-rigidity (centre-of-stiffness): The point at a floor of a building such that when the horizontal force along a direction acts at that point, there is only translation of the level.

Chord reinforcement: Additional reinforcement provided at the edge of the slab or the beam supporting the slab perpendicular to the direction of seismic forces, for carrying the tension generated due to the diaphragm action of the slab.

Collector (drag strut / diaphragm strut): A member that transfers lateral forces from the diaphragm of the structure to vertical elements of the lateral-load-resisting system.

Compact section: A steel cross-section which can develop plastic moment but has inadequate plastic rotation capacity needed for formation of a plastic collapse mechanism of the member or structure. The width-to-thickness ratios of the plates are limited.

Component (member): An element of a building such as slab, beam, column, shear wall, footing, infill wall, stair slab, water tank is termed as a component. A component is classified as a structural component if it participates in resisting loads. Else, the component is classified as a non-structural component.

Composite beam: It consists of a steel beam and part of a concrete floor that are integrally connected to deflect as a single unit under loads.

Condition assessment: The process of gathering information and evaluating the current condition of a structure or a member in relation to its use.

Concentric bracing: It is a type of bracing in a frame where the axis of a brace meets the beam-column joint at the point of intersection of the axes of the beam and column.

Concrete jacketing: A method in which a concrete column or beam is covered with concrete and reinforcing bars in order to strengthen the member.

Coupling beam: A component that ties or couples adjacent shear walls acting in the same plane.

Coupled shear walls: Two or more shear walls acting in the same plane that are connected adequately by beams.

Damping: It refers to the forces that resist the vibration of a structure and are related to the velocity of the structure.

Degrees-of-freedom: The number of variables required to specify a response parameter (for example deformation) of a structure completely is called the degrees-of-freedom.

Demand: The value of force or deformation in a member or of the building calculated from structural analysis.

Diaphragm: A floor or roof system that interconnects the lateral-load-resisting systems such as frames or walls and transmits the lateral forces to them. A diaphragm is classified as rigid or flexible depending upon its in-plane deformability.

Diaphragm chord: A component provided to resist tension or compression at the edges of a diaphragm.

Differential settlement: The ratio of difference in vertical settlement between two adjacent columns or between two adjacent parts of the same structure, to the horizontal distance between the columns/parts.

Drag reinforcement: Additional reinforcement provided at the edge of the slab or the beam supporting the slab parallel to the direction of seismic forces, for transferring the forces from the slab diaphragm to vertical elements of the lateral-load-resisting system.

Drift: The relative horizontal displacement between two levels of a building due to lateral loads such as from an earthquake. The displacement between two adjacent floors is termed as inter-storey drift.

Ductility: The ability of a structure or a component of the structure to deform under the load beyond the elastic range without significant loss of strength.

Durability: It is the ability to resist weathering action, chemical attack, abrasion or any other form of deterioration.

Dynamic analysis: It is a type of analysis in which the inertia forces and damping forces due to motion, the spring forces due to deformation of a structure and the applied forces are considered.

Eccentric bracing: It is a type of bracing in a frame where the axis of a brace does not meet the point of intersection of the axes of the beam and column but meets the beam away from the beam-column joint.

Energy dissipation device: It is an element designed to dissipate energy in a stable manner during repeated cycles of earthquake.

Epicentre: It is the point on the earth's surface directly above the hypocentre. The epicentre is described by the location of the point of fault rupture on the map in terms of latitude and longitude.

Epoxy resin: It is a polymeric agent capable of forming strong, tough, corrosion and chemical resistant adhesive between two components.

Equivalent static method: This method is based on "equivalent" static (independent of time) applied forces on a structure and the spring forces due to its deformation. The inertia forces and damping forces due to motion of the structure are not explicitly considered.

Fault: Plane or zone along which earth materials on opposite sides move differentially in response to tectonic forces.

Ferrocement: It is made of cement mortar reinforced with closely spaced layers of small diameter wire mesh.

Fibre reinforced concrete: It is made by adding fibrous materials (usually steel or glass) to fresh concrete, which improves the crack resisting properties.

Fibre reinforced polymer composite: It consists of a matrix of resin reinforced with fibres (glass, carbon or aramid).

Geotextile: It is a synthetic fabric used to protect the soil from erosion and to reinforce the soil at a slope for stability.

Gravity load resisting system: The frames, walls and foundation that resist the vertical loads from the roof and floor slabs in a building.

Ground Motion: It is the movement of the ground on which a building is supported. It is characterised by acceleration, velocity and displacement.

Grout: It is a mixture of water, cement and optional materials like sand, water-reducing admixtures, expansion agent and pozzolans. The desired properties are fluidity and minimum segregation during pumping, and strength and durability after hardening.

Horizontal seismic coefficient: It is the ratio of the base shear to the weight of a building. It includes the zone factor, importance factor, response reduction factor and the spectral acceleration.

Hypocentre: The point on a fault where rupture initiates is referred to as the 'focus' or hypocentre of an earthquake. The hypocentre is described by the depth of the point and its location on the map in terms of latitude and longitude.

Infill wall: A wall made of masonry or concrete and placed within a frame of a building. An infill wall is considered to be integral if the wall is connected to the frame by dowel bars or shear connectors. Else, it is considered to be non-integral.

Intensity: The intensity of an earthquake is a measure of the damage caused by an earthquake at a particular location. One way of measurement is the Modified Mercalli intensity scale.

Knowledge factor: A factor to represent the uncertainty of the available information about the structural configuration or present condition of the materials or components of an existing building.

Landslide: The lateral and downward movement of unstable soil/rock mass due to combination of gravity, earthquake, seepage or superimposed loading.

Lateral-load-resisting system: The collection of frames, shear walls, bearing walls, and inter-connecting horizontal diaphragms in a building that provides resistance to lateral loads such as due to an earthquake. The frames and walls are termed as vertical lateral-load-resisting elements.

Lateral spreading: The lateral flow of the soil mass within the ground due to inadequate bearing capacity or excessive settlement.

Lateral torsional buckling: It is the lateral deflection and twisting of a section due to the buckling of the compression flange of the section under loads.

Linear analysis: It is a type of analysis where the deformation in a member is considered to be proportional to the internal force.

Lintel band: A reinforced concrete or reinforced brick runner provided in the walls at the lintel level to tie them together and to impart horizontal bending strength in them.

Liquefaction: A phenomenon in which loose saturated soil loses the shear strength and behaves like a liquid. The earthquake (or other cyclic) loading reduces the soil volume and develops pore water pressure. This reduces the effective stress to zero and the soil is not able to support a structure.

Load bearing wall: A wall that provides vertical support for a floor or roof.

Load path: A course along which the seismic inertia forces are transferred from the superstructure to the foundation and finally to the ground.

Magnitude: The magnitude of an earthquake is a measure of the amount of energy released during an earthquake. One way of measurement is the Richter scale.

Mass: It is a measure of the amount of quantity at a particular level of a structure. The inertia forces are proportional to the masses during the vibration of the structure.

Micro-concrete: It is a type of concrete made of small aggregates for placement in thin sections.

Mitigation: Any action taken that has the potential to reduce the consequences of a future earthquake.

Mode shape: It is the deflection configuration when a structure vibrates under a natural frequency.

Moment resisting frame: It is a vertical lateral-load-resisting element consisting of columns (vertical) and beams (horizontal). A beam-column connection is designed to resist moment.

Natural frequency: It is equal to the number of cycles of vibration in one second when the structure vibrates without any external force.

Near source effect: It is the amplification of ground motion due to constructive interference of the earthquake waves in a strike slip fault.

Non-buckling brace (unbonded brace): It is a type of brace consisting of an inner core and an outer sleeve that overcomes the problem of buckling and low energy dissipation of regular braces.

Non-compact section: A steel section that has width-to-thickness ratios of the plates exceeding the limiting values for compactness.

Non-destructive test: The test of a component of a building which does not cause any damage.

Non-engineered buildings: The buildings which are not formally designed but built using traditional techniques.

Nonlinear analysis: It is a type of analysis where beyond a certain level of force, the deformation in a member is not proportional to the internal force.

Ordinary moment resisting frame: A moment resisting frame that does not have the ductile detailing for resisting seismic forces.

Overturning: It is the effect when the moment produced at the base of a building due to the lateral forces is larger than the resistance provided by the foundation's uplift resistance and building weight.

P-Δ effect: It is the generation of additional moment in a column due to the lateral deformation of the column or due to the relative horizontal displacement between two the two ends of the column.

Performance based analysis: It is a type of analysis which is based on quantifying the deformations of the members and the building as a whole under the seismic forces.

Plan irregularity: It is a deficiency of a building that can be detected by observation and simple calculations based on the plan of the building.

Plate tectonics: The movements of the earth crust (top layer) due to the flows of the mantle (layer beneath the crust). The crust consists of pieces which are termed as plates.

Plinth band: A reinforced concrete or reinforced brick runner provided in the walls at the plinth level to tie them together and to impart horizontal bending strength in them.

Polymer modified concrete: It is made by incorporating a polymer latex with fresh concrete, which improves the tensile properties of concrete.

Pounding: The action of two adjacent buildings coming in to contact during earthquake excitation because they are too close together and/or exhibit different dynamic drift characteristics.

Precast member: A concrete member that is cast away from it final position.

Prestressed member: A concrete member in which internal compressive stress is introduced in the concrete to counteract the tensile stress generated due to the service loads.
Primary component: A component that is part of a building's lateral load resisting system.

Pushover analysis: It is a type of nonlinear static analysis where the magnitudes of the lateral loads are incrementally increased, maintaining a predefined distribution pattern along the height of the building, until a collapse mechanism develops.

Rapid visual screening: It is a form of survey to identify the buildings which are expected to be more vulnerable under an earthquake. It is used to prioritise the buildings in a jurisdiction for further evaluation and retrofit for seismic forces.

Redundancy: The attribute of a building with alternative load paths by which the lateral forces are resisted. This allows the building to remain stable following the failure of any single component.

Refurbishment: Actions that improve or change the functionality of a building or its component. Refurbishment involves architectural modification without intervention of structural members. Refurbishment is also referred to as remodelling.

Rehabilitation: Actions that improve the strength of a structure or a member. The structure or member is either deteriorated or damaged due to reasons stated under 'Repair'. Rehabilitation includes repair. The action is intended to regain the original strength of the structure or member.

Reinforced masonry: A masonry wall with adequate vertical and horizontal reinforcement.

Repair: Actions that improve the functionality of a member of a structure. The member is either defective, deteriorated or damaged due to any reason such as earthquake, cyclone, flood, fire, explosion, vehicle collision, corrosion, cracking, insect infestation etc. The action may not be intended to regain the original strength of the member completely.

Response spectrum: It is a plot of the maximum value of a quantity (usually the acceleration) defining the movement of the mass of a single degree of freedom (single mass) system subjected to a base motion, with respect to the natural period of the system.

Response spectrum analysis: It is a type of linear analysis where the values of spectral acceleration corresponding to the natural frequencies of a structure are combined based on certain factors to get the total base shear.

Restoration: Actions that improve the strength and appearance of a structure. The term is used mostly for historical structures. Restoration may include repair for a deteriorated or damaged structure.

Retrofit: Actions that improve the strength and other attributes of the integrity of a structure or a member with respect to seismic forces. The structure or member need not be deteriorated or damaged. Retrofit may include repair or rehabilitation. The action is intended to mitigate the effect of a future earthquake.

Retrofit programme: The complete process of retrofitting a structure. It includes collection of data, seismic evaluation, decision to retrofit, selection of retrofit strategies and construction.

Retrofit scheme: A combination of retrofit strategies for improving the strength and other attributes of resistance of a structure to seismic forces.

Retrofit strategy (technique): A technical option for improving the strength and other attributes of resistance of a structure or a member to seismic forces.

Roof band: A reinforced concrete or reinforced brick runner provided in the walls at the roof level to tie them together and to impart horizontal bending strength in them.

Rubble masonry: A masonry wall made of undressed blocks and mortar.

Secondary component: A component that is not part of a building's lateral load resisting system. It may or may not actually resist some lateral loads.

Seismic assessment (evaluation): The process of gathering information and evaluating the current condition of a structure or a member for resistance to seismic forces.

Seismic band: A reinforced concrete, reinforced brick or wooden runner provided horizontally in the walls to tie them together and to impart horizontal bending strength in them.

Seismic belt: A cast-in-place ferro-cement plating installed on an existing masonry wall in lieu of seismic bands or vertical reinforcing bars.

Seismic strengthening: Actions that improve the strength and other attributes of the integrity of a structure or a member with respect to seismic forces. Both rehabilitation and retrofit as defined here involve seismic strengthening.

Seismic upgradation: Actions that improve the strength and other attributes of seismic resistance of a structure or a member to meet the requirements of the current versions of the codes on seismic analysis and design.

Seismic waves: It is the mode of transmission of the strain energy released during an earthquake. The seismic waves are classified in to primary (P) waves, secondary (S) waves, Love waves and Rayleigh waves.

Seismic zone: It is an area where the maximum intensity of ground motion during an earthquake is expected to be of similar value. A zone factor is assigned to each zone which is an estimate of the peak ground acceleration.

Settlement: The vertical downward movement of the structure due to elastic compression or consolidation of the soil layers under the foundation.

Shear wall: It is a wall in a building designed to resist lateral force in its own plane. If the building frame is properly connected to the shear wall, the drift of the building and the forces in the members of the frame reduce.

Short column: A column with reduced height due to surrounding parapet, infill wall etc. The reduced height is less than five times the dimension of the column in the direction of parapet / infill wall or 50% of the height of the typical columns at that storey.

Shotcrete: It is a method in which compressed air forces mortar or concrete through a nozzle to be sprayed on a surface, such as wall, at high velocity.

Site amplification: The increase of the ground motion at the foundation level as compared to the motion at the level of the underlying rock.

Site specific response spectrum: A response spectrum which considers the soil conditions and geology of a certain location.

Soil-structure interaction: It is the inter-dependent behavior of the foundation of a structure and the supporting soil. The stresses and deformation cannot be solved by static equilibrium equations. A detailed analysis is required considering the rigidity of the foundation and modulus of sub-grade reaction of soil.

Special moment resisting frame: A moment resisting frame that has the ductile detailing for resisting seismic forces.

Spectral acceleration: It is the value of the acceleration from the response spectrum corresponding to a natural period of a structure.

Static analysis: It is a type of analysis in which only the spring forces due to deformation of a structure and constant values (time-independant) of the applied forces are considered.

Stiffness: The resistance to deformation under load. The deformation can be deflection under vertical loads, rotation under moment, drift under lateral loads, etc.

Storey shear: The summation of the lateral forces acting at the various levels of a building above the storey under investigation due to a base motion.

Strength: The maximum axial force, moment, shear force or torsion that can be resisted by a member of the building.

Subsidence: The vertical downward movement at the ground level due to soil liquefaction or lowering of ground water table with or without a structure supported on it.

Tectonic plates: These are the major parts of the earth's outermost crust (also called lithosphere). The earthquakes are attributed to the movement of these parts.

Time history analysis: It is a type of analysis in which the time-wise variations of the inertia forces and damping forces due to motion, the spring forces due to deformation of a structure and the applied forces are considered.

Time period: It is the time taken by a single oscillation of a system under free vibration, that is without any external excitation.

Tsunamis: It is earthquake generated waves due to instantaneous rise in seabed. The waves travel with very high velocity. The wave height increases significantly as the waves reach the shallow depth zone along the coast.

Unreinforced masonry: A masonry wall without any vertical or horizontal reinforcement.

Vertical irregularity: It is a deficiency of a building that can be detected by observation and simple calculations based on the elevation of the building.

Weight: It is a measure of the downward force at a particular level of a structure or on the whole structure due to gravity.

Wing wall: A wall type structure built adjacent to a column in a frame for support against seismic forces.

1.9 SUMMARY

Seismic resistant design of new buildings and seismic retrofit of existing buildings are essential to reduce the vulnerability of the buildings during an earthquake. The present Handbook compiles the available information on seismic retrofit of buildings. This chapter introduces the attributes of seismic design and explains the goals, objectives and steps of seismic retrofit. A retrofit can be undertaken based on the conventional force based analysis and design. The objectives in this approach are to satisfy the strength of the members and stiffness of the building. Else, a performance based approach can be adopted when the expertise and tools are available. The objectives in this approach are to satisfy a building performance level under a selected earthquake level. The pushover analysis is suitable for a performance based approach of seismic retrofit. Guidelines are suggested to select the level of seismic retrofit of existing buildings so that the risk of failure involved is acceptable. Definitions of various technical terms used in the Handbook are also provided in this chapter.

1.10 ORGANISATION OF THE HANDBOOK

The Handbook is organised in to various chapters to present the subject of seismic retrofit of buildings in a systematic format. The basic concepts of seismic retrofit are elucidated in Chapter 0. Chapter 2 gives an overview of the vulnerability aspects of buildings that are relevant in seismic design. The rapid visual screening, data collection and preliminary evaluation are compiled in Chapter 3. Chapter 4 introduces the available techniques of assessing the condition of an existing building. The retrofit of non-engineered buildings and masonry buildings are covered in Chapters 5 and 6, respectively. The material on historical and heritage structures is placed in Chapter 7. The methods of structural analysis of engineered buildings are presented in Chapter 8. Chapters 9 and 10 cover the retrofit of reinforced concrete buildings and steel buildings, respectively. The geotechnical aspects of seismic hazards are explained in Chapter 11. Chapter 12 covers the retrofit of foundations. The use of composite materials in seismic retrofit is discussed in Chapter 13. Chapter 14 presents the advanced technology of energy dissipation and base isolation devices. The quality assurance and quality control aspects of construction are briefly introduced in Chapter 15. The case studies in Chapter 16 illustrate the steps and strategies of seismic retrofit through practical examples.

1.11 REFERENCES

1. ATC 40 (1996), “Seismic Evaluation and Retrofit of Concrete Buildings: Vol. 1”, Applied Technology Council, USA.
2. FEMA 356 (2000), “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, American Society of Civil Engineers, USA.
3. IS 800 (Draft), “Indian Standard Code of Practice for General Construction in Steel”, Bureau of Indian Standards.
4. IS 1893: 2002, “Criteria for Earthquake Resistant Design of Structures”, Part 1: General Provisions and Buildings, Bureau of Indian Standards.

5. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings – Code of Practice", Bureau of Indian Standards.
6. IS 13920: 1993, "Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces", Bureau of Indian Standards.
7. IS 13935: 1993, "Indian Standard for Repair and Seismic Strengthening of Buildings – Guidelines", Bureau of Indian Standards.
8. Basu, P. C. (2002), "Seismic Upgradation of Buildings: An Overview", The Indian Concrete Journal, The Associated Cement Companies Ltd., August, pp. 461-475.
9. Park, R. and Paulay, T. (1975), "Reinforced Concrete Structures", John Wiley & Sons.
10. NZSEE Study (2002), "Assessment and Improvement of the Structural Performance of Buildings in Earthquake", New Zealand Society for Earthquake Engineering.

2

INTRODUCTION TO SEISMIC ANALYSIS AND DESIGN

2.1 INTRODUCTION

The subject of earthquake engineering has received a major emphasis in recent years, and especially in the wake of the 2001 earthquake in Gujarat. It is now well realized that earthquake loads need to be considered explicitly in the design of structures. In this chapter, the causes and effects of earthquakes are described briefly. The characterisation of earthquakes and the design considerations are explained. Comments on good engineering practice are also given. Finally, the provisions in the Indian seismic codes are mentioned.

Earthquakes cause the ground to shake violently, thereby triggering landslides, creating tsunamis and floods, and causing the ground to heave and crack, resulting in large-scale destruction to life and property. The study of why and where of earthquakes comes under geology. The study of the characteristics of the earthquake ground motion and its effects on engineered structures are the subjects of earthquake engineering. In particular, the effect of earthquakes on structures and the design of structures to withstand earthquakes with low damage is the subject of earthquake resistant structural design. The secondary effects on structures, due to floods and landslides are generally outside its scope. However, local site effects such as ground subsidence, liquefaction and site amplification are studied under geotechnical earthquake engineering.

2.2 CAUSES AND EFFECTS OF EARTHQUAKES

Earthquakes are natural phenomena, which cause the ground to shake. The earth’s interior is very hot and in a molten state. As the lava comes to the surface, it cools and new land is formed. The lands so formed have to continuously keep drifting to allow new material to surface. According to the theory of plate tectonics, the entire surface of the earth can be considered to be made of several plates, constantly on the move. These plates periodically brush against each other or collide at their boundaries, giving rise to earthquakes. Therefore, regions close to the plate boundary are highly seismic and regions farther from the boundaries exhibit less seismicity. Earthquakes may also be caused by other actions such as underground explosions.

The Indian sub-continent, which forms part of the Indo-Australian plate, is pushing against the Eurasian plate along the Himalayan belt. Therefore, the Himalayan belt is highly seismic whereas peninsular India, which is not traversed by any plate boundary, is relatively less seismic. Earthquakes became frequent after the construction of Koyna dam and this is regarded as a classic case of man-made seismicity. However, the occurrence of the Latur earthquake of 1993, in what was previously considered to be the most stable region on the earth indicates that no region is entirely safe from devastating earthquakes.

Earthquakes usually originate at major cracks or fissures in the ground, called faults. The gradual movements of the plates generate strains at the faults, which are very irregular and when the strain energy becomes large enough to overcome the frictional forces, relative slip takes place, leading to a release of the energy in the form of earthquake waves.

The recent earthquake in Kutch, Gujarat on 26 Jan 2001 has exposed the weaknesses in the Indian construction industry. An important thing to remember is that, as of today, earthquakes cannot be predicted by any means scientific or otherwise.

Earthquake load differs from other loads in many respects, which makes it more difficult to design for it. Some of the characteristics of earthquake loading are as follows

1. Earthquake loading is uncertain with respect to its amplitude, duration, and frequency content.

2. Earthquake loading is predominantly lateral and can cause severe damage unless special provisions are made to resist them.
3. Earthquake loading is cyclic and induces reversal of stresses.
4. Earthquake loading is dynamic and produces different degree of response in different structures.

These characteristics make seismic analysis and design extremely difficult and time-consuming and so simplified procedures are often used in practice.

2.3 SIZE OF THE EARTHQUAKE

2.3.1 Magnitude and Intensity

The intensity is a qualitative description of the effects of the earthquake at a particular location, as evidenced by observed damage and human reactions at that location. Different scale have been developed for the measurement of intensities, as described below. The MSK scale is described in detail in IS 1893 (2002).

1. Rossi-Forrel scale (1880) -- 1 to 10
2. Modified Mercalli intensity (MMI) -- 1 to 12
3. Japanese Meteorological agency -- 1 to 7
4. MSK Scale -- 1 to 12

Earthquake magnitude is a quantitative measure of the size of the earthquake. *Ritcher Magnitude* is the most commonly used magnitude scale.

2.3.2 The Response Spectrum

The *response spectrum* is a plot of the maximum response (usually the acceleration S_a) of single-degree-of-freedom (SDOF) systems as a function of their natural period T. For design purposes, the smoothed average of a number of elastic response spectrums corresponding to various possible earthquakes at a particular site, known as the *smoothed elastic design response spectrum* (SEDRS), is used. The SEDRS is further simplified so that it can be represented by a set of equations corresponding to different period ranges. SEDRS is usually specified for different soil conditions (IS 1983 -2002).

Most structures, such as multi-storeyed buildings, are multi-degree-of-freedom (MDOF) systems whose response can be approximated by considering only the first few natural modes. This fact is used to great advantage in *modal spectral analysis*, where the first few natural vibration mode shapes are calculated as a first step. Each mode can then be considered to represent the vibration shape of an SDOF with a corresponding natural period and so its maximum response can be directly determined from the response spectrum. The total response of the structure can then be calculated as a combination of these individual responses. A variety of ways are available to combine the individual responses considering the fact that these maximum responses occur at different instants of time. When the natural periods are sufficiently apart, the most common way of combining the maximum responses is by taking the square root of the sum of the squares (SRSS) method (Clough and Penzien 1993).

2.4 EARTHQUAKE RESISTANT DESIGN AND CONSTRUCTION

The key for good seismic design is simplicity in plan and elevation. Structures, which have more than one axis of symmetry and have uniform distribution of strength and stiffness are said to be *regular structures*. Structures, which do not satisfy one or more of the above requirements, are said to be irregular. The IS 1893 (2002) code defines several types of irregularities. Irregular structures exhibit special problems during earthquakes and should be avoided as far as possible.

Masonry and infill (non-structural) walls should be reinforced by vertical and horizontal reinforcing bands to avoid their failure under a severe earthquake. It should be noted that wood is not ductile and needs to be reinforced with steel to withstand severe earthquakes. Also other non-structural elements should be carefully designed so that they do not cause injury to people.

Reinforced Concrete elements should be detailed as per IS 13920-1993 which requires extra stirrups at potential hinging locations and extra anchorage lengths. It should be remembered that steel structures perform better than RC structures and should be adopted for all important buildings such as schools, multi-storied buildings and hospitals. Pre-cast elements should be tied securely so that they don’t get dislodged during the earthquake.

Projecting elements such as porches, sun-shades, water tanks, balconies and parapet walls cause serious injuries to people and so should be designed to withstand earthquake loading without developing instability.

Some of the common irregularities and their effects along with possible retrofitting strategies are described below.

1. Asymmetric plan/Asymmetric structural action

Building plan with one or no axis of symmetry or building with asymmetric structural action (Figure 2.1).

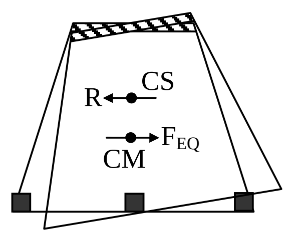


Figure 2.1 Asymmetric plan building

- Earthquake induced inertia force acts at centre of mass, CM.
- Building resistance acts at the centre of stiffness, CS.
- Resulting couple twist the building.

Affect all types of buildings such as masonry, reinforced concrete with or without shear walls and steel buildings (Figure 2.2).

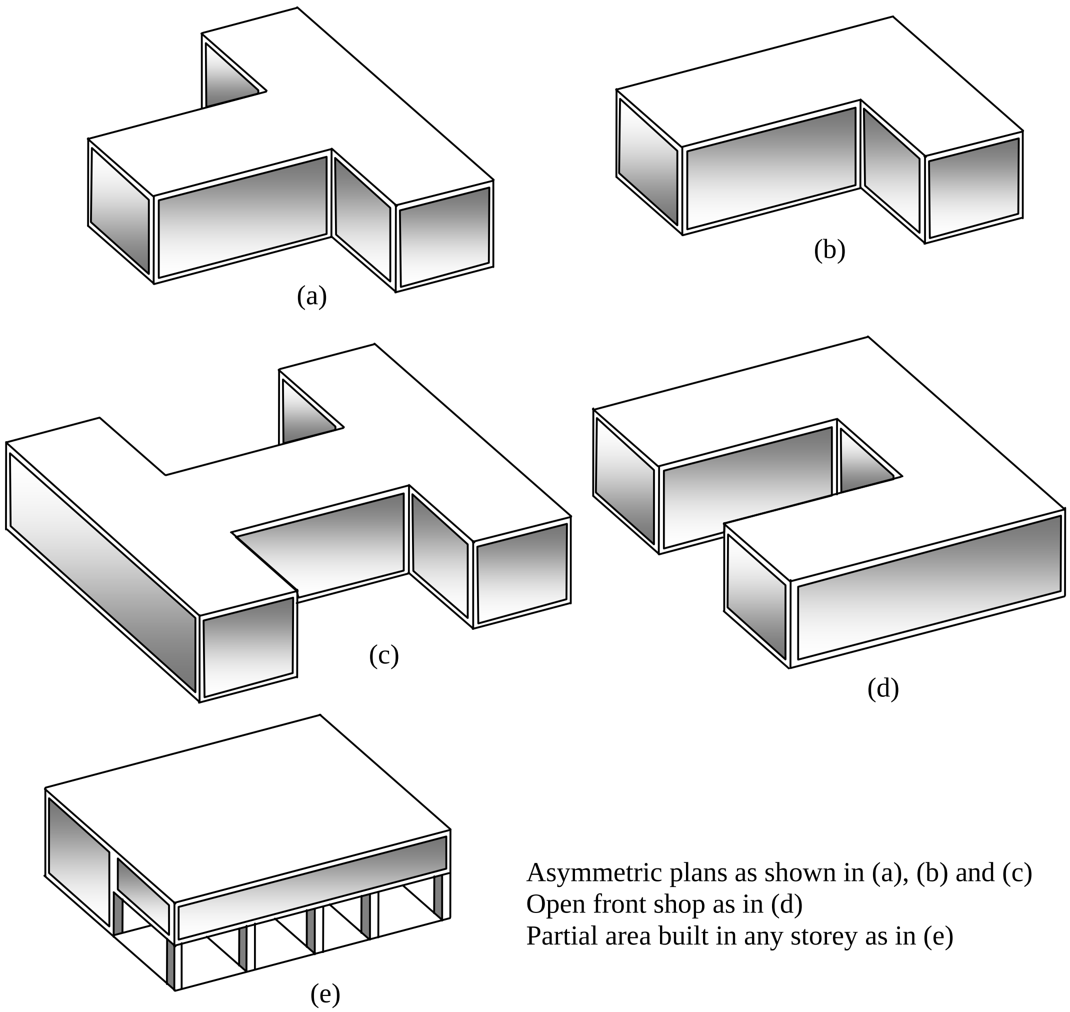


Figure 2.2 Asymmetric structural action

Retrofitting strategy: reduce asymmetry to bring CM near CS.

<u>2. Reentrant corners</u>

Building plans with re-entrant corners (Figure 2.3).

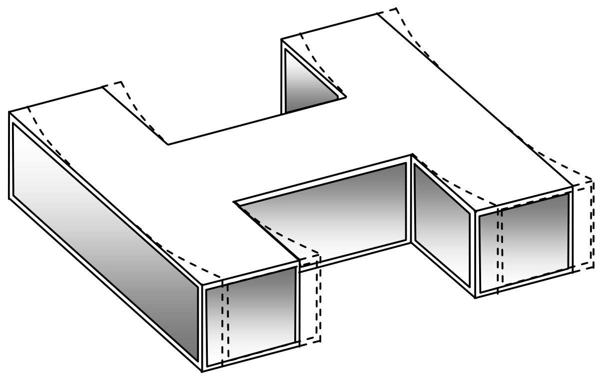


Figure 2.3 Buildings with re-entrant corners

- Re-entrant corners tend to open and close during vibration.
- Opening leads to cracking and closing leads to crushing at reentrant corners.

Affect all types of buildings such as given in Figure 2.4.

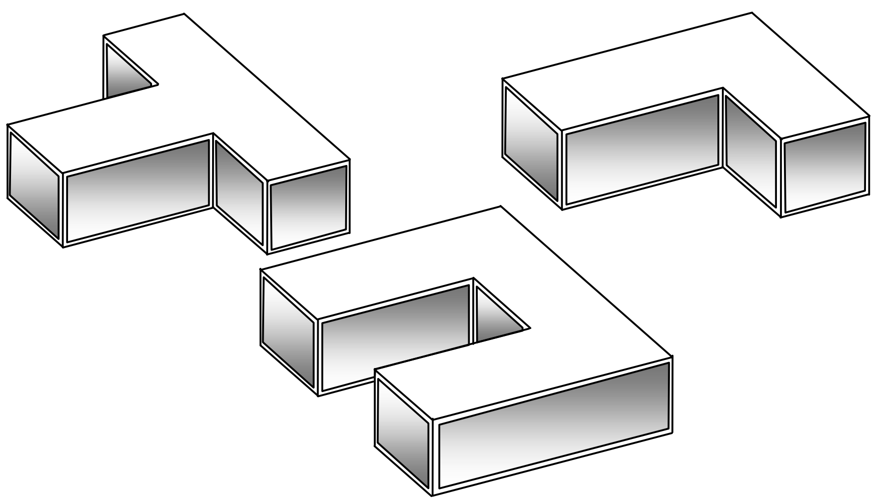
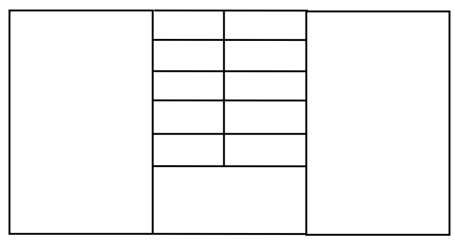


Figure 2.4 Buildings affecting due to re-entrant corners

Retrofitting strategy: Cut plan into separate wings (Ref IS:4326)

3. Open to sky, Ducts /atriums

Building with large open to sky ducts and/or staircases placed between floor areas (Figure 2.5).



Plan

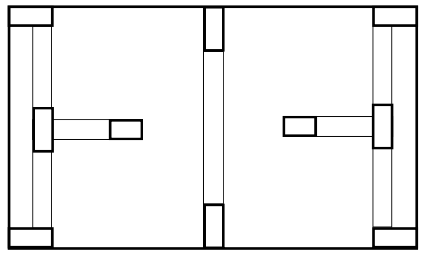
Figure 2.5 Building plans with open to sky ducts and staircases placed between floor areas

- Cutting of floor diaphragms for light, ventilation, staircases and lifts disturbs force transfer between floor areas.
- Affect multi-storeyed building with reinforced concrete slabs over floor areas.

Retrofitting strategy: Install horizontal steel bracings in ducts and separate the staircase. (Ref: IS - 4326)

4. Staggered column buildings

Buildings with columns not in line and/or oriented in different directions. Buildings with 230mm columns (Figure 2.6)



Plan

Figure 2.6 Buildings with staggered columns and 230 mm columns

- Inadequate frame action to resist seismic loads in either direction.
- 230mm columns are too weak and flexible for buildings over 2-storeys.
- Affect reinforced concrete framed buildings

Retrofitting strategy: Provide reinforced concrete shear walls over entire height.

5. Stilt floor buildings

Buildings with full or partial area of ground floor having stilts to facilitate parking or other activity, like shops with rolling shutters (Figure 2.7).

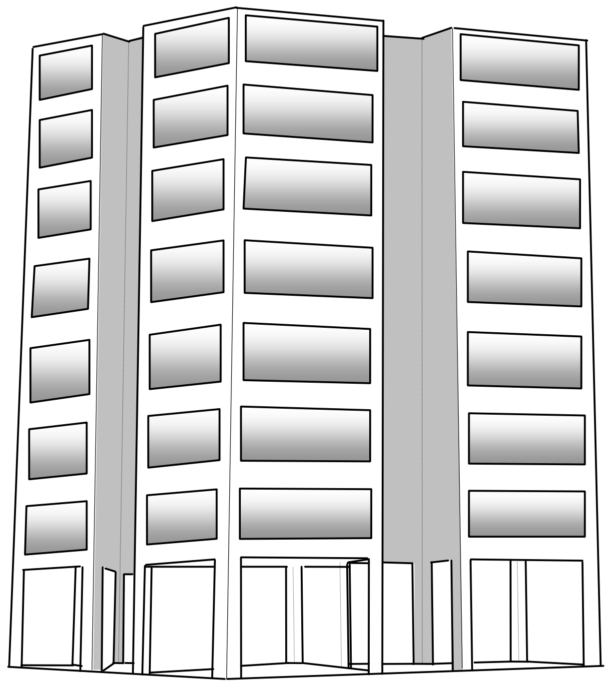


Figure 2.7 Buildings with full or partial area of ground floor having stilts

- Absence of walls reduces stiffness of ground storey making it “soft”
- The soft storey gets damaged during earthquake and the building tends to sit due to crushing of columns.
- Affect reinforced concrete buildings with strong masonry walls in upper floors.

Retrofitting strategy: Stiffen the ground storey by RC shear walls or steel bracings.

6. Plaza type buildings

Buildings with large built-up areas in lower storeys and a tower rising above as in hotels shopping mall cum office buildings (Figure 2.8).

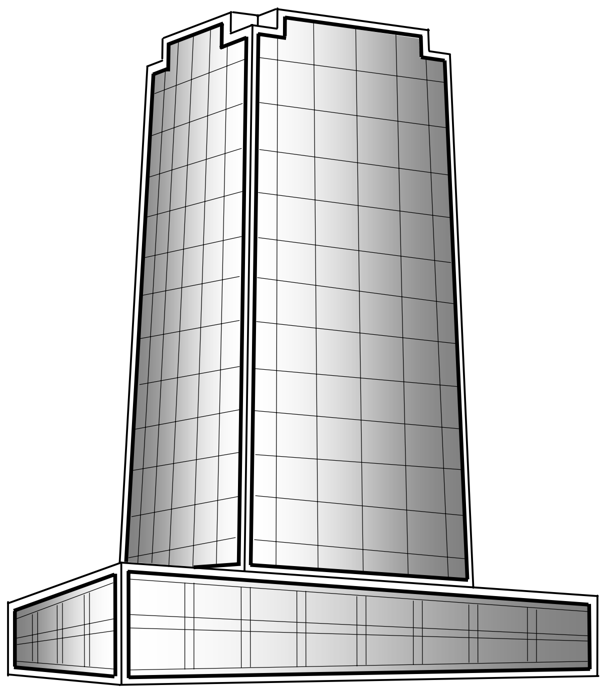


Figure 2.8 Plaza type buildings

- Sudden reduction in stiffness causes damage at the base of tower.
- Affect all types of buildings including modern reinforced concrete buildings and older/historic masonry buildings.

Retrofitting strategies: Provide steel bracing and/or reduce tower height.

7. Clustered buildings

Buildings close to one another as in city business areas and sometimes having common walls (Figure 2.9).

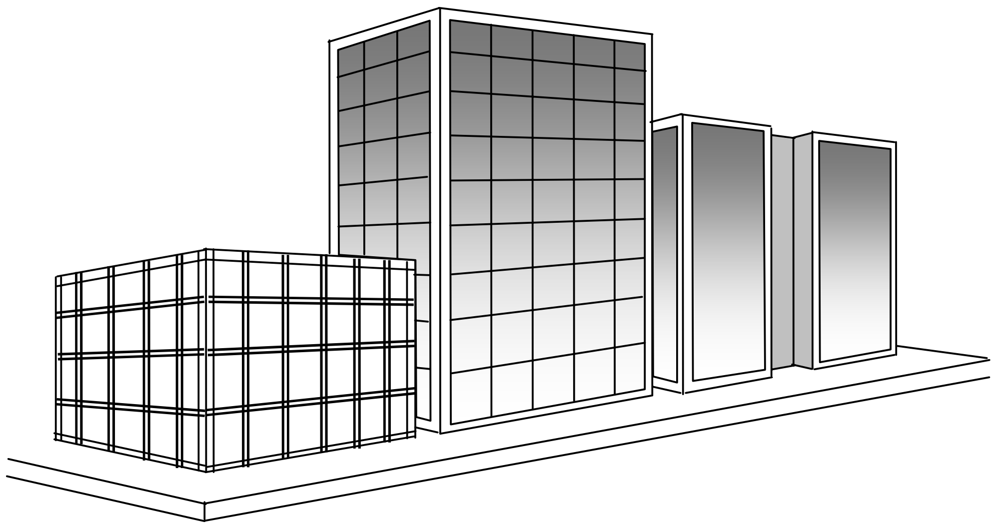


Figure 2.9 Closely spaced buildings

- Building hit or pound each other during earthquakes due to insufficient space for vibration.
- Affect all types of buildings.

Retrofitting strategy: Increasing gap by demolishing or provide bridge bearings in between.

<u>8. Non-ductile buildings</u>

Reinforced buildings not detailed as per IS: 13920 (Figure 2.10) and masonry buildings without reinforcement bands.

- **Member b/D > 0.3, b > 200**
- **Hoop spacing < d/4, 8ϕ, 100 mm elsewhere < d/2**
- **Anchorage Ld+10 ϕ**
- **%p < 2.5**
- **Column stirrups @ 75 c/c over *lo***
- **Hoop legs < 300 mm else cross ties**
- **Confining stirrups in footing**

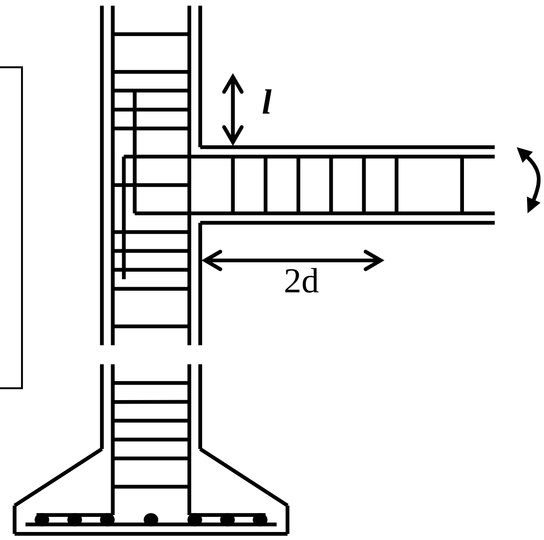


Figure 2.10 Ductile Detailing of RC

- Buildings disintegrate due to inadequate integral action.
- Affect reinforced concrete and masonry buildings.

Retrofitting strategy: Provide extra frames and tie all elements together.

<u>9. Buildings with projecting elements.</u>

Buildings with large projections like canopies, balconies, sunshades, parapets, and water tanks in the roofs (Figure 2.11).

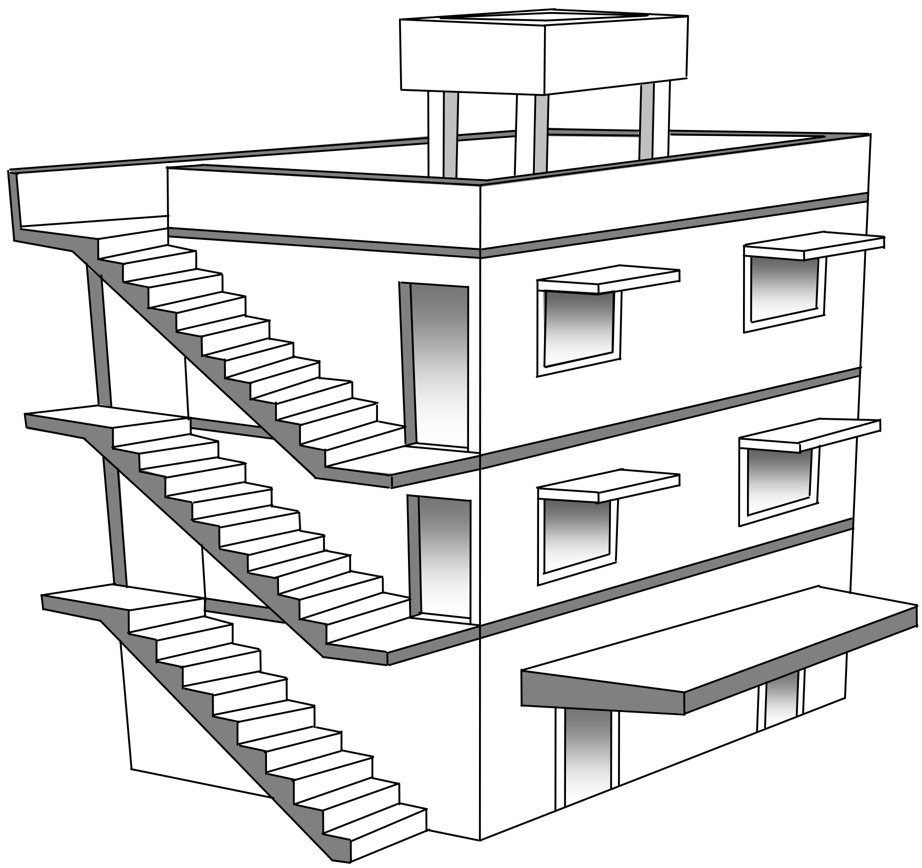


Figure 2.11 Buildings with large projections and elevated water tanks

- Horizontal projecting elements generally develop stability problems and tend to overturn.
- Vertically projecting elements experience amplified excitations and so develop stability problems.
- Affect all types of buildings except light weight sheetings.

Retrofitting strategy: Reduce projections or reduce their weight. Alternatively they may be braced or anchored to main elements.

2.4.1 Seismic Design Philosophy

Severe earthquakes have an extremely low probability of occurrence during the life of a structure. If a structure has to resist such earthquakes elastically, it would require an expensive lateral load resisting system, which is unwarranted. On the other hand, if the structure loses its aesthetics or functionality quite often due to minor tremors and needs repairs, it will be a very unfavourable design. The usual strategy is:

> *'to ensure elastic behaviour under a design basis earthquake which has a return period equal to the life of the structure and prevent collapse under the maximum credible earthquake'.*

For example, if the expected life of the structure is fifty years, then it is designed to remain elastic under an earthquake, which is likely to occur at least once in fifty years. Thus, no major repair will be necessary as a consequence of such earthquakes during the life of the structure. However, structures are designed to prevent collapse and loss of life under the most severe earthquake. The reason for adopting such a strategy is that it is extremely expensive to design structures to respond elastically under severe earthquakes, which may not occur during their expected life. Thus, it is well worth the risk to let them get damaged beyond repair in case the severe earthquake occurs, the chances of which are low.

The important properties of structures, which contribute to their elastic resistance under moderate earthquakes, are its yield strength and elastic stiffness. During a severe earthquake, the structure is likely to undergo inelastic deformations and has to rely on its *ductility* to avoid collapse. *Ductility* is the property, which allows the structure to undergo large plastic deformations without significant loss of strength or stiffness.

In addition to strength requirements at the ultimate load, structures are also designed to have adequate stiffness in the lateral direction under service loads. This is usually ensured, by limiting the relative displacement between successive floors, known as the *storey drift*. For buildings, a maximum allowable storey drift of *0.004* times the storey height is normally used under moderate earthquakes.

Although all of the above mentioned concepts are important for ensuring the safety of structures during a severe earthquake, one should keep in mind the great uncertainty associated with the seismic behaviour of structures. Past earthquakes have demonstrated that simplicity is the key to avoid unforeseen effects and so attention given at the planning stage itself can go a long way in ensuring safety and economy in seismic design.

2.4.2 Analysis and Design for Earthquake Loads

Structures are usually designed for gravity loads and checked for earthquake loading. In conformity with the design philosophy, this check consists of two steps – the first ensures elastic response under moderate earthquakes and the second ensures that collapse is precluded under a severe earthquake. Due to the uncertainties associated in predicting the inelastic response, the second check may be dispensed with, by providing adequate ductility and energy dissipation capacity. In this section, the various methods of performing these checks are described.

The important factors, which influence earthquake resistant design are, the geographical location of the structure, the site soil and foundation condition, the importance of the structure, the dynamic characteristics of the structure such as the natural periods and the properties of the structure such as strength, stiffness, ductility, and energy dissipation capacity. These factors are considered directly or indirectly in all the methods of analysis.

The response spectrum method has the advantage that, it can account for irregularities as well as higher mode contributions and gives more accurate results. Therefore, this is the most widely used method in seismic analysis. The method proposed by the Indian code is explained in the following sections.

2.5 THE INDIAN SEISMIC CODES

There are several codes giving details on design, detailing and construction of earthquake resistant structures. The code IS 1893-2002, given guidelines for calculating the design seismic loads on buildings. IS 13920 – 1993 gives guidelines for detailing reinforced concrete buildings for ductile performance. Similar details for steel structures are given in the revised IS 800 -2005. Guidelines for earthquake resistant construction are given in IS 4326 – 1993. There are also separate codes for mud masonry (IS13827-1993) and stone masonry constructions (IS13828-1993).

2.5.1 General Design Requirements

The IS 1893 code gives guidelines to classify buildings as regular or irregular based on simple calculations. If a building is regular, a simplified method of analysis can be adopted. The following types of irregularities are defined in the code:

Plan Irregularities

1. Torsion Irregularity: To be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resist the lateral forces. Torsional irregularity to be considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structures transverse to an axis is more than 1.2 times the average of the storey drifts at the two ends of the structure.
2. Re-entrant Corners: Plan configurations of a structure and its lateral force resisting system contain re-entrant corners, where both projections of the structure beyond the re-entrant corner are greater than 15 percent of its plan dimension in the given direction.
3. Diaphragm Discontinuity: Diaphragms with abrupt discontinuities or variations in stiffness, including those having cut-out or open areas greater than 50 percent of the gross enclosed diaphragm area, or changes in effective diaphragm stiffness of more than 50 percent from one storey to the next.
4. Out-of-plane offsets: Discontinuities in a lateral force resistance path, such as out-of-plane offsets of vertical elements.
5. Non-parallel systems: The vertical elements resisting the lateral force are not parallel to or symmetric about the major orthogonal axes or the lateral force resisting element.

Vertical Irregularities

1. a) Stiffness Irregularity – Soft Storey: A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above

1. b) Stiffness Irregularity – Extreme Soft Storey: A extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category.

2. Mass Irregularity: Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs.
3. Vertical Geometric Irregularity: Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the lateral force resisting system in any storey is more than 150 percent of that in its adjacent storey.

4. In-Plane Discontinuity in Vertical Elements Resisting Lateral Force: A in-plane offset of the lateral force resisting elements greater than the length of those elements.
5. Discontinuity in Capacity – Weak Storey: A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above. The storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

2.5.2 Calculation of Earthquake Loads

The horizontal seismic coefficient A_h takes into account the location of the structure by means of a zone factor Z, the importance of the structure by means of a factor I and the ductility by means of a factor R. It also considers the flexibility of the structure-foundation system by means of an acceleration ratio Sa/g , which is a function of the natural time period T. This last ratio is given in the form of a graph known as the response spectrum. The horizontal seismic coefficient A_h is given by

$$A_n = \frac{Z\ I\ S_a}{2\ R\ g}$$

where

Z = Zone factor corresponding to the seismic zone obtained from a map (Table 2.2)
I = Importance factor
R = Response reduction factor.

Table 2.2 Zone factor, Z

Seismic Zone	II	III	IV	V
Seismic Intensity	Low	Moderate	Severe	Very Severe
Z	0.10	0.16	0.24	0.36

For important service and community buildings, such as hospitals, schools; monumental structures; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire station buildings; large community halls like cinemas, assembly halls and subway stations, power stations, the importance factor may be taken as 1.5 but for most buildings it may be taken as 1.0.

The response reduction factor R is also specified for various buildings depending on the type of detailing adopted. For example, if a building is detailed as per ductile detailing provisions given in IS 13920, then the R factor is 5, else it is 3.

The natural time period T is very important and should be calculated correctly. For single storey structures, it may be taken as $T = 2\pi\sqrt{(k/m)}$ where k is the lateral (horizontal) stiffness of the supporting structure and m is the mass of the roof usually taken as the sum of the roof dead load plus 50% of the live load divided by the acceleration due to gravity.

Fundamental natural period(sec) of MR frame buildings without infills (solid walls),

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

$Ta = 0.085\ h^{0.75}$ for steel buildings

where, h= height of building in m.

And, for frames with infills,

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

where

d= base dimension of the building at the plinth level in m

Finally, the acceleration ratio Sa/g can be obtained from the graph corresponding to Ta and the soil type as shown in Figure 2.12. In this figure, medium soil corresponds to stiff clay or sand and soft soil corresponds to loose clay and loamy soils.

The base shear is then given by

$$V_b = A_h W$$

The base shear calculated above is then distributed along the height of the building using the formula,

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2}$$

where,

Q_i is the lateral force at the top of floor *i*,

W_i is the total of dead and appropriate amount of live load at the top of floor *i*,

h_i is the height measured from the base of the building to the top of floor *i*,

n is the number of storeys.

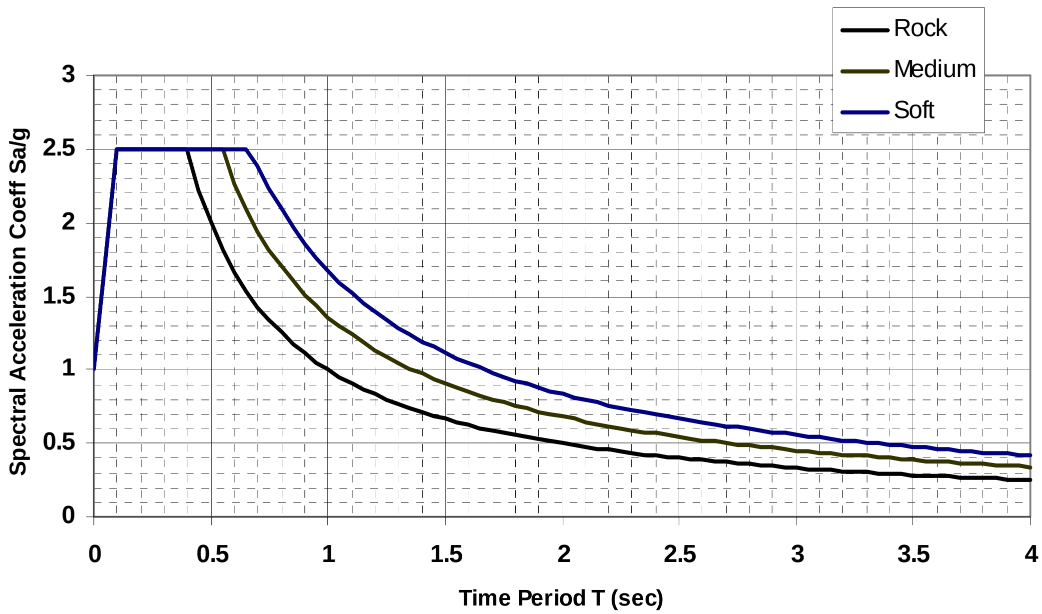


Figure 2.12 Response spectrum for 5% damping

According to IS 1893 code, the load combinations to be checked are as follows:

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

2.5.3 Capacity Design and Detailing

With reference to framed structures, it has been found that some collapse mechanisms ensure larger energy dissipation capacities compared to some other collapse mechanisms. The technique of ensuring a preferred collapse mechanism by suitably adjusting the capacities of the members is called *Capacity Design*. In practice, due to the difficulties associated with inelastic analysis and design, no attempt is made to calculate the actual capacities in relation to seismic demand and it is only ensured that the members and joints of the structure have adequate ductility and energy dissipation capacities and the structure as a whole will fail in a preferred collapse mechanism.

The type of collapse mechanism developed largely dictates the overall ductility and energy dissipation capacity of the frame and so capacity design is invariably carried out. In capacity design, the type of collapse mechanism required is pre-decided and attempts are made to make sure that no other mechanism develops. In multi-story frames, the strong column-weak beam mechanism is preferred since this mechanism requires the formation of many plastic hinges and also the plastic rotation capacity required at the hinges is less (Figure 2.13). The term plastic rotation capacity is used in place of ductility when talking about the moment-rotation curve instead of the usual force-deformation curve (this is explained in the chapter on plastic analysis).

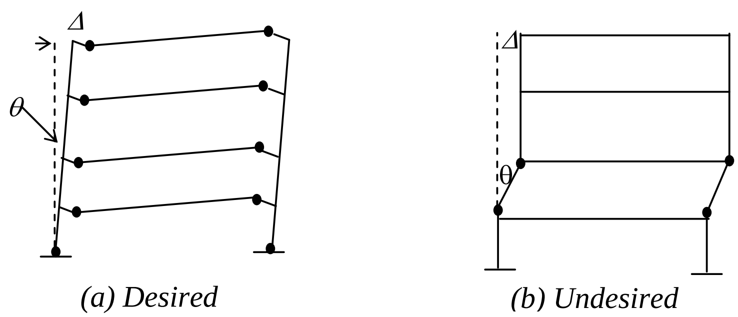


Figure 2.13 Desired and Undesired Collapse Mechanisms in Capacity Design

2.6 SUMMARY

The characteristics of earthquake loads were described. The dual strategy of ensuring elastic response under moderate earthquakes and preventing collapse under a severe earthquake was explained. The properties of the structure, particularly ductility and hysteretic energy dissipation capacity, which aid in resisting earthquake loads, were pointed out. The architectural considerations, which can simplify the design process and assure good seismic performance, were described.

The elastic and inelastic response prediction methods such as seismic coefficient, response spectrum and time-history analysis were explained. The background concepts on which most codal provisions are based were also explained. Guidelines to improve the seismic behaviour of steel structures were given at the material, member and structure levels. In particular, the hysteretic behaviour and collapse modes of bracing members and flexural members with various cross-sections were described in detail. The behaviour of lateral load resisting systems such as bracings and moment resistant frames was described. The concept of capacity design, which aims at maximising the energy dissipation capacity of moment resisting frames by choosing an appropriate collapse mechanism, was explained. Finally, an overview of special devices and systems, which can be used to control the response and thus reduce the design forces for members, was given.

2.7 REFERENCES

1. IS 1893: 2002, “Criteria for Earthquake Resistant Design of Structures”, Bureau of Indian Standards, New Delhi.
2. IS 4326: 1993, “Earthquake Resistant Design and Construction of Buildings”, Bureau of Indian Standards, New Delhi.
3. IS 13827: 1993, “Improving Earthquake Resistance of Earthen Buildings”, Bureau of Indian Standards, New Delhi.
4. IS 13828: 1993, “Improving Earthquake Resistance of Low-Strength Masonary Buildings”, Bureau of Indian Standards, New Delhi.
5. IS 13920: 1993, “Ductile Detailing of Reinforced Concrete Structures subjected to Seismic Forces”, Bureau of Indian Standards, New Delhi.
6. Bolt, B. A. (1988), “Earthquakes”, W. H. Freeman and Company, New York
7. Clough, R. W. and Penzien, J. (1975), “Dynamics of Structures”, McGraw-Hill, New York.
8. Naeim F. (1996), “The Seismic Design Handbook”, Van Nostrand Reinhold, New York.
9. Green, N. B. (1978), “Earthquake Resistant Building Design & Construction”, Van Nostrand Reinhold Company, New York.
10. Key, D. E. (1988), “Earthquake Design Practice for Buildings”, Thomas Telford, London.

3

RAPID VISUAL SCREENING, DATA COLLECTION AND PRELIMINARY EVALUATION

3.1 OVERVIEW

The initial step in a seismic retrofit programme can be Rapid Visual Screening (RVS). When there are several buildings within a jurisdiction, the RVS is a form of survey to identify the buildings which are expected to be more vulnerable under an earthquake and need further seismic evaluation. The RVS for masonry buildings is given in IS 13935, *Seismic Evaluation, Repair and Strengthening of Masonry Buildings – Guidelines* (Draft version, 2005). For low-rise masonry buildings, further evaluation may not be possible due to lack of resources or trained professionals. For such buildings, there are recommendations for simple retrofit schemes. The RVS procedures for reinforced concrete and steel buildings are presently at a draft stage, yet to be published (Arya, 2003). In this chapter, the general methodology of RVS is explained.

To facilitate seismic evaluation of a building, it is necessary to collect relevant data of the building, as much as possible and systematically. This step in a retrofit programme is referred to as Data Collection. Data sheets for systematic data collection are provided for ready reference. The proposed tables in the data sheets are adopted from the *Model Town and Country Planning Legislation, Zoning Regulations, Development Control, Building Regulation/Byelaws for Natural Hazards Zones of India* (Draft version, 2004).

For an engineered building identified for seismic evaluation, the preliminary evaluation first involves a set of initial calculations to identify areas of potential weaknesses in the building. These calculations are called configuration-related or strength-related checks. The preliminary evaluation also checks the compliance with the provisions of the seismic design and detailing codes. This is conveniently done by completing a check-list. It is recommended to perform a preliminary evaluation before undertaking a detailed evaluation. The methodology from ASCE 31-03 (2003) is presented in a simplified form.

3.2 RAPID VISUAL SCREENING

> As the name suggests, the Rapid Visual Screening involves a quick assessment of a building based on visual inspection alone. It is a kind of statistical guideline to the inspectors to identify and inventory the vulnerable buildings.

The Rapid Visual Screening (RVS) procedures are simple enough to be used by an inspector with proper training. The steps of RVS are planning for the survey, execution of the survey and interpretation of the results.

The main uses of RVS are as follows.

1. To identify if a particular building requires further evaluation for its seismic vulnerability.
2. To assess the seismic damageability (probability of damage) of the building and seismic retrofit needs.
3. To identify simplified retrofit schemes for the buildings for which further evaluation is not considered necessary or not found to be feasible.

Since the RVS is based on visual inspection, the results may occasionally vary from that of a detailed analysis. For example, a building found to be vulnerable by RVS may perform better under the detailed analysis.

The RVS as proposed in the Indian codes is based on the classification of damage for the varied types of buildings under earthquakes of different intensities. These concepts are summarised in Annex D of IS 1893: 2002. There are three important aspects of the RVS procedure. First, the buildings of one group (say masonry buildings) are classified into different types based on the vulnerability under seismic forces. Second, the states of damage of the same group of buildings are classified into grades. Third, a table provides an estimate of the distribution of damage for each type of building in a particular seismic zone. Based on these three aspects, data sheets have been developed for the collection of the required information and the interpretation of the results. These aspects are discussed for masonry, reinforced concrete and steel buildings.

3.2.1 Masonry Buildings

The RVS procedure for masonry buildings is included in IS 13935, *Seismic Evaluation, Repair and Strengthening of Masonry Buildings – Guidelines* (Draft version, 2005). The masonry buildings with load bearing walls are classified into the following seven types: A, A+, B, B+, C, C+ and D. Out of the buildings under the same alphabet type, those under a '+' type have some better features. A brief description of each type is given below. Expanded descriptions are available in Table A-1 of IS 13935.

1. **A**: The walls are made of rubble masonry (field stone) with mud mortar.
2. **A+**: The walls are made of un-burnt blocks or bricks with mud mortar.
3. **B**: The walls are made of semi-dressed rubble masonry, bricks or concrete blocks with mud mortar or weak lime mortar.
4. **B+**: The walls are same as in Type B, but strengthened with vertical or horizontal wood members, or, if lime mortar is used for the walls in Type B.
5. **C**: The walls are made of dressed stone masonry or concrete blocks or burnt bricks with lime or cement mortar. The walls are connected by reinforced concrete slab or horizontal bracing or seismic band.
6. **C+**: The walls are same as in Type C with additional seismic band at lintel level.
7. **D**: The walls are same as in Type C with additional seismic band at lintel level and vertical reinforcement. Else, the walls are made of confined masonry with horizontal and vertical reinforcement.

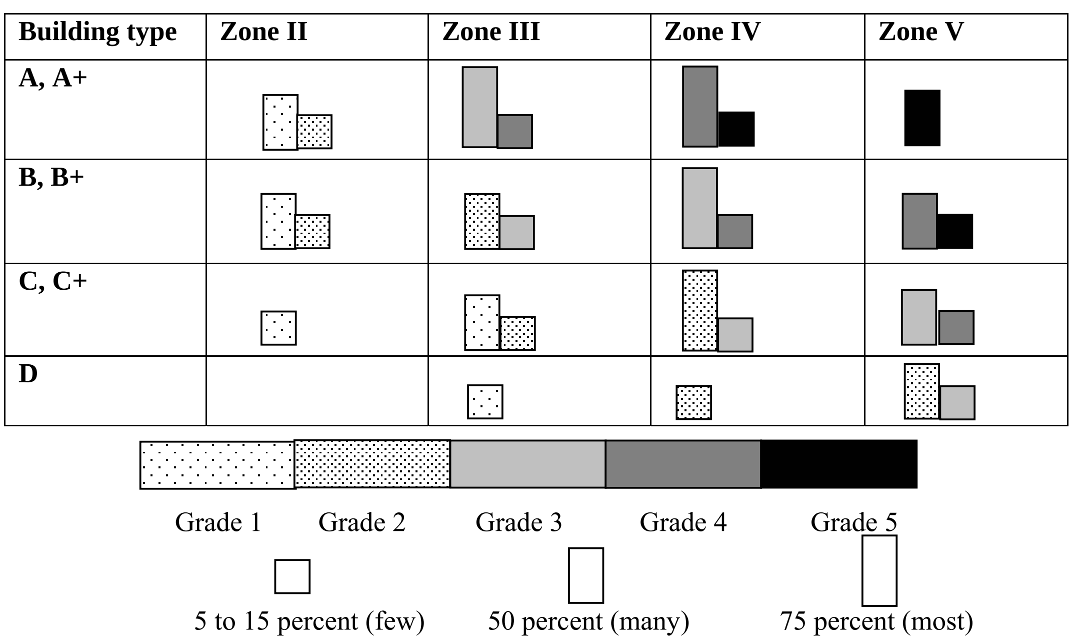
The states of damage of masonry buildings are classified into five grades. The connotation of each grade is given below briefly. The detailed descriptions are available in Table A-2 of IS 13935. The damages in structural and non-structural elements are referred to as structural damage and non-structural damage, respectively.

1. <u>Grade 1 (Negligible to slight damage):</u> There is no structural damage, but the building may have slight non-structural damage, such as fine cracks in the plaster over the walls.

2. Grade 2 (Moderate damage): There is slight structural damage such as cracks in walls and thin cracks in slabs. There is moderate non-structural damage such as fall of plaster from the walls and damage to parapets and sun-shades.
3. Grade 3 (Substantial to heavy damage): This refers to moderate structural damage along with heavy non-structural damage. There are large cracks in walls and piers. The roof tiles get detached and there is failure of the partition walls.
4. Grade 4 (Very heavy damage): There is heavy structural damage like failure of walls and partial failure of roofs.
5. Grade 5 (Destruction): This implies very heavy structural damage like collapse of the building.

Table A-3 of IS 13935 provides an estimate of the distribution of damage for each type of masonry building in the different seismic zones. The information in the table is represented in the form of bar graphs in Table 3.1. The shading of a bar represents a grade of damage. The height of a bar represents the percentage of the buildings of the particular type expected to be damaged under an earthquake of the intensity specific for the seismic zone. The seismic zones are as per IS 1893 (Part 1): 2002.

Table 3.1 Damageability of masonry buildings



It can be observed that for a particular type of masonry building, the estimated damage increases in a higher seismic zone. Also, within a specific seismic zone, the damage is higher for a more vulnerable building type.

There are four data sheets for the four seismic zones. The information to be filled up is grouped under the following sections.

1. Building data
2. Occupancy
3. Special hazard
4. Falling hazard
5. Probable damageability
6. Recommended action.

Figure 3.1 shows a data sheet schematically.

Seismic Zone V

Photograph

Sketch Plan with Length and Breadth

Building Name
Use
Address
Other Identifiers
No. of Storeys Year Built
Total Covered Area; all floors (sq. m)
Ground Coverage (sq. m)
Soil Type Foundation Type
Roof Type Floor Type
Structural Components
Wall Type: BB Earthen UCR CCB
Thickness of Wall Slab Thickness
Mortar Type: Mud Lime Cement
Vertical R/F Bars: Corners T-junctions Jambs
Seismic Bands: Plinth Lintel Eaves Gable

OCCUPANCY	SPECIAL HAZARDS	FALLING HAZARDS
Important buildings: Hospitals, schools; emergency buildings; large community halls; power stations, VIP residences.	1.High water table and if sandy soil, liquefiable site indicated.	1.Chimneys ☐
	2. Land slide prone site	2.Parapets ☐
Any building having more than 100 occupants may be treated as **important*	3. Severe vertical irregularity	3.Cladding ☐
Ordinary buildings: Other buildings having occupants < 100	4. Severe plan irregularity	4.Others ☐

Recommended action:
1) A, A^+ or B, B^+: evaluate in detail for need of reconstruction or possible retrofitting to achieve type C^+ or D.
2) C: evaluate in detail for need for retrofitting to achieve type C^+ or D.
3) Wood: evaluate in detail for retrofitting.
4) If any Special Hazard found, re-evaluate for possible prevention/retrofitting.
5) If any of the falling hazard is present, either remove it or strengthen against falling.

Surveyor's signature
Name:
Date:

Executive Engineers's signature:
Date of survey:

Probable damageability in few/many buildings

Building type	Masonry Building			
	A / A^+	B / B^+	C / C^+	D
Damageability in Zone V	G5 / G4	G5 / G4	G4 / G3	G3

Figure 3.1 Data sheet for rapid visual screening of masonry buildings

The required information under each section is briefly explained below.

1. Building data

The building data covers overall information of the building. It consists of the name, use, and address of the building, number of storeys, year in which the building was completed, total covered area, ground coverage, soil type, foundation type, structural components, roof type, floor type, thickness of slabs, wall type, thickness of wall, mortar type, presence of vertical reinforcing bars and seismic bands.

2. Occupancy

The information on occupancy is required to categorise the building as either 'important' or 'ordinary'. For the important buildings, the data sheet for a zone higher than the zone in which the building is located (except for Zone V) should be selected.

3. Special hazard

The presence of high water table in sandy soil, susceptibility to landslide, occurrence of vertical or plan irregularity are noted under this section.

4. Falling hazard

The falling hazard includes chimneys, parapets, sunshades, cladding and any other object susceptible to cause damage or injury.

5. Probable damageability

In this section, the building type is identified and the corresponding damageability is encircled. A table is provided which is an abridged form of Table A-3, IS 13935 (Table 3.1 given earlier). The information provides an estimate of the distribution of damage for the identified type of building in the particular seismic zone.

6. Recommended action

Based on the type of building, some actions are recommended. These include further seismic evaluation and retrofitting.

3.2.2 Reinforced Concrete and Steel Buildings

The RVS procedures for reinforced concrete and steel buildings are in the draft stage, yet to be published (Arya, 2003). The proposed RVS procedure for reinforced concrete (RC) and steel buildings is similar to that of masonry buildings. The buildings are classified into the following six types: C, C+, D, E, E+ and F. A brief description of each type is given below.

1. **C**: Any building which is not properly designed for wind or earthquake forces.
2. **C+**: A building similar to those in Type C, but has moment resisting frames.
3. **D**: A moment resisting framed building which is designed for earthquake forces, but does not have the special detailing for earthquake resistance.
4. **E**: A moment resisting framed building which is designed for earthquake forces and provided with the special detailing for earthquake resistance.
5. **E+**: An RC building similar to Type E, but in addition has well designed infill walls. A steel building similar to those in Type E with well designed braces.
6. **F**: An RC building similar to Type E, but additionally with well designed shear walls. A steel building similar to those in Type E with well designed and detailed braces. A building with base isolation is also classified in this type.

A building which has severe vertical irregularity such as soft / weak storey, or has floating columns or in which the plinth beams are absent, is not covered in the above classification. Such a building needs further seismic evaluation.

The states of damage of RC buildings are classified into the following five grades.

1. Grade 1 (Negligible to slight damage): There is no structural damage, but the building may have slight non-structural damage, such as fine cracks in the plaster over frame members or in the infill walls.
2. Grade 2 (Moderate damage): There is slight structural damage such as cracks in columns, beams or structural walls. There is moderate non-structural damage such as cracks in the infill walls and fall of plaster or mortar from the walls.
3. Grade 3 (Substantial to heavy damage): This refers to moderate structural damage along with heavy non-structural damage. There are cracks in columns, beams, beam-column joints and joints of coupled walls. There may be buckling of reinforcing bars.

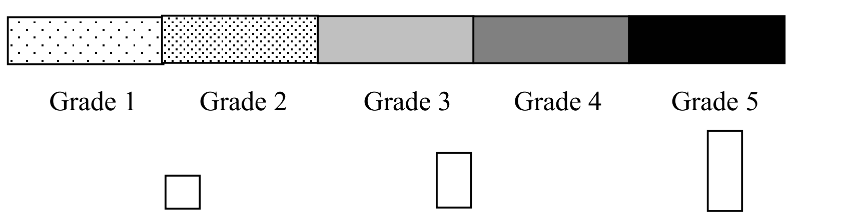
4. Grade 4 (Very heavy damage): There is heavy structural damage like large cracks, crushing of concrete and failure of rebar in the frame members. There can be collapse of a few columns or of a single upper floor.

5. Grade 5 (Destruction): This implies very heavy structural damage like collapse of the ground storey.

Table 3.2 provides an estimate of the distribution of the damage for each type of building in the different seismic zones. Similar to Table 3.1, the shading of a bar represents a grade of damage and the height of a bar represents the percentage of the buildings of a particular type expected to be damaged.

Table 3.2 Damageability of RC / steel buildings

Building type	**Zone II**	**Zone III**	**Zone IV**	**Zone V**
C,C+				
D				
E,E+				
F				



It can be observed that for a particular type of building, the estimated damage increases in a higher seismic zone. Within a specific seismic zone, the damage is higher for a more vulnerable building type. The proposed RVS data sheets are similar to that for masonry buildings (Figure 3.2). The required information is collected and the damageability is identified. Based on the type of building, some actions are recommended.

Rapid Visual Screening of Indian Buildings for Potential Seismic Hazards

Seismic Zone V

Elevation to Scale	Building Name: Use: Address: Other Identifiers: No. Stories: Year Built: Total Floor Area (Sq.m)
Plan to Scale	Photograph

OCCUPANCY		SPECIAL HAZARDS	FALLING HAZARDS
□ □ □ □ *Resi:* Ordinary / Important School, Health, Assembly, Office, Commercial, Historic, Emergency Service, Induatrial.	Max. Number of Persons 0-10 11-50 51-103 >100 Residents: Floating:	High W T (Within 8m) Liquefiable (if sandy soil) Site. Land Slide Prone Site Severe Vertical Irregularity	Chimney, Parapets, Cladding other

Building Type	Masonry Building				R C or Steel Frame Building				URM Infill	Wood
	A /A+	B / B+	C / C+	D	C / C+	D	E / E+	F		
Damage Grade in Zone V	G5 / G4	G5 / G4	G4 / G3	G3	G4 / G3	G3	G2 / G1	G1	G4	G4

Note: +sign indicates higher strength hence somewhat lower damage expected as 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:

1) A, A^+ or B, B^+: evaluate in detail for need of reconstruction or possible retrofitting to achieve type C or D.
2) C, C^+: evaluate in detail for need for retrofitting to achieve type D
3) URM infill: evaluate for need of reconstruction or possible retrofitting to level D
4) Wood: evaluate in detail for Retrofitting
5) In Case of Special Hazard, evaluate for possible

Surveyor's signature
Name:
Date:

3.2.3 Rapid Visual Screening as per FEMA 154 and 155

Applied Technology Council, USA, proposed an RVS procedure in the publications FEMA 154 and FEMA 155. The procedure uses a scoring system which is based on a probabilistic hazard analysis. In the data collection form, for a particular type of building, the scoring system consists of a basic structural hazard (BSH) score and a set of score modifiers. The BSH score is the probability of collapse of a low-rise regular building for a certain ground motion. For the building under study, the BSH score is modified by the score modifiers to arrive at a final score. The score modifiers account for the irregularities and other features of the specific building. The final score is an estimate of the probability of collapse of the building. It is compared with a cut-off score and a decision on the need for a detailed analysis is undertaken.

3.3 DATA COLLECTION

> In order to facilitate seismic evaluation, it is necessary to collect relevant data of a building as much as possible through drawings, enquiry, design calculations, soil report (if available), inspection reports, reports of previous investigation, previous repair works, any complaints by the occupants etc. A site visit is essential for data collection.

The data of the building includes the plan, necessary elevations, the basic architectural features, material properties and their deterioration and other helpful information. In order to have a systematic data collection, the following tables are provided.[1] Each of the following tables has three columns. The left column gives the description of the information to be collected. The middle column is to be filled up during the data collection. The right column gives options for filling up the middle column or reference to the relevant clause numbers from the codes. The first table (Table 3.3) covers the generic information of an existing building and the parameters required for seismic analysis. The second table (Table 3.4) checks the compliance of a load bearing masonry building with the essential provisions of IS 4326: 1993. The third table (Table 3.5) checks the compliance of an RC framed building with the provisions of IS 456: 2000 and IS 13920: 1993. The fourth table (Table 3.6) checks the compliance of a steel building with the provisions of IS 800 (Draft). It may not be possible to fill up the information from the original design if the drawings are not available.

[1] The tables were adopted from the *Model Town and Country Planning Legislation, Zoning Regulations, Development Control, Building Regulation/Byelaws for Natural Hazards Zones of India* (Draft version, 2004). The original tables are part of structural design basis report which should accompany the building development permission. They are modified to be used for data collection.

Table 3.3 Building survey data sheet: General data

S. No.	Description	Information	Notes
Building Description			
1	Address of the building • Name of the building • Plot number • Locality / Township • District • State		
2	Name and type of owner / tenant		Private / government
3	Name of builder		
4	Name of architect		
5	Name of engineer		
6	Use of building		Residential / office / commercial / industrial
7	Year of construction and subsequent remodelling, if any		
8	Plan size (approximate)		
9	Building height		
10	Number of storeys above ground level		
11	Number of basements below ground level		
12	Type of structure • Load bearing wall • RC frame • RC frame and shear wall • Steel frame		
13	Open ground storey	Yes / No	
14	Roof-top water tank, heavy machinery or any other type of large mass	Yes / No	
15	Architectural features		
16	Expansion / Separation joints		

17	Photograph / sketch		Attach with sheet
Survey			
1	Visited building site	Yes / No	
2	Structural drawings available	Yes / No	
3	Architectural drawings available	Yes / No	
4	Geotechnical report available	Yes / No	
5	Construction specifications available	Yes / No	
6	Designer contacted	Yes / No	
Exposure condition			
1	Environment		Hot / temperate / cold Dry / wet / humid Industrial / coastal etc.
2	Deterioration noticed		
Geotechnical and geological			
1	Type of soil		Soft / medium / hard rock IS 1893:2002, Table 1
2	Type of foundation		Isolated / combine footing / pile / raft
3	Design safe bearing capacity		IS 1904: 1986
4	Footings on sloping ground	Yes / No	
5	Seismic zone		IS 1893: 2002, Figure 1
6	History of past earthquakes		
7	Presence of liquefaction-susceptible, saturated, loose granular soil at foundation level	Yes / No	
8	Building situated close to slope	Yes / No	

	susceptible to fail under earthquake		
9	Building situated close to known surface fault	Yes / No	
Variables for analysis			
1	Dead loads (unit weights) • Masonry • Concrete • Steel • Other materials		IS 875: 1987 (Part 1)
2	Imposed (live) loads • Floor loads • Roof loads		IS 875: 1987 (Part 2)
3	Wind loads		IS 875: 1987 (Part 3)
4	Snow loads		IS 875: 1987 (Part 4)
5	Safe bearing capacity		IS 1904: 1986
6	Importance factor, I		IS 1893: 2002, Table 6
7	Seismic zone factor, Z		IS 1893: 2002, Table 2
8	Response reduction factor, R		IS 1893: 2002, Table 7
9	Fundamental natural period, T		IS 1893: 2002, Cl. 7.6
10	Horizontal seismic coefficient (A_h)		IS 1893: 2002, Cl. 6.4
11	Seismic design lateral force		
12	Any other assumed data		

Table 3.4 Building survey data sheet: Building Data (Load bearing masonry buildings)

<table>
<tr><th>S. No.</th><th>Description</th><th>Information</th><th>Notes</th></tr>
<tr><td>1</td><td>Building category</td><td></td><td>IS 4326: 1993, Section 7
Read with IS 1893: 2002<table><tr><td rowspan="2">Building</td><td>Zones</td></tr><tr><td>II III IV V</td></tr><tr><td>Ordinary</td><td>B C D E</td></tr><tr><td>Important</td><td>C D E F</td></tr></table></td></tr>
<tr><td>2</td><td>Type of wall masonry</td><td></td><td></td></tr>
<tr><td>3</td><td>Type and mix of mortar</td><td></td><td>IS 4326: 1993, Cl. 8.1.2</td></tr>
<tr><td>4</td><td>Size and position of the openings
• Minimum distance (b_5)
• Ratio $(b_1+b_2+b_3)/l_1$ or $(b_6+b_7)/l_2$
• Minimum pier width between consecutive openings (b_4)
• Vertical distance (h_3)
• Ratio of wall height-to-thickness
• Ratio of wall length between cross wall to thickness</td><td></td><td>IS 4326: 1993, Table 4, Figure 7.</td></tr>
<tr><td>5</td><td>Horizontal seismic band
• At plinth level
• At window sill level
• At lintel level
• At ceiling level
• At eave level of sloping roof
• At top of gable walls
• At top of ridge walls</td><td>Yes / No</td><td>
IS 4326: 1993, Cl. 8.4.6

IS 4326: 1993, Cl. 8.4.2
IS 4326: 1993, Cl. 8.4.3

IS 4326: 1993, Cl. 8.4.4</td></tr>
<tr><td>6</td><td>Vertical reinforcing bar
• At corners and T-junction of wall
• At jambs of doors and window opening</td><td>Yes / No</td><td>
IS 4326: 1993, Cl. 8.4.8
IS 4326: 1993, Cl. 8.4.9</td></tr>
<tr><td>7</td><td>Integration of prefab roofing/flooring elements through reinforced concrete screed</td><td>Yes / No</td><td>IS 4326: 1993, Cl. 9.1.4</td></tr>
<tr><td>8</td><td>Horizontal bracing in pitched truss
• In horizontal plane at the level of ties
• In the slopes of pitched roofs</td><td>Yes / No</td><td></td></tr>
</table>

Table 3.5 Building survey data sheet: Building Data (RC framed buildings)

S. No.	Description	Information	Notes
1	Type of building • Regular frames • Regular frames with shear wall • Irregular frames • Irregular frames with shear wall • Open ground storey		IS 1893: 2002, Cl. 7.1
2	Horizontal floor system • Beams and slabs • Waffle slab • Ribbed floor • Flat slab with drops • Flat plate without drops		
3	System of interconnecting foundations • Plinth beams • Tie beams		IS 1893: 2002, Cl. 7.12.1
4	Grades of concrete used in different parts of building		
5	Method of analysis		
6	Computer software used		
7	Base shear a) Based on approximate fundamental period b) Based on dynamic analysis c) Ratio of a)-to-b)		IS 1893: 2002, Cl. 7.5.3
8	Distribution of seismic forces along the height of building		IS 1893: 2002, Cl. 7.7.1
9	Torsion included		IS 1893: 2002, Cl. 7.9
10	The columns of soft ground storey specially designed		IS 1893: 2002, Cl. 7.10
11	Clear minimum cover provided in • Footing • Column • Beams • Slabs • Walls		IS 456: 2000, Cl. 26.4

12	Ductile detailing of RC frame • Type of reinforcement used • Minimum dimension of beams • Minimum percentage of reinforcement of beams at any cross section • Spacing of transverse reinforcement at any section of beam • Spacing of transverse reinforcement in 2d length of beam near the ends • Ratio of shear capacity to flexural capacity of beams • Beam bar splices location and spacing of hoops in the splice • Minimum dimension of columns • Maximum percentage of reinforcement in column • Confining stirrups near ends of columns and in beam-column joints – Diameter – Spacing • Column bar splices location and spacing of hoops in the splice • Ratio of shear capacity of columns to flexural capacity of supported beams		IS 456: 2000, Cl. 5.6 IS 13920: 1993, Cl. 6.1 IS 13920: 1993, Cl. 6.2.1 IS 13920: 1993, Cl. 6.3.5 IS 13920: 1993, Cl. 6.3.3 IS 13920: 1993, Cl. 6.2.6 IS 13920: 1993, Cl. 7.1 IS 456: 2000, Cl. 26.5.3 IS 13920: 1993, Cl. 7.4, Section 8 IS 13920, Cl. 7.2.1 IS 13920: 1993, Cl. 7.3.4

Table 3.6 Building survey data sheet: Building Data (Steel buildings)

S. No.	Description	Information	Notes
1	Adopted method of design • Simple • Semi rigid • Rigid		IS 800 (Draft), Cl. 4.2.1
2	Design based on • Elastic analysis • Plastic analysis		IS 800 (Draft), Cl. 4.4 and Cl. 4.5
3	Floor construction • Composite • Non-composite • Boarded		
4	Roof construction • Composite • Non-composite • Metal • Any other		
5	Horizontal force resisting system • Moment Frames • Braced frames • Frames and shear walls		IS 800 (Draft), Section 12
6	Slenderness ratios maintained		IS 800 (Draft), Cl. 3.8.1
7	Structural members • Encased in concrete • Not encased		
8	Structural connections • Rivets • High strength friction grip bolts • Black bolts • Welding field		IS 800 (Draft), Section 10

3.4 PRELIMINARY EVALUATION

> The purpose of the preliminary evaluation is to identify the areas of seismic deficiencies in an engineered building before a detailed evaluation is undertaken.

In the preliminary evaluation, first some calculations are done to study the seismic vulnerability of the building. The checks to be investigated are classified into two groups: configuration-related and strength-related. The checks presented below are based on the draft document *Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings*. Next, a check list of evaluation statements is to be completed. The evaluation statements express desirable attributes, which if non-compliant, suggest that further investigation is required. In this chapter, the methodology from ASCE 31-03 (2003) is presented in a simplified form.

3.4.1 Configuration-related checks

The configuration-related checks examine the structural integrity of a building. The checks are briefly explained.

1. Load path: Is there a complete load path for the seismic forces?
2. Geometry: Is there more than 50% change in the horizontal dimension of the lateral force resisting system within storeys?
3. Weak storey: Is there a weak storey?
4. Soft storey: Is there a soft storey?
5. Vertical discontinuity: Is there a vertical discontinuity in the lateral load resisting system?
6. Mass: Is there a change in mass more than 100% between storeys?
7. Torsion: Is the estimated distance between the centre-of-mass and centre-of-rigidity greater than 30% of the building dimension?
8. Adjacent buildings: Is the clear distance between two buildings sufficient?
9. Short columns: Are there short columns due to partial infill walls, parapets?
10. Redundancy: Is there sufficient number of frames in each principal direction?
11. Deterioration of concrete: Is there visible deterioration of the concrete or reinforcing steel?
12. Mezzanine / loft / sub-floor: Are these components braced or anchored to the primary lateral load resisting system?

3.4.2 Strength-related Checks

The strength-related checks are simple calculations of quantities that reflect the demands in the primary members due to the seismic forces. The calculations are based on the base shear (V_B) and the distribution of the base shear to the floor levels of a building. The calculations of V_B and Q_i, the design lateral force at floor *I*, are explained in Chapter 2, Introduction to Seismic Analysis and Design.

Shear Stress in Reinforced Concrete Frame Columns

The average shear stress in the columns (assuming that nearly all the columns in the frame have similar stiffness) is given as follows.

$$v_{avg} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right) \qquad \textbf{(3.1)}$$

Here,

n_c = total number of columns,
n_f = total number of frames in the direction of loading,
A_c = total cross-sectional area of columns in the storey under consideration,
V_j = storey shear at level *j*.

If v_{avg} exceeds $0.1\sqrt{f_{ck}}$, it reflects large seismic demand. The characteristic strength of concrete is represented as f_{ck} (in N/mm^2).

Shear Stress in Shear Walls or Un-reinforced Masonry Bearing Walls

The average shear stress can be approximately calculated as follows.

$$v_{avg} = \frac{V_j}{A_w} \qquad \textbf{(3.2)}$$

Here,

V_j = storey shear,
A_w = total horizontal cross sectional area of all walls in the direction of loading.

If v_{avg} exceeds $0.1\sqrt{f_{ck}}$ for a shear wall or 0.2 N/mm^2 for a masonry wall, it reflects large seismic demand.

Axial Stress in Columns

The approximate axial force in columns subjected to overturning is given by the following equation.

$$P = \frac{3}{4}\left(\frac{V_B}{n_f}\right)\left(\frac{H}{L}\right) \tag{3.3}$$

Here,

H = height of the building from the base,
L = total length of the building
n_f = total number of frames in the direction of loading.

If the axial stress calculated from P exceeds 0.25 f_{ck}, it reflects large seismic demand.

Axial Stress in Braces

The approximate average axial stress in the braces of a braced frame is given by the following equation.

$$f_j^{avg} = \left(\frac{V_j}{sN_{br}}\right)\left(\frac{L_{br}}{A_{br}}\right) \tag{3.4}$$

Here,

A_{br} = average area of a diagonal brace
L_{br} = average length of the braces
N_{br} = number of braces in tension and compression if the braces are designed for compression; number of braces in tension if the braces are designed for tension only.
s = average span length of braced spans
V_j = storey shear at level j.

3.4.3 Evaluation Statements

The evaluation statements express desirable attributes, which if non-compliant, suggest that further investigation is required. The evaluation statements should be marked "compliant" (C), "non-compliant" (NC) or "not applicable" (NA). Any statement marked NC needs further investigation.

The evaluation statements are presented under the following tables.[2]

[2] The evaluation statements in ASCE 31-03 are presented for two performance levels, life safety and immediate occupancy and three levels of seismicity, low, moderate and high. In this chapter the evaluation statements are presented without any reference to the performance levels or levels of seismicity. The presented statements are for life safety performance level under any level of seismicity. Some checks are

1. Building system: In this category, attributes related to the building as a whole are listed.
2. Vertical irregularities: Each attribute refers to an absence of a vertical irregularity.
3. Plan irregularities: Each attribute refers to an absence of a plan irregularity. The irregularities are defined as per IS 1893: 2002. They are explained in Chapter 2, Introduction to Seismic Analysis and Design.
4. Lateral load resisting system: The buildings are grouped based on the following types of lateral load resisting system.
    a. Load bearing un-reinforced masonry walls
    b. RC moment resisting frames and masonry infill walls
    c. RC shear walls
    d. Steel braced frames
    e. Steel moment resisting frames.

5. Geologic site hazards: The attributes are related to the geologic conditions that affect the performance of a building under seismic forces.
6. Foundations: The attributes related to the foundation are grouped in this category.
7. Non-structural components: The components of a building which are not designed to carry any vertical load from the members of the building or any lateral load are termed as non-structural components. With the advancement of building technology, the number and diversity of non-structural components are increasing. The dislodging and falling of non-structural components during an earthquake may cause injury or death and heavy damage to the building. Hence, in the recent years greater emphasis is laid on the proper placement and anchorage of the non-structural components. The evaluation statements are presented for several types of components that are present in existing buildings or are being observed in current Indian construction.

made similar to the provisions in the following documents: a) IS 800 (Draft), b) IS 1905: 1987, c) IS 13920: 1993, d) Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings.

Table 3.7 Building system

Evaluation Statements	C / NC / NA
*Load path***:** The building shall contain one complete load path for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the masses to the foundation.	
*Adjacent buildings***:** An adjacent building shall not be located next to the building being evaluated closer than 4 percent of the height of the shorter building.	
Mezzanines: Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure.	
*Deterioration of concrete***:** There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force-resisting elements.	
Masonry units: There shall be no visible deterioration of masonry units.	
Masonry joints: The mortar shall not be easily scrapped away from the joints by hand with a metal tool and there shall be no areas of eroded mortar.	
Cracks in infill walls: All existing diagonal cracks in the infilled walls that extends throughout a panel shall be less than 3 mm wide.	
Cracks in boundary columns: All existing diagonal cracks in the columns that encase masonry infills shall be less than 3 mm wide.	
Post-tensioning anchors (in buildings with post-tensioned members): There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used.	
Concrete wall cracks (in buildings with shear walls): All existing diagonal cracks in the walls shall be less than 3 mm wide, shall not be concentrated in one location and shall not form an X-pattern.	
Deterioration of steel (for steel frames): There shall be no visible rusting, corrosion, cracking or other deterioration in any of the steel elements or connections in the vertical- or lateral-load resisting systems.	

Table 3.8 Vertical irregularities

Evaluation Statements	C / NC / NA
No weak storey: The lateral strength of a storey shall not be less than 80 percent of that in the storey above.	
No soft storey: The lateral stiffness of a storey shall not be less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.	
No mass irregularity: There shall be no storey with seismic weight more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs.	
No vertical geometric irregularity: There shall be no storey with the horizontal dimension of the lateral-force-resisting system more than 150 percent of that in its adjacent storey.	
No vertical discontinuities: All vertical elements in the lateral-load-resisting system shall be continuous to the foundation.	

Table 3.9 Plan irregularities

Evaluation Statements	C / NC / NA
No Torsion irregularity: The distance between the storey centre of rigidity and the storey centre of mass shall be less than 30 percent of the building dimension at right angles to the direction of loading considered.	
No diaphragm discontinuity: The diaphragms shall not be composed of split-level floors and shall not have expansion joints.	

The plan irregularities of re-entrant corner, out-of-plane offset and non-parallel systems need not be checked under evaluation statements. The centre of mass and centre of rigidity can be approximately evaluated as follows. The rigorous calculation of these quantities is provided in Chapter 8, Structural Analysis for Seismic Retrofit.

Centre of Mass

The centre of mass (CM) can be approximately located based on the centroid of the floor above the storey. The floor is divided into component rectangles and a suitable reference axis is selected for this purpose. The coordinates of the CM (CM_x and CM_Y) are calculated by the following expressions.

$$CM_x = \frac{\sum m_j x_j}{\sum m_j} \qquad \textbf{(3.4a)}$$

$$CM_y = \frac{\sum m_j y_j}{\sum m_j} \qquad \textbf{(3.4b)}$$

Here,

m_j = mass of j^{th} floor segment
x_j = distance measured from Y axis to the centre of j^{th} segment
y_j = distance measured from X axis to the centre of j^{th} segment.

The summation is for all the floor segments.

Centre of Rigidity

The centre of rigidity (CR) for each storey can be calculated based on the moments of the stiffness of the columns or walls of the storey about the reference axes. The coordinates of the CR (CR_x and CR_Y) are calculated by the following expressions.

$$CR_x = \frac{\sum k_j x_j}{\sum k_j} \qquad \textbf{(3.5a)}$$

$$CR_y = \frac{\sum k_j y_j}{\sum k_j} \qquad \textbf{(3.5b)}$$

Here,

k_j = stiffness of j^{th} column or wall
x_j = distance measured from Y axis to the centre of j^{th} column or wall
y_j = distance measured from X axis to the centre of j^{th} column or wall.

Table 3.10 Load bearing un-reinforced masonry walls

<table>
<tr><th>Evaluation Statements</th><th>C / NC / NA</th></tr>
<tr><td colspan="2">Lateral load resisting system</td></tr>
<tr><td>Redundancy: The number of lines of walls in each principal direction shall be greater than or equal to 2.</td><td></td></tr>
<tr><td>Shear stress check: The shear stress in the walls calculated using the Strength-related Check procedure shall be less than 0.2 N/mm^2.</td><td></td></tr>
<tr><td>Proportions: The effective height-to-thickness ratio of the walls in a storey shall be less than the following.
<table>
<tr><td>Building with number of storeys</td><td>Walls with cement mortar</td><td>Walls with lime mortar</td></tr>
<tr><td>Not exceeding 2</td><td>27</td><td>20</td></tr>
<tr><td>Exceeding 2</td><td>27</td><td>13</td></tr>
</table>
</td><td></td></tr>
<tr><td>Masonry lay-up: Filled collar joints of multi-wythe masonry walls shall have negligible voids.</td><td></td></tr>
<tr><td colspan="2">Diaphragms</td></tr>
<tr><td>Diaphragm openings at walls: Diaphragm openings immediately adjacent to the walls shall be less than 25 percent of the wall length.</td><td></td></tr>
<tr><td>Diaphragm openings at exterior walls: Diaphragm openings immediately adjacent to exterior walls shall not be greater than 2400 mm.</td><td></td></tr>
<tr><td>Beam, girder and truss supports: Beams, girders and trusses supported by unreinforced masonry walls or pilasters shall have independent secondary columns for support of vertical loads.</td><td></td></tr>
<tr><td colspan="2">Connections</td></tr>
<tr><td>Wall anchorage: Exterior walls that are dependent on the diaphragm for lateral support shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels or straps that are developed into the diaphragm.</td><td></td></tr>
<tr><td>Transfer to walls: Diaphragms shall be connected for transfer of loads to the walls.</td><td></td></tr>
</table>

Table 3.11 RC moment resisting frames with masonry infill walls

Evaluation Statements	C / NC / NA
Lateral load resisting system	
Redundancy: The number of lines of moment frames in each principal direction shall be greater than or equal to 2. The number of bays of moment frames in each line shall be greater than or equal to 2.	
Shear stress check: The average shear stress in the columns calculated using the Strength-related Check procedure shall be less than $0.1\sqrt{f_{ck}}$. The characteristic strength of concrete is represented as f_{ck} (in N/mm^2).	
Axial stress check: The axial stress in the columns due to overturning forces calculated using the Strength-related Check procedure shall be less than 0.25 f_{ck}.	
Flat slab frames: The lateral-force-resisting system shall not be a frame consisting of columns and a flat slab/plate without beams.	
Prestressed frame elements: The lateral-load-resisting frames shall not include any prestressed or post-tensioned elements, where the average prestress exceeds the lesser of 4.8 N/mm^2 or 0.13 f_{ck} at potential hinge locations.	
Captive columns: There shall be no columns at a level with height/depth ratios less than 50 percent of the nominal height/depth ratio of the typical columns at that level.	
No shear failures: The ultimate shear capacity (V_{uR}) of a frame column shall be greater than the shear demand which occurs when the column attains the probable moment capacity (M_{pr}). i.e., $V_{uR} \geq 2M_{pr}/h$. Here, h is the height of the storey. Consider M_{pr} = 1.4 M_{uR}, where M_{uR} is the ultimate moment capacity (including material safety factor) in absence of axial load.	
Strong column – weak beam: The sum of the moment capacities of the columns shall be 10 percent greater than that of the beams at a beam-column joint.	
Column bar splices: All column bar splices shall be tension splices and provided only in the central half of the member length and hoops provided at spacing not exceeding 150 mm centre to centre.	
Column tie spacing: The columns shall have ties spaced at or less than $B/2$ throughout their length and at or less than $B/4$ or 100 mm at all potential plastic hinge locations. Here, B is the least lateral dimension of the column.	
Beam bars: At least two longitudinal top and two longitudinal bottom bars shall extend continuously throughout the length of each frame beam. At least 25 percent of the longitudinal bars provided at the joints for either positive or negative moment shall be continuous throughout the length of the members. The bars shall be properly anchored at an exterior joint.	

Beam bar splices: The lap splices for the longitudinal reinforcement shall not be located within 2d from the joint face and within L/4 from the location of potential plastic hinges. Here, d is the effective depth and L is the length of the beam.	
Stirrup spacing: All beams shall have stirrups spaced at or less than d/2 throughout their length. At potential hinge location, stirrups shall be spaced at or less than the minimum of 8d_b or d/4. Here, d_b is the diameter of the smallest longitudinal bar.	
Bent-up bars: Bent-up longitudinal steel shall not be used for shear reinforcement.	
Joint reinforcing: Column ties shall be extended at their typical spacing through the beam-column joints. For an interior joint, which has beams framing from all sides and the width of each beam is at least ¾ times the corresponding width of the column, the number of ties may be reduced to half.	
Deflection compatibility: Secondary components shall have the shear capacity to develop the flexural strength of the elements.	
Flat slabs: Flat slabs/plates not part of lateral-force-resisting system shall have continuous bottom bars through the beam-column joints.	
Wall connections: The masonry infill shall be in full contact with the bounding frame.	
Connections	
Column connection: All column reinforcement shall be dowelled into the foundation.	
Uplift at pile caps: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.	

Table 3.12 RC shear walls

Evaluation Statements	C / NC / NA
Lateral load resisting system	
Complete frames: Steel or concrete frames classified as secondary components shall form a complete vertical-load-carrying system.	
Redundancy: The number of lines of shear walls in each principal direction shall be greater than or equal to 2.	
Shear stress check: The shear stress in the shear walls calculated using the Strength-related Check procedure shall be less than 0.7 N/mm^2 or $0.15\sqrt{f_{ck}}$.	
Reinforcing steel: The area of reinforcing steel for concrete walls shall be greater than 0.0025 times the gross area of the wall along both the longitudinal and transverse axes and the maximum spacing of bars shall not exceed $l_w/5$, $3t_w$ and 450 mm. Here, l_w and t_w are the horizontal length and thickness of the wall, respectively.	
Deflection compatibility: Secondary components shall have the shear capacity to develop the flexural strength of the components.	
Flat slabs: Flat slabs/plates not part of lateral-force-resisting system shall have continuous bottom bars through the beam-column joints.	
Coupling beams: The stirrups shall be spaced at or less than 100 mm and shall be anchored into the core with 135° hooks.	
Diaphragms	
Diaphragm openings at shear walls: Diaphragm openings immediately adjacent to the shear walls shall be less than 25 percent of the wall length.	
Connections	
Wall connection: Wall reinforcement shall be dowelled into the foundation.	
Transfer to shear walls: Diaphragms shall be reinforced and connected for transfer of loads to the shear walls.	
Uplift at pile caps: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.	

Table 3.13 Steel braced frames

Evaluation Statements	C / NC / NA
Lateral load resisting system	
Redundancy: The number of lines of braced frames in each principal direction shall be greater than or equal to 2. The number of braced bays in each line shall be greater than 2.	
Axial stress check for columns: The axial stress in the columns due to overturning forces calculated using the Strength-related Check procedure shall be less than 0.3 f_y. The characteristic strength of steel is represented as f_y.	
Axial stress check for braces: The axial stress in the braces calculated using the Strength-related Check procedure shall be less than 0.5 f_y.	
Compact members: All frame elements shall meet the requirements of IS 800	
Slenderness of braces: All braces required to carry compression shall have the slenderness ratio less than 120.	
Connection strength: All the brace connections shall develop the yield capacity of the braces.	
K-bracing: The bracing system shall not include K-braced bays.	
Diaphragms	
Openings at braced frames: Diaphragm openings immediately adjacent to the braced frames shall extend less than 25 percent of the frame length.	
Connections	
Transfer to frames: Diaphragms shall be connected for transfer of loads to the frames.	
Column connection: The columns in lateral load resisting frames shall be anchored to the foundation.	
Uplift at pile caps: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.	

Table 3.14 Steel moment resisting frames

Evaluation Statements	C / NC / NA
Lateral load resisting system	
*Redundancy***:** The number of lines of moment frames in each principal direction shall be greater than or equal to 2. The number of bays of moment frames in each line shall be greater than or equal to 2.	
Interfering wall: All infill walls placed in moment frames shall be isolated from structural elements.	
Drift check: The drift ratio calculated using the Strength-related Check procedure shall be less than 0.025.	
Axial stress check: The axial stress in the columns due to overturning forces calculated using the Strength-related Check procedure shall be less than 0.3 f_y. The characteristic strength of steel is represented as f_y.	
Moment-resisting connections: All moment connections shall be able to develop the strength of the adjoining members or panel zones.	
Panel zones: All panel zones shall have the shear capacity to resist the shear demand required to develop 0.8 times the sum of the flexural strengths of the beams framing in at the face of the column.	
Column splices: All column splice details located in moment-resisting frames shall include connection of both flanges and the web.	
*Strong column / weak beam***:** The percentage of strong column / weak beam joints in each storey of each line of frames shall be greater than 50 percent.	
Compact members: All frame elements shall meet the requirements of IS 800	
Connections	
Transfer to frames: Diaphragms shall be connected for transfer of loads to the frames.	
Column connection: The columns in lateral load resisting frames shall be anchored to the foundation.	
Uplift at pile caps: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps.	

Table 3.15 Geologic site hazards

Evaluation Statements	C / NC / NA
Liquefaction: Liquefaction-susceptible, saturated, loose granular soils that could jeopardise the building's seismic performance shall not exist in the foundation soils at depths within 15 m under the building.	
Slope failure: The building site shall be sufficiently remote from potential earthquake induced slope failures or rockfalls to be unaffected by such failures or shall be capable of accommodating any predicted movements without failure.	
Surface fault rupture: Surface fault rupture and surface displacement at the building site is not anticipated.	

Table 3.16 Foundations

Evaluation Statements	C / NC / NA
Foundation performance: There shall be no evidence of excessive foundation movement such as settlement or heave that would affect the integrity or strength of the structure.	
Overturning: The ratio of the effective horizontal dimension, at the foundation level of the lateral load resisting system, to the building height (base/height) shall be greater than 0.6 S_a/g, where S_a is the spectral acceleration.	
Ties between foundation elements: The foundation shall have ties adequate to resist seismic forces where footings, piles, piers are not restrained by beams, slabs, or soils classified as Type I (as per IS 1893: 2002).	

Table 3.17 Non-structural components

Evaluation Statements	C / NC / NA
Partitions	
Unreinforced masonry: Unreinforced masonry partitions shall be braced at a spacing equal to or less than 3 m.	
Ceiling systems	
Support: The integrated suspended ceiling system shall not be used to laterally support the tops of gypsum board or masonry partitions.	
Integrated ceilings: Integrated suspended ceilings at exits and corridors or weighing more than 0.1 kN/m^2 shall be laterally restrained with a minimum of four diagonal wires or rigid members attached to the structure above at a spacing equal to or less than 3.5 m.	
Suspended lath and plaster: Ceilings consisting of suspended lath and plaster or gypsum board shall be attached to resist seismic forces for every 1 m^2 of area.	
Light fixtures	
Emergency lighting: Emergency lighting shall be anchored or braced to prevent falling during an earthquake.	
Independent support: Light fixtures in suspended grid ceilings shall be supported independently of the ceiling suspension system by a minimum of two wires at diagonally opposite corners of the fixtures.	
Cladding and glazing	
Cladding anchors: Cladding components weighing more than 0.5 kN/m^2 shall be mechanically anchored to the exterior wall framing at a spacing equal to or less than 1 m.	
Glazing: Glazing in curtain walls and individual panes over 1.5 m^2 in area located up to a height of 3 m above an exterior walking surface, shall have safety glazing. Such glazing located over 3 m above an exterior walking surface shall be laminated annealed or laminated heat-strengthened safety glass or other glazing system that will remain in the frame when glass is cracked.	
Deterioration: There shall be no evidence of deterioration, damage or corrosion in any of the connection elements.	
Cladding isolation: For moment frame buildings of steel or concrete, panel connections shall be detailed to accommodate a storey drift ratio of 0.02.	
Multi storey panels: For multi storey panels attached at each floor level, panel connections shall be detailed to accommodate a storey drift ratio of 0.02.	

Bearing connections: Where bearing connections are required, there shall be a minimum of two bearing connections for each wall panel.	
Inserts: Where inserts are used in concrete connections, the inserts shall be anchored to reinforcing steel or other positive anchorage.	
Parapets, cornices, ornamentations and appendages	
Un-reinforced masonry parapets: There shall be no laterally unsupported masonry parapets or cornices with height-to-thickness ratios greater than 1.5.	
Concrete parapets: Concrete parapets with height-to-thickness ratio greater than 2.5 shall have vertical reinforcement.	
Canopies: Canopies located at building exits shall be anchored to the structural framing at a spacing of 2 m or less.	
Appendages: Cornices, parapets, signs and other appendages that extend above the highest point of anchorage to the structure or cantilever from exterior wall faces and other exterior wall ornamentations shall be reinforced and anchored to the structural system at a spacing equal to or less than 3 m.	
Masonry chimneys	
Un-reinforced masonry chimneys: No un-reinforced masonry chimneys shall extend above the roof surface more than twice the least dimension of the chimney.	
Anchorage: Masonry chimneys shall be anchored at each floor level and the roof.	
Stairs	
Un-reinforced masonry walls: Walls around stair enclosures shall not consist of unbraced un-reinforced masonry with a height-to-thickness ration greater than 12-to-1.	
Stair details: In moment frame structures, the connection between the stairs and the structure shall not rely on shallow anchors in concrete. Alternatively the stair details shall be capable of accommodating the drift calculated using the Strength-related Check procedure without including tension in the anchors.	
Building contents and furnishing	
Tall narrow contents: Contents over 1.25 m in height with a height-to-depth or height-to-width ratio greater than 3-to-1 shall be anchored to the floor slab or the adjacent structural walls.	
Mechanical and electrical equipment	
Emergency power: Equipment used as a part of an emergency power system shall be mounted to maintain continued operation after an earthquake.	
Hazardous material equipment: Any equipment containing hazardous material	

shall not have damaged supply lines or unbraced isolation supports.	
Deterioration: There shall be no evidence of deterioration, damage, or corrosion in any of the anchorage or supports of mechanical or electrical equipments.	
Attached equipment: Equipment going over 9 kg that is attached to ceilings, walls or other supports 1.25 m above the floor shall be braced	
Vibration isolators: Equipment mounted on vibration isolators shall be quipped with restraints or snubbers.	
Piping	
Fire suppression piping: Fire suppression piping shall be anchored and braced.	
Flexible couplings: Fluid, gas and fire suppression piping shall have flexible couplings.	
Ducts	
Stair and smoke ducts: Stair pressurisation and smoke control ducts shall be braced and shall have flexible connection seismic joints.	
Hazardous material storage and distribution	
Toxic substances: Toxic and hazardous substances stored in breakable containers shall be restrained from falling by latched doors, shelf lips, wires or other methods.	

3.5 SUMMARY

The Rapid Visual Screening (RVS) is a form of statistical guideline to identify the buildings from a stock which are more vulnerable under seismic forces. This chapter explains the methodology of RVS for masonry, reinforced concrete (RC) and steel buildings.

A systematic data collection provides the relevant data for the evaluation of a building. Data sheets are provided for generic building information and information specific to masonry, RC and steel buildings.

The preliminary evaluation gives an insight about the attributes of the building related to its seismic performance. First, some configuration-related and strength-related checks are presented. Next a set of evaluation statements are provided for the building system, irregularities, different types of lateral load resisting systems, geologic site hazards, foundations and non-structural components.

3.6 REFERENCES

1. ASCE 31-03 (2003), “Seismic Evaluation of Existing Buildings”, American Society of Civil Engineers, USA.
2. FEMA 154 (2002), “Rapid Visual Screening of Buildings for Potential Seismic Hazards: A Handbook”, Applied Technology Council, USA.
3. FEMA 155 (2002), “Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation”, Applied Technology Council, USA.
4. FEMA 178 (1992), “NEHRP Handbook for the Seismic Evaluation of Existing Buildings”, Building Seismic Safety Council, USA.
5. FEMA 310 (1998), “Handbook for the Seismic Evaluation of Buildings − A Prestandard”, American Society of Civil Engineers, USA.
6. IS 456: 2000, “Plain and Reinforced Concrete – Code of Practice”, Bureau of Indian Standards.
7. IS 800 (Draft), “Indian Standard Code of Practice for General Construction in Steel”, Bureau of Indian Standards.
8. IS 875: 1987, “Code of Practice for Design Loads (other than earthquake) for Buildings and Structures”, Part 1: Dead Loads – Unit Weights of Building Materials and Stored Materials, Part 2: Imposed Loads, Part 3: Wind Loads, Bureau of Indian Standards.

9. IS 1893: 2002, "Criteria for Earthquake Resistant Design of Structures", Part 1: General Provisions and Buildings, Bureau of Indian Standards.
10. IS 1904: 1986, "Code of Practice for Design and Construction of Foundations in Soils: General Requirements", Bureau of Indian Standards.
11. IS 1905: 1987, "Code of Practice for Structural Use of Unreinforced Masonry", Bureau of Indian Standards.
12. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings – Code of Practice", Bureau of Indian Standards.
13. IS 13920: 1993, "Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces", Bureau of Indian Standards.
14. IS 13935 (Draft version, 2005), Seismic Evaluation, Repair and Strengthening of Masonry Buildings – Guidelines, Bureau of Indian Standards.
15. Model Town and Country Planning Legislation, Zoning Regulations, Development Control, Building Regulation/Byelaws for Natural Hazards Zones of India (Draft version, 2004), Ministry of Home Affairs, Government of India.
16. Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings (Draft version, 2006), Bureau of Indian Standards.
17. Arya, A. S. (2003), "Rapid Visual Screening of Buildings in Various Seismic Zones in India", Submitted under National Disaster Risk Management Programme, an initiative by Government of India and United Nations Development Programme.

4

CONDITION ASSESSMENT OF BUILDINGS

4.1 OVERVIEW

This chapter describes the process of determining the suitability of an existing building for further use with the intended purpose. This chapter follows closely the chapter on Condition Survey and Non-Destructive Evaluation of a previous publication (Handbook on Repair and Rehabilitation of RCC Buildings, 2002).

First, the basic properties and causes of deterioration of masonry, concrete, steel and timber buildings are briefly discussed. Next, the technique of visual inspection is explained. An initial visual inspection of the structure can reveal useful information about areas that need closer look. The visual inspection helps to plan a strategy to investigate the structure further, using more detailed techniques.

A number of tests are available to study the condition of the material in a structure. The tests are grouped as follows.

a) Non-destructive tests: The material is tested without causing any damage to the structure.

b) Intrusive tests: A sample of the material is removed for evaluation. Adequate repair is necessary to compensate for the removal of the sample.

c) Destructive load tests: The structure is tested to failure to gather information for similar type of buildings.

The non-destructive tests range in sophistication from simple ones such as striking the surface of the material with a hammer, to techniques where ultrasonic signals travelling through the material are analyzed by instruments. The tests are briefly explained, with the reference of the corresponding standards for detailed information. Under the intrusive tests, testing of cores from masonry or concrete buildings and testing of steel coupons are covered. The destructive load tests are project specific and hence, are outside the scope of this chapter.

4.2 INTRODUCTION

> Condition assessment describes the process of assessing the actual condition of a structure in relation to the requirement. The assessment indicates whether the structure is satisfactory, or, whether repair and rehabilitation are necessary.

The actual performance or condition of a building evolves with time as depicted in Figure 4.1. The condition of any building typically deteriorates with age. Moreover if there is a design defect, then the performance is expected to fall below the requirement of previous standards. Any rehabilitation in the past may have boosted the performance. But with the implementation of new codes, the required performance is even higher. Condition assessment is necessary before retrofit is undertaken to assess the actual condition of the structure in relation to the current requirement. After an appropriate retrofit scheme is implemented, the structural performance improves. The performance is expected to meet the requirement throughout the remaining life of the building.

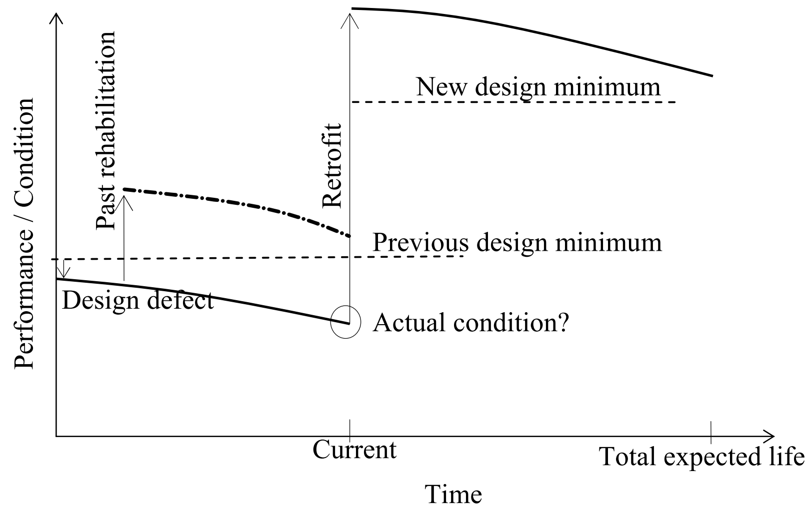


Figure 4.1 Evolution of actual performance or condition of a building

Condition assessment includes the following steps:

1. Initial inspection and appraisal.
2. Review of documents.
3. Detailed investigation.
4. Reporting and recommendations.

The steps are briefly discussed below.

<u>Initial inspection and appraisal</u>

A visual inspection helps to plan a strategy to investigate the structure further, using more sophisticated techniques.

<u>Review of documents</u>

The collection of data and documents is explained in the chapter on Rapid Visual Screening, Data Collection and Preliminary Evaluation. In case the records are not available, an attempt can be made to retrieve some information based on interviews with those who were involved in the design and construction of the building or familiar with contemporary methods and the owners/residents.

Detailed investigation

This includes the following:

1. Obtaining the properties of the structural materials used in the building.
2. Determining the type and disposition of reinforcement in reinforced concrete members.
3. Locating deteriorated material and other defects, and identifying their causes.

As there are many causes for the deterioration of structures, it may be difficult to identify precisely the cause that has led to the deterioration. Moreover, several types of damage, whether they are load related, environment related, or earthquake related, lead to similar signs of deterioration, such as cracking, scaling, de-lamination, discolouration, etc. The areas of deterioration should be investigated closely.

Reporting and recommendations

After completion of detailed investigation, a report should be prepared and suitable recommendations should be given. Recommendations include whether to go for repair or rehabilitation of the structural members.

Once the condition assessment is complete, the building can be analysed. The methods of analysis are covered in the chapter on Structural Analysis for Seismic Retrofit.

4.3 PROPERTIES OF MATERIALS IN EXISTING BUILDINGS

In this section, the basic properties apart from the member sizes, that are required in the analysis and assessment of a building, are discussed.

4.3.1 Masonry Buildings

The unit weight, crushing strength and the elastic modulus of the masonry units and the type of mortar are required in the structural analysis of a masonry building. In absence of records, since the types of bricks or blocks vary widely, historical information can only be of limited use. Core specimens can be extracted from the building and tested for the properties. Non-destructive techniques can also be used to complement the data obtained from the intrusive tests.

4.3.2 Concrete Buildings

The unit weight, compressive strength and the elastic modulus of concrete and the yield stress of the reinforcing bars (rebar) are required in the structural analysis of a reinforced concrete building. In absence of records, the period of the construction can give an idea of the type of concrete and reinforcement that have been used in the building. Non-destructive tests can be used to estimate the compressive strength of the concrete. In addition, cores can be extracted to determine the actual strength and the elastic modulus. For rebar, a minimum of three samples can be extracted for testing.

4.3.3 Steel Buildings

The yield stress and ultimate tensile strength of the steel members and the type of connections are required in the analysis of a steel building. In absence of records, the period of the construction can give an idea of the type of steel that have been used in the building. Samples can be removed from the structure under unloaded condition and tested in the laboratory. If a connector, such as a bolt or rivet, is removed, a comparable connector should be reinstalled. Any destructive testing of a welded connection should be repaired immediately.

4.3.4 Timber Buildings

The type of wood needs to be identified for estimating the unit weight, elastic modulus and the allowable stresses under bending, shear, tension and compression. In absence of records, the type of wood can often be identified by visual inspection. Thereafter, the corresponding tabulated properties can be used. Care should be taken to check the condition of the wood and the moisture content.

4.4 DETERIORATION OF MATERIALS IN EXISTING BUILDINGS

> The most important step for proper condition assessment of a building is the identification of any existing damage and the possible causes of the damage.

In this section, the primary causes of deterioration of the different types of buildings are discussed.

4.4.1 Masonry Buildings

In masonry buildings, the mortar joint is a weak link, and most of the failures take place through these joints. In general, the mortar shall not be easily scraped away with a metal tool or eroded. Sometimes, sudden and more disastrous failure can occur by cracking through the masonry units, as shown in Figure 4.2. Masonry buildings are also damaged by spalling, staining, moisture ingress, differential settlement of foundation, shrinkage (due to drying) or expansion / contraction (due to temperature variation). These are further described in the chapter on Retrofit of Masonry Buildings.

Figure 4.2 Vertical cracks in a brick masonry building

4.4.2 Concrete Buildings

Deterioration of concrete is a complex phenomenon because more than one mechanism occurs simultaneously. Hence, it is necessary to have an understanding of the basic underlying causes of damage in concrete and their manifestation. Table 4.1 presents the common distress mechanisms in concrete.

Table 4.1 Causes of deterioration of concrete

	Symptoms	Causes
1	Rust staining, cracks run in straight parallel lines at uniform intervals as per the reinforcement position, spalling of concrete cover.	**Corrosion of reinforcing bars:** Exposure to normal atmospheric conditions, cyclic wetting and drying.
2	Cracks mostly on the surface (shallow in depth, 20-50 mm), parallel to each other, 1 to 2 m apart, vary in length from 50mm - 3m.	**Plastic shrinkage:** Caused by surface tension forces, environmental effects of temperature (concrete and ambient), high wind velocity and low relative humidity.
3	Fine cracks, shallow in depth and absence of any indication of movement, typically orthogonal or blocky.	**Drying shrinkage:** The restraint to shrinkage causes tensile stresses that lead to cracking. For example in a footing on a rough base, or in new concrete placed over existing concrete.
4	Cracks are regularly spaced, extend to full depth and perpendicular to the larger dimension of concrete.	**Thermal effects:** Induced by exothermal reaction in mass concrete. If the change in volume is restrained during cooling, tensile stresses can cause cracking.
5	Spalling and scaling of the surface, exposing the aggregates which are not cracked; cracks parallel to the surface and gaps around the aggregates.	**Freeze-thaw deterioration:** Alternate cycles of freezing and thawing, use of de-icing chemicals.
6	Surface dissolution of cement paste exposing the aggregates.	**Acid attack:** Acid in smoke, exhaust gases or rain.
7	Rough surface, presence of sand grains (resembles a coarse sand paper).	**Aggressive water attack:** Causes serious effects in hydraulic and marine structures, results in washing away of aggregate particles because of leaching of cement paste
8	Map or pattern cracking, general appearance of swelling of concrete.	**Alkali-carbonate reaction:** Chemical reactions between alkali in cement with certain dolomitic aggregates, expansion due to de-dolomitisation and subsequent crystallization of brucite.
9	Map or pattern cracking, expands freely, silica gel leaches from cracks, paste depleted of calcium hydroxide.	**Alkali-silica reaction:** Chemical reaction between alkali ions (Na^+ and K^+) in cement with silica in aggregates.

10	Map and pattern cracking, general disintegration of concrete.	**Sulphate attack:** Formation of gypsum, thaumasite and ettringite which have higher volumes than the reactants.
11	Single or multiple long diagonal cracks (usually larger than 3mm in width), accompanying misalignment and displacements.	**Structural damage:** Induced by improper construction and maintenance throughout the life of the structure.
12	Honey-comb, bug holes (small holes less than about 3mm in diameter), cracking in concrete leading to cold joints.	**Construction errors:** Improper mix design, consolidation, curing etc., inadequate expansion joints, incorrect position of reinforcement.
13	Surface is generally smooth with localized depressions, long shallow grooves, spalling along monolith joints (abrasion). Severely pitted and extremely rough surface (cavitation)	**Erosion:** Rolling and grinding of debris (abrasion), sub-atmospheric pressure, turbulent flow and impact energy (cavitation).
14	Cracking or spalling of concrete, complete deterioration of the structure.	**Design errors:** Abrupt changes in capacity, insufficient reinforcement, inadequate provision for deflection and drainage.
15	Crushing of concrete, complete collapse of structure.	**Accidental loading:** Generates stresses higher than strength of concrete, resulting in localized or complete failure of the structure.

The dominant cause for deterioration of concrete is the corrosion of steel reinforcing bars. Figure 4.3 presents a schematic depiction of the process of corrosion of reinforcing steel. Figures 4.4 to 4.6 depict structural members that have suffered varying degrees of deterioration due to corrosion.

In the highly alkaline environment of concrete, the reinforcing steel is protected from corrosion by a thin oxide film on its surface. Due to several reasons, the film may get disrupted, initiating the process of corrosion. The product of the reactions is iron oxide, or ‘rust’, which is six to seven times the original volume of steel. This leads to an expansive stress on the surrounding concrete, causing it to crack, and subsequently delaminate (the cover peels off as a layer). The moisture can carry some of the rust in solution and show up on the concrete surface as a dark coloured stain.

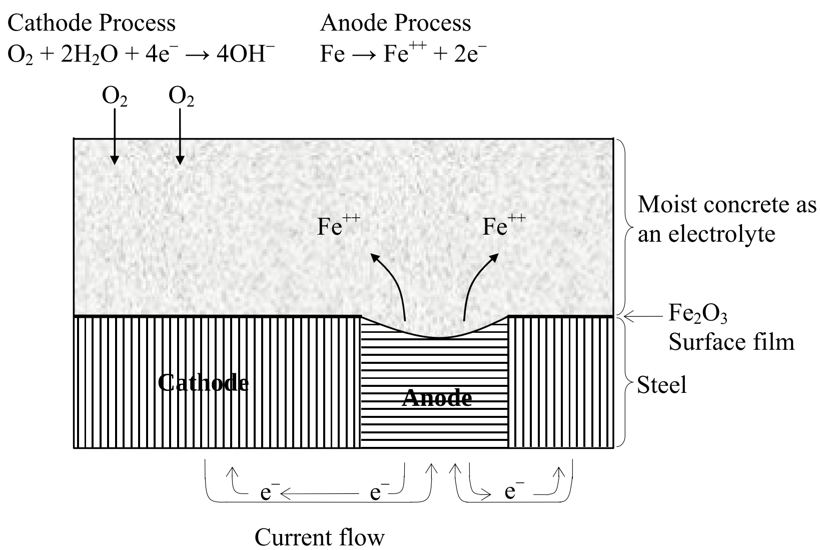


Figure 4.3 Corrosion of reinforcing steel in concrete

Figure 4.4 Cracking in a beam from corrosion of steel due to water seepage

Figure 4.5 Severe corrosion damage to underside of a beam

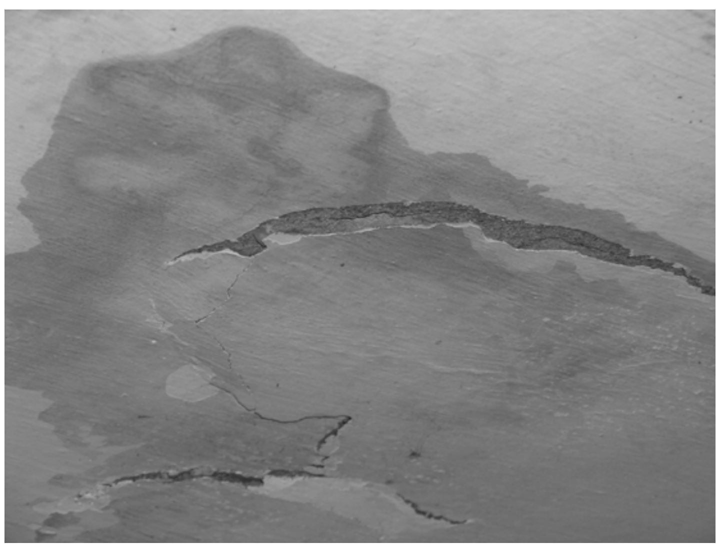


Figure 4.6 Delamination in a slab from corrosion of steel due to water seepage

4.4.3 Steel Buildings

Compared to concrete, steel being a more homogeneous material is relatively easy to understand. Problems in steel buildings mainly arise due to corrosion, failure of joints and unexpected loading (impact, fire, etc.). Corrosion damage to steel results in the failure of the cladding or cover, and a severe loss in the integrity of the structure. Welds, rivets and other joints in steel construction are beset with their own corrosion problems. The joints can fail due to other reasons such as stress concentration, pre-existing damage etc. The earthquakes in Northridge, USA (in 1994) and in Kobe, Japan (in 1995) exposed the vulnerability of welds in the beam-column joints of steel moment-frame buildings. Therefore, the welds and the conditions of the joints should be checked during condition assessment. The following photos show some typical deterioration of steel members.

Figure 4. 7 Pitting corrosion of steel

Figure 4. 8 Stress corrosion of bolts

Figure 4.9 Crevice corrosion between concrete and steel

4.4.4 Timber Buildings

Presence of defects in wood can lead to mechanically induced damage. The following schematic diagram depicts typical defects in wooden members. Apart from mechanically induced damage, wood can be severely affected by weathering, fungus, insects and marine borers, if not properly seasoned and protected.

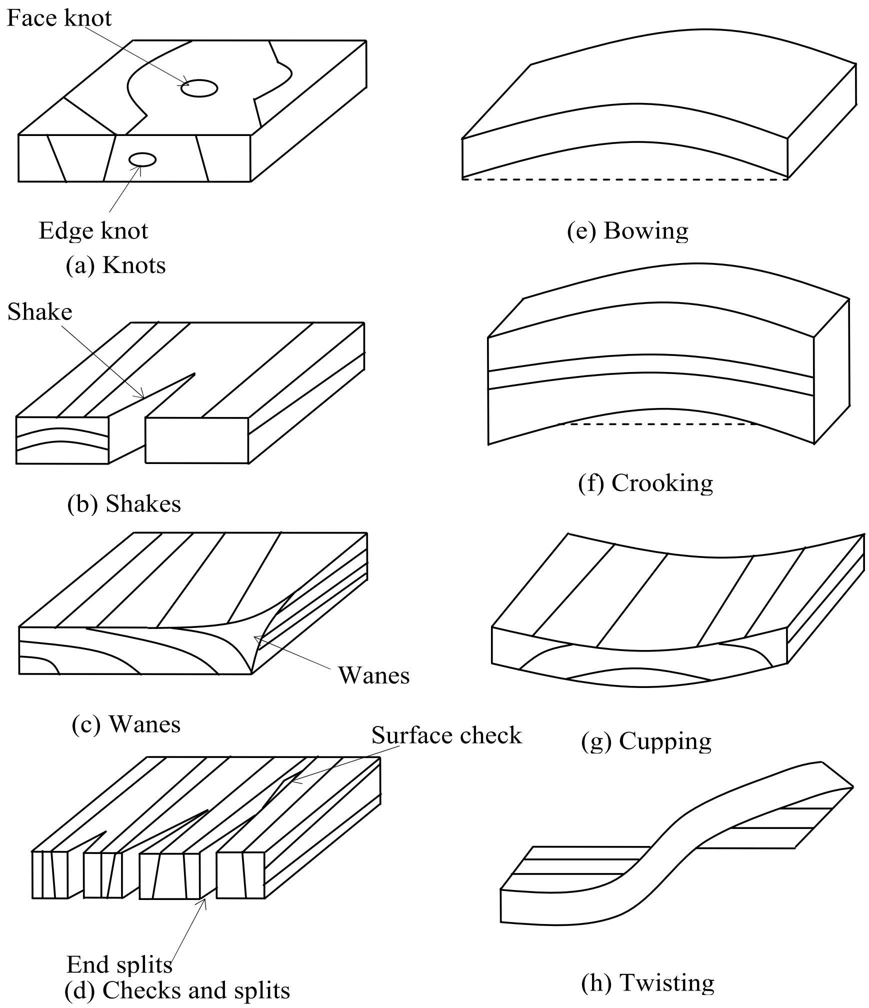


Figure 4.10 Defects in wood

4.5 VISUAL INSPECTION

> Visual inspection is the initial non-destructive test. It forms the basis of the subsequent inspections. A detailed visual inspection makes it possible to narrow down the critical areas in a structure that need further investigation using sophisticated techniques. The trained eye of an inspector can often reveal information that is sometimes difficult to pick up using hi-tech instruments.

4.5.1 Accessories

Visual inspections may be performed directly or indirectly (when photographs, radiographs or videos of the damaged areas are analyzed at a later stage). The direct inspection can be aided by a number of tools:

- Magnifying lenses
- Binoculars / telescopes
- Boroscopes
- Camera
- Ruler, measuring tape, hand held microscope for measuring crack width
- Light hammer, chipping / scraping tools
- Flash light
- Compass
- Markers
- Pad and clip board.

4.5.2 Items of Inspection

The items of inspection are listed based on the type of buildings.

<u>Masonry</u>

- Quality of bricks / masonry units, mortar
- Cracking and differential movement

Concrete

- Pattern, location, and orientation of cracks
- Scaling, spalling, staining, disintegration of the surface, honey-combing
- Exposed reinforcement and corrosion

Steel

- Corrosion
- Stress concentration (evident from cracks in the paint)
- Crippling or buckling of webs or flanges
- Bowing, misalignment, deformation, twisting
- Cracks in welds or missing welds
- Missing bolts / rivets

Timber

- Defects in wood
- Insect damage
- Decay
- Cracking

Other general defects that can be identified by visual inspection are as follows.

- Water seepage as evident from surface dampness, or leakage through joints or cracks.
- Movements as evident from deflection, heaving or differential settlement.
- Miscellaneous: Blistering membranes and coatings, pounding of water, discolouration.

4.5.3 Limitations of Visual Inspection

- Can only detect surface defects; a clean surface is usually necessary.
- Low reliability.
- Good lighting is necessary.
- Subjective; quality will vary with the inspector.

4.6 DETAILED INVESTIGATION

Visual inspection can reveal the areas in the building that require further investigation. For example, areas in concrete where rust stains are observed need to be checked for the extent of corrosion, in order to assess the residual strength. In steel structures also, the extent of corrosion can be checked using appropriate techniques.

The overall plan for detailed investigation may be drawn up based on the total area of the building, as well as on the extent of damage. For example, a building with occasional complaints of water seepage would demand lesser priority compared to a building that has just been gutted by fire or damaged by an earthquake. The plan should first cover those areas that are structurally vulnerable and are liable to compromise the integrity of the building. Next, the suitable type of tests should be identified.

> There are three types of tests: (a) non-destructive, (b) intrusive and c) destructive load tests. Non-destructive tests do not cause any damage to the material or structure upon completion of the test. Intrusive tests cause minor damage which can be repaired. In a destructive load test, the structure is tested to failure to gather information for similar type of buildings.

Before chalking out the plan, an inventory of the equipment available for testing, whether non-destructive or intrusive, should be prepared. The type of test to be carried out and the extent of the investigation (that is, whether only specific areas are to be selected or the entire structure needs investigation) should be properly detailed. The limitations and scope of the tests must be well understood. The following sections briefly explain the non-destructive and intrusive tests. Further information is available from the corresponding referred standards.

4.7 NON-DESTRUCTIVE TESTS

The following sub-sections describe the commonly used non-destructive tests.

4.7.1 Basic Tests

First, some basic tests are mentioned, which are to follow visual inspection.

Key test

Scraping by a key or a sharp knife will enable to identify porous portion of mortar at the bed joints of a masonry wall. The weak portions of mortar can be identified and suitable further investigation at these locations can be carried out.

Spray test

Spraying the surface of a concrete or masonry member with water will show up cracks because they are more visible when wet.

Push test

A weak portion of a wall can be identified by pushing it. The weak portion will be revealed due to its flexibility. This test is suitable for thin walls.

4.7.2 Rebound Hammer and Penetration Techniques

- Rebound hammers measure the elastic rebound from the surface of concrete. The rebound value indicated by the hammer is related empirically to the compressive strength of concrete. Rebound hammers are able to provide a quick estimate of the quality of concrete. The common type of rebound hammers are the Schmidt hammers (Figure 4.11). They are available in two varieties: regular and pendulum-type. The pendulum type hammer is applicable to lower strength concrete, such as lightweight concrete, and to weak masonry blocks. Adequate care must be taken for preparation of the surface. Figure 4.12 depicts the use of a Schmidt rebound hammer to assess the quality of concrete in a slab. The manufacturer typically specifies the approximate correlation between the compressive strength of the concrete and the rebound number obtained from the tests. The procedure of the test is given in IS 13311: 1992, Part 2.
- Penetration techniques, such as the Windsor probe method, work on the principle of resistance to penetration of a probe that is shot into the concrete with a definite amount of energy. The depth of penetration of the hardened steel alloy probe is empirically related to the compressive strength of the concrete. The procedure of the test is given in ASTM C803-03.

Both the tests described above are able to assess only the surface condition of concrete. Hence, an estimation of the compressive strength of the interior concrete may not be possible by these tests. Also, these tests may not give reliable data near edges and corners. However, a quick indication of the damaged areas can be obtained. Good calibration and training is necessary to produce reliable results from these tests.

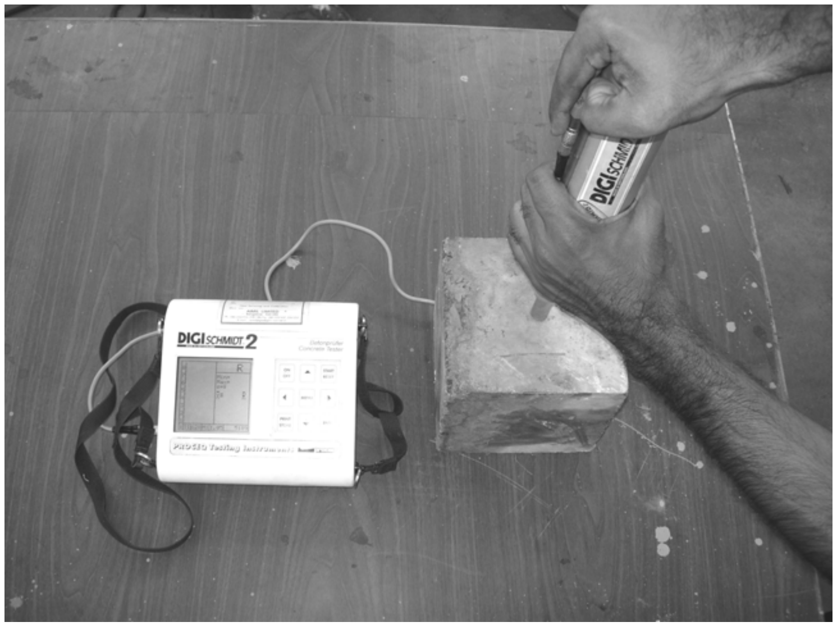


Figure 4.11 Equipment for rebound hammer test

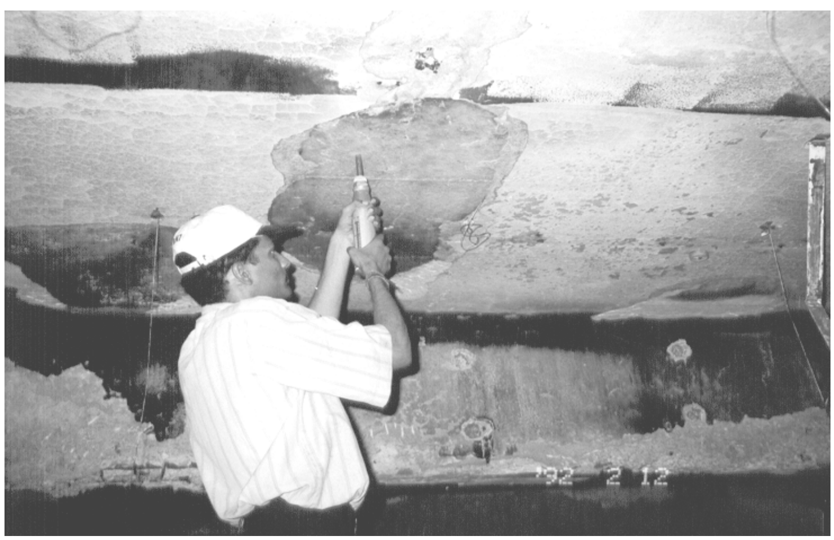


Figure 4.12 Use of Schmidt rebound hammer in a slab of building damaged by fire

4.7.3 Thermal Methods

The detection of heat flow through a body can indicate the presence of flaws or defects (ASTM C1046-95). In the infrared thermography technique, heat is passed through the material, and an infrared detector detects the heat patterns emanating from the body. As shown in the following schematic diagram, when a defect is present in the body, it would show up as a hot spot when heat is flowing inward, and as a cold spot when the heat is flowing outward.

The limitations of this technique are that its accuracy is somewhat limited to the near-surface areas, and the application necessitates the presence of clement weather. Also, the surface conditions might have a bearing on the result.

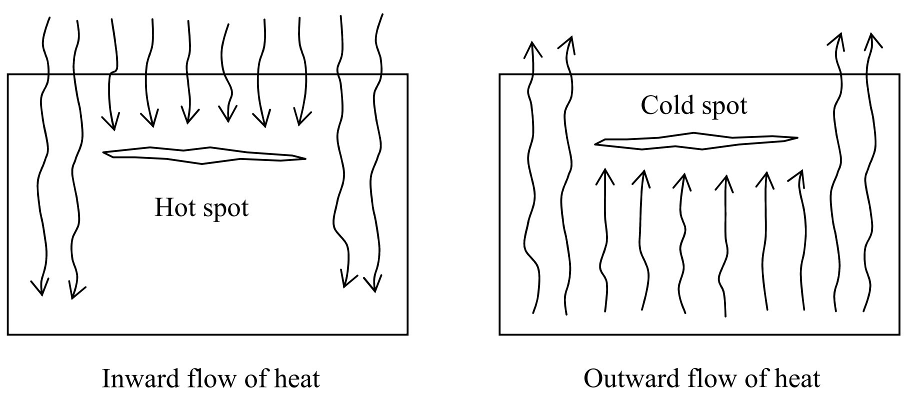


Figure 4.13 Heat flow through damaged concrete

4.7.4 Radiography

In radiography, X-rays or neutrons are passed through the structural element, and the resultant image is captured on a film. This film is then studied to find the location of defects (ASTM E748-02). The transmittance of X-rays or neutrons depends on the density of the material. Defective areas will show a larger transmittance. Radiography can be used to obtain a 360° image reconstruction, with techniques such as computerized axial tomography scan. Internal flaws are easy to detect using radiography.

One major limitation of using radiography is the hazard associated with the testing. They can be used effectively only if the source can be placed out of contact with the operator. Another limitation is the high costs of the equipment.

4.7.5 Electromagnetic Techniques

In concrete, electromagnetic techniques are typically used to detect the depth of rebar (in other words, concrete cover over rebar). As shown in the following figure, this can be done by magnetic reluctance or electromagnetic current (eddy current) (ASTM E 1004-02).

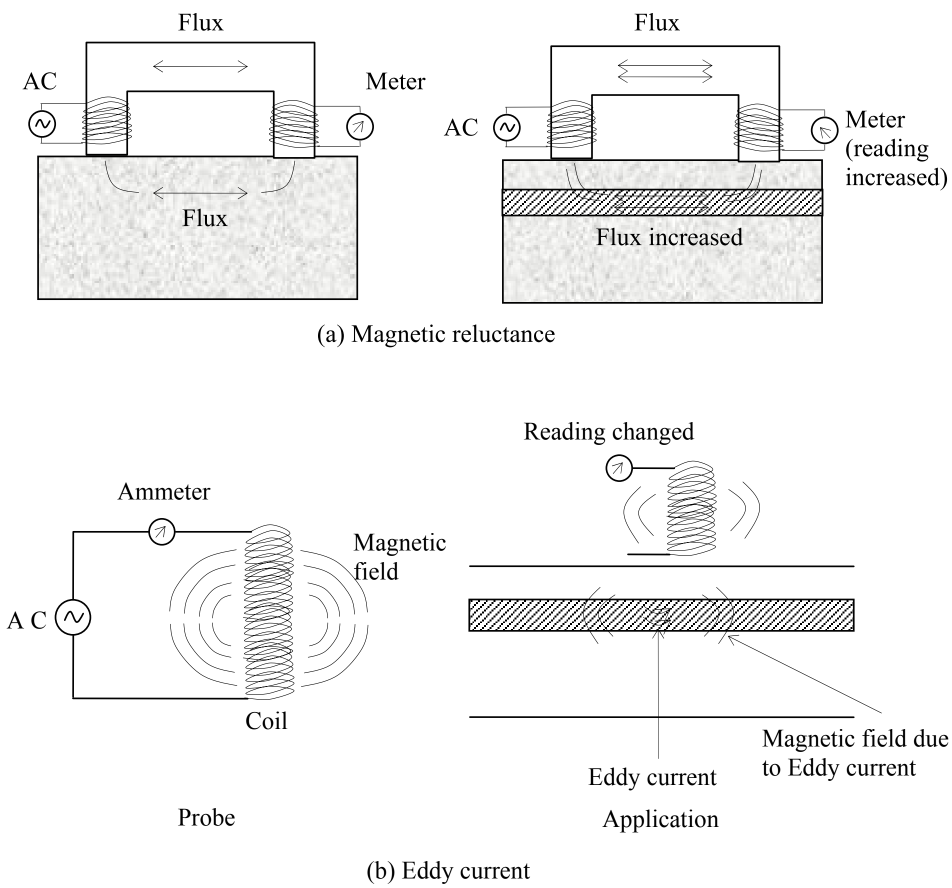


Figure 4.14 Determination of concrete cover using electromagnetic techniques

In the magnetic reluctance method, the presence of rebar increases the electromagnetic flux in the U-magnet and this is detected by the meter. In the Eddy current technique, the magnetic field in a coil (which is a part of the Eddy current probe) induces Eddy currents in the rebar. This Eddy current generates a magnetic field of its own that interferes with the main magnetic field. The change in inductance of the coil is then measured using the meter. Such probes are commonly used for rebar locating devices such as the Pachometer. A schematic diagram of the use of a Pachometer is shown in the following figure.

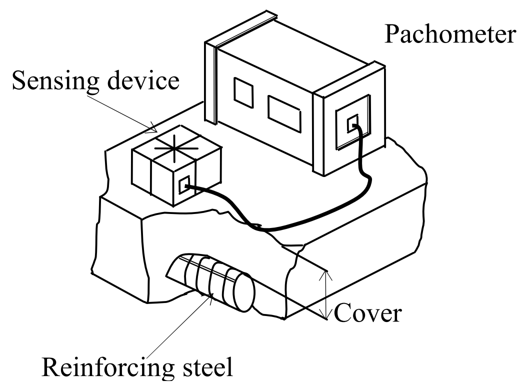


Figure 4.15 Use of a Pachometer for detecting rebar in concrete

Another electromagnetic technique called the magnetic particle method can also be used for conductive metallic elements. In the technique, a magnetic field is applied using a coil to the test piece. Magnetic particles (such as iron filings) are then sprinkled upon the test piece. These particles line up along surface cracks that are perpendicular to the orientation of the applied magnetic field. This makes visual detection of the surface cracks possible. One obvious limitation of these techniques is that they are limited to surface defects.

The RADAR technique makes use of the change in the speed of electromagnetic waves travelling through a medium based upon the dielectric properties of the medium. This is mostly used in bridge decks and pavements.

4.7.6 Stress Wave Propagation Methods

Stress wave propagation methods constitute the most reliable techniques for qualification of damage in concrete structures. These methods involve the propagation of stress waves through concrete. The stress waves are created either by pulses generated using ultrasonic transducers, or by impact.

The different techniques of generating the stress waves are described next.

(i) Sounding

A qualitative evaluation of concrete can be easily obtained by just sounding it (that is, tapping it) with a hammer. When the hammer is struck on good concrete or masonry, a ringing sound is created. However, on areas where delaminations or cracks occur, the striking of the hammer produces a drum-like sound. The limitation of this method is that it cannot detect defects that exist deep in the member. Also, defects lying under overlays are also difficult to detect.

In the case of steel members, the joints can be tested using a hammer. Tapping with a hammer can give information on the tightness of the rivets and bolts, and also whether there is any movement. Excessive movement under repeated loading may cause fatigue damage.

Chain drag is another way of finding out delaminated parts and voids. Compared to sounding with a hammer, chain drag can cover more area in a given time and has the potential to be mechanized. In this method, the operator passes a heavy chain on the surface of the concrete. The quality of sound generated is picked up using microphones and characterized. The following figure shows a schematic diagram of the use of sounding and chain drag methods. The areas identified as 'defective' in the sounding technique could be marked using paint for further investigation.

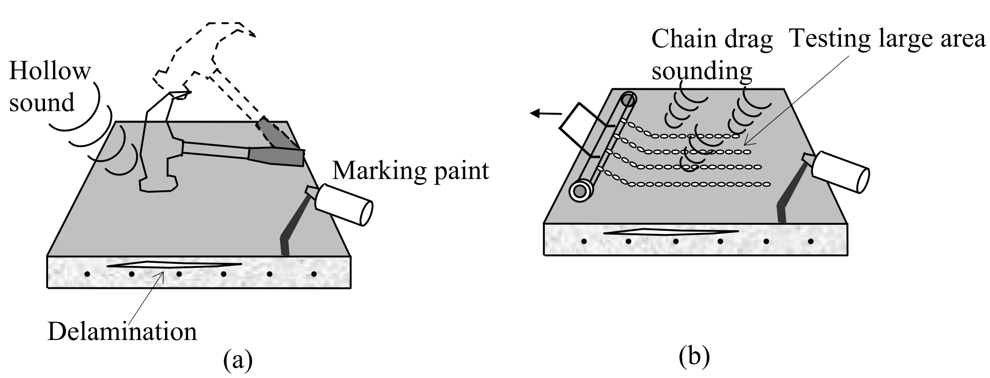


Figure 4.16 Use of (a) hammer and (b) chain drag

(ii) Ultrasonic pulse velocity (UPV)

The procedure of this test is given in IS 13311: 1992, Part 1. In the UPV method, the velocity of a pulse travelling through concrete is measured and correlated to its stiffness. The velocity of the pulse increases with the stiffness of the concrete, but decreases with increasing density. As shown in the following figure, the test can be conducted in three transmission modes: direct, semi-direct, and indirect. The direct mode, or the through-transmission mode, is the most reliable, but needs access to both sides of the member. Adequate calibration is necessary to use the semi-direct and the indirect modes. Although an overall assessment of the quality of concrete can be obtained, it may be difficult to point out the exact location of a defect by the UPV test.

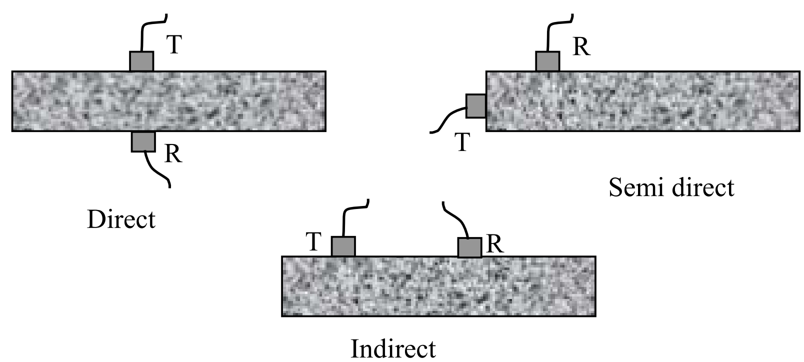


Figure 4.17 Various transmission modes for ultrasonic pulse velocity test (T: Transducer, R: Receiver)

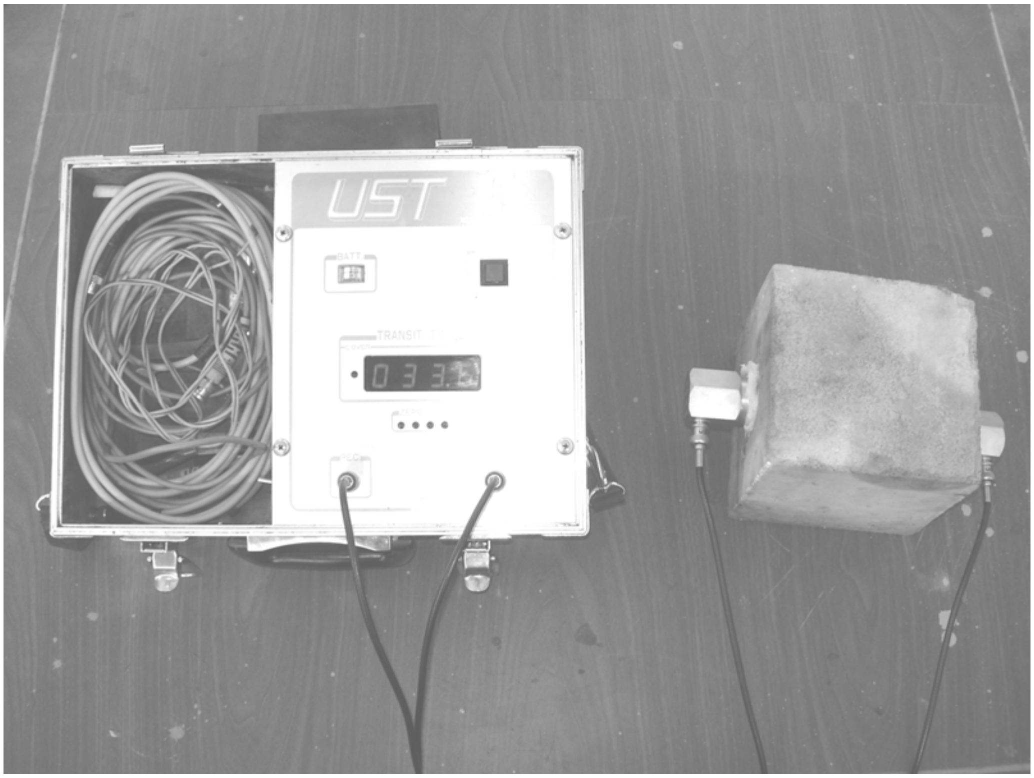


Figure 4.18 Equipment for ultrasonic pulse velocity test

IS 13311: 1992 (Part I) gives a guideline for the analysis of data from UPV test. This guideline is presented in the following table.

Table 4.2 Velocity criterion for grading of concrete quality (IS 13311: 1992, Part I)

S. No.	Pulse velocity obtained in direct transmission mode (km/sec)	Condition of concrete
1	> 4.5	Excellent
2	3.5 – 4.5	Good
3	3.0 – 3.5	Medium
4	< 3.0	Doubtful*

* Either quality is poor or more tests necessary

(iii) Impact Echo / Pulse Echo Method

In the impact echo method, the impacting device such as a hammer is struck on the concrete surface. The sound waves that reflect off defects or other features are picked up by a receiving transducer, and conveyed to a signal processor. The waveform is analyzed at the signal processor. From this analysis, the amplitude and travel time of the waves can be evaluated. A schematic diagram of the system is shown in Figure 4.19. Also shown in the same figure is a diagram of the pulse-echo system. In this system, the pulses are generated by a pulsing transducer (IS 3664: 1981). The same transducer can then act as a receiver, or an alternate receiver may be provided. The signal is again relayed to a signal processor.

Some limitations of this technique are that defects lying under other defects are not easy to detect. Also, reflections from sides, edges, and corners can confuse the data. A sufficient difference in acoustic impedance between the two media is necessary to obtain good information.

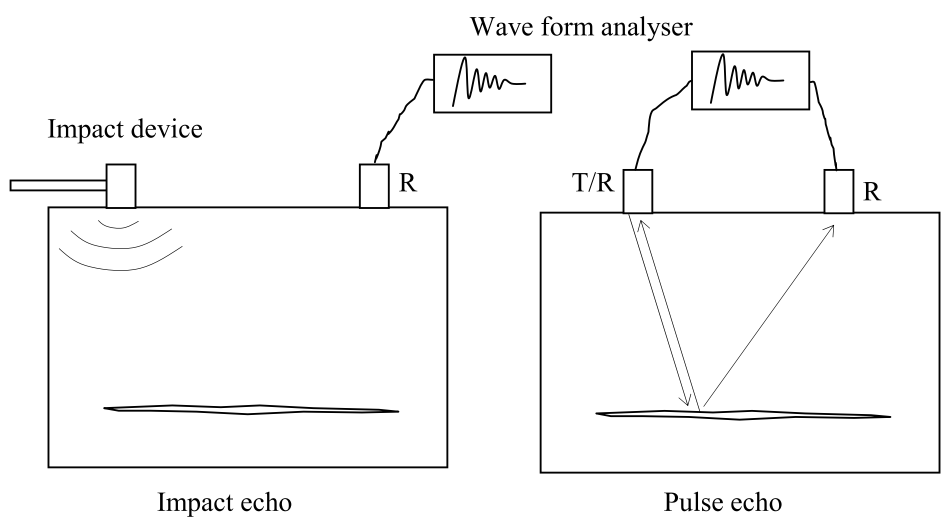


Figure 4.19 Schematic diagrams of the impact-echo and pulse-echo methods (T: transducer, R: receiver)

Measurement of corrosion of reinforcing bars

A simple method to measure the likelihood of corrosion in a reinforcing steel bar inside concrete is the half cell potential test. In this method, the electrostatic potential of the bar is measured with respect to a reference electrode. Depending on the potential values, the likelihood of corrosion of the bar is judged. After the likelihood of corrosion has been detected by the half-cell potential test (ASTM C876-91), more sophisticated techniques such as potentio-dynamic polarization or electrical impedance spectroscopy can be used to get a measure of the actual corrosion currents in the system. These tests can suggest the extent of corrosion present in the structure, and then an appropriate remedial mechanism can be formulated.

4.8 INTRUSIVE TESTS

The necessity for intrusive tests should be brought out by visual inspection. In general, these tests may be needed when non-destructive tests are limited in their accuracy. For example, when the structural element is covered by cladding or protective material, some intrusive exploratory work is required. Intrusive tests cause some damage to the structural member, that needs to be repaired.

4.8.1 Core Tests

In order to ascertain the compressive strength of concrete or masonry, a cylindrical core is usually removed from the structural member. Proper care should be taken to ensure that the core removal process does not induce any extra damage to the member. IS 1199: 1959 gives the procedure of obtaining cores from hardened concrete. The core holes should be filled with concrete or grout immediately after the drilling. Thus, core sampling needs the use of good equipment and skilled manpower. Cores can be both along the horizontal and vertical directions. Before removing the core, the location of reinforcing bars inside the concrete should be clearly mapped. If the removed core consists of bars, then the results are affected. In any case, the relevant standards (ASTM C42-04) and recommendations should be followed closely, with regard to the interpretation of test results. Correction factors should be used for the presence of bars inside the core and for different height-to-diameter ratios of the core.

A concrete core can be tested as per the procedure of IS 516: 1959. Compared to non-destructive tests for determination of the in-place compressive strength, core tests yield a much more reliable result. In general, cores removed from a concrete structure give lower strength compared to the cylinder specimens cast along with the structure and cured under laboratory conditions. According to American Concrete Institute, if at least 3 cores are removed from a representative part of concrete and none of them shows strength less than 75% of the characteristic strength (also, average not less than 85% of characteristic strength), then the concrete is in sound condition.

For masonry buildings, samples taken along the horizontal and vertical directions tend to give different compressive strengths. Also, the elastic modulus measured along a masonry unit and across a mortar joint may be different. The following figure shows the set up for testing a core sample from a masonry building.

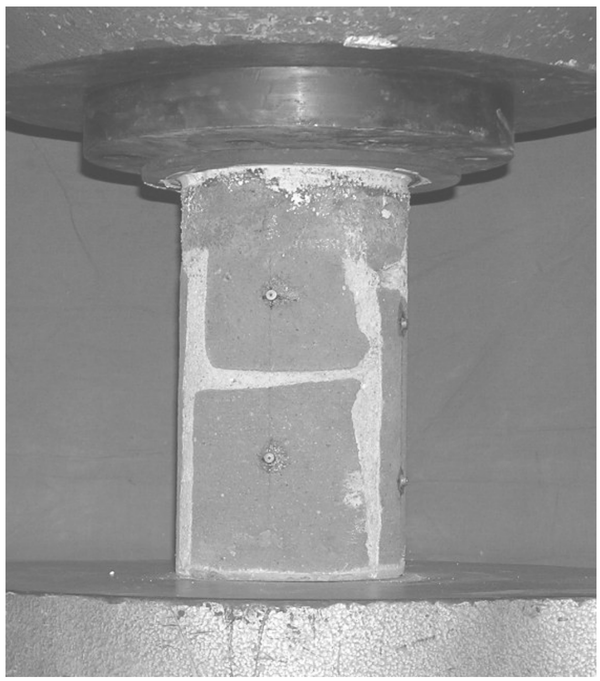


Figure 4.20 Testing of a core sample from a masonry building

Apart from the determination of compressive strength and elastic modulus, core samples may be used to further study the material proportions and microstructure. For concrete, the cement content and water-to-cement ratio can be determined using standard investigative chemical techniques. The microstructure can be studied for the presence of excess voids (Figure 4.21), or to monitor changes due to durability problems.

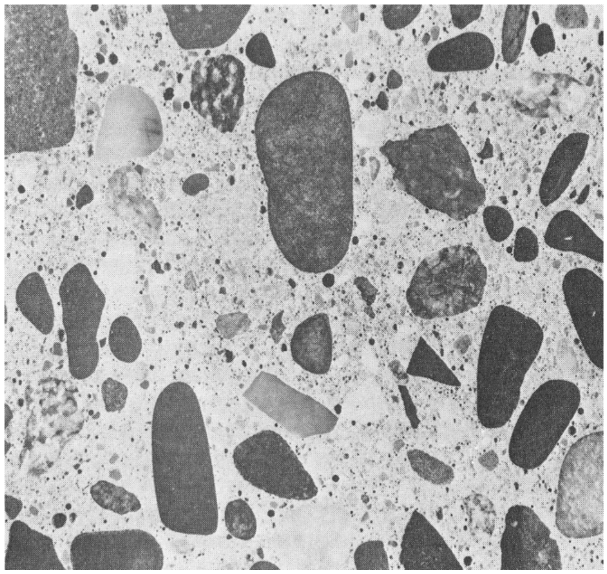


Figure 4.21 Petrographic examination of concrete using thin sections
(Source: Kosmatka, S. H. and Panarese, W. C., 1988, "Design and Control of Concrete Mixtures", Portland Cement Association)

A few other tests commonly performed on cores include the following.

- Determination of depth of carbonation

  The carbonation test is easy to perform. Phenolphthalein solution is sprayed immediately on the outer surface of the extracted core. The depth of carbonation is measured as the depth over which the concrete does not develop any pink colouration. If it turns pink then the concrete is uncarbonated.

- Determination of sulphates, chlorides and pH

Sulphates are determined using the test method in IS 4032: 1985, while chlorides may be determined using one of a number of tests available. The pH is measured using a pH meter as per IS 2720: 1987.

4.8.2 In-situ Shear Test

The in-situ shear test is used to find the actual shear strength of the mortar joint in a masonry wall. First, the mortar joints around one or two masonry units are removed. Next, a small jack is inserted in the wall and the in-situ shear test on mortar joint is carried out (Figure 4.22). While interpreting the results, the redistribution of vertical load due to the intervention should be accounted for. The procedure of the test is given in ASTM C1531-03.

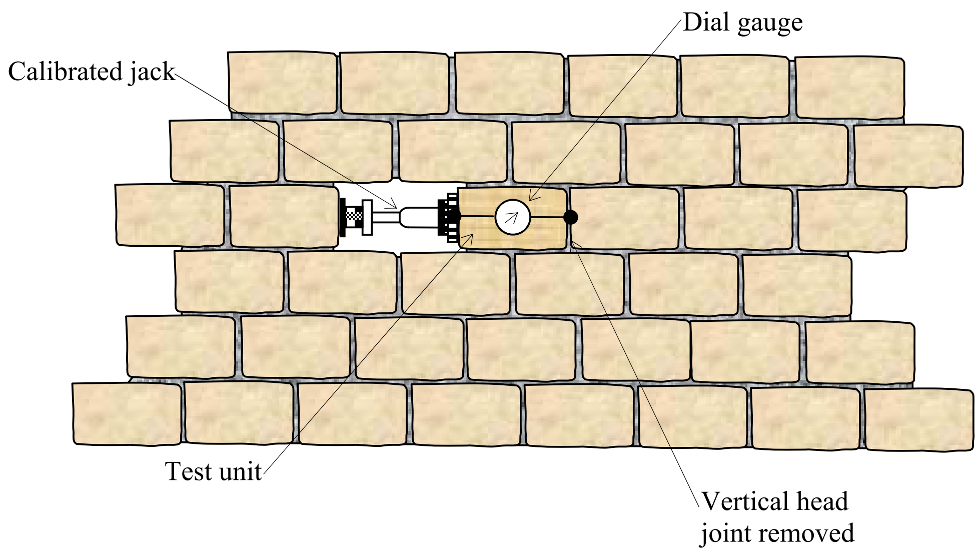


Figure 4.22 In-situ shear test

4.8.3 Flatjack Test

The flatjack test is used to find the in-situ compressive stress in a masonry wall. Flat jacks are inserted into horizontal slots cut in a wall (Figure 4.23). The pressure required to bring back the deformation across the opening is used to arrive at the in-situ compressive stress in the wall. The procedure of the test is given in ASTM C1196-04. The flatjack can also be used to measure deformability properties of masonry (ASTM C1197-04) and shear strength of mortar joints (ASTM C1531-03).

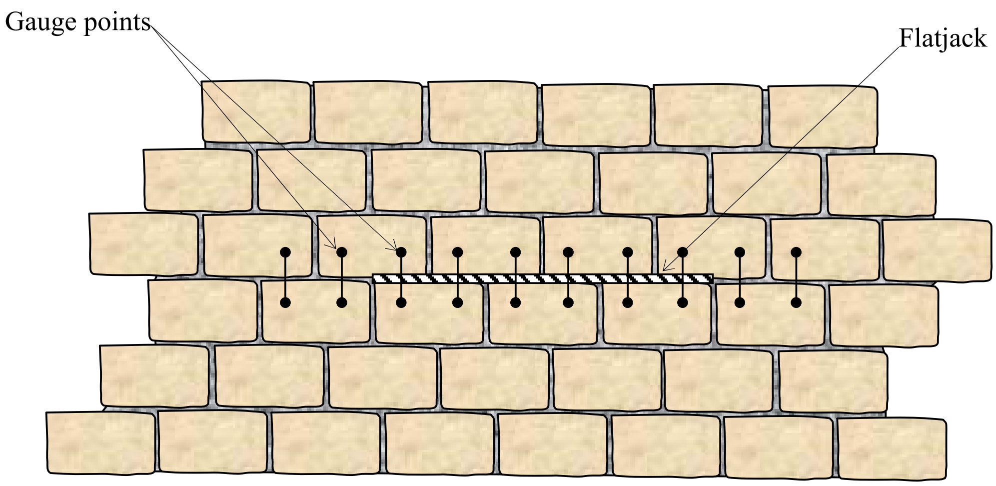


Figure 4.23 Flatjack test

4.8.4 In-situ Permeability Test

In-situ permeability can be assessed by introducing water into the cavities. The procedure of the test is given in IS 11216: 1985.

4.8.5 Tests of Masonry Prisms

Individual masonry units such as bricks, grout, mortar can be cut from walls and tested for their properties. For mortar, mini cubes can be made and tested. It is important to correct the results of such tests for size effects. Masonry prisms removed from a building provide most reliable properties. Such prisms (one unit wide and 2 to 5 units long) can be used for axial compression test, bond wrench test or other types of flexure test (ASTM E518-03).

4.8.6 Coupon Tests

Coupons from reinforcing bars in a concrete building or members of steel buildings can be tested for the yield stress and ultimate tensile strength. IS 1608: 1995 provides the procedure for making and testing coupons from steel sheets, bars, rolled sections and tubes. The corrosion rate can be determined by testing the loss of weight per unit time.

4.9 SUMMARY

In this chapter, first, the basic properties and causes of deterioration of masonry, concrete, steel and timber are briefly discussed. Next, under visual inspection, the accessories and items of inspection are explained. The detailed investigation techniques are presented under two sections. The non-destructive tests cover rebound hammer and penetration techniques, thermal methods, radiography, electromagnetic techniques, sounding, ultrasonic pulse velocity test, impact echo / pulse echo methods and measurement of corrosion. The intrusive tests cover the core tests for concrete and coupon tests for steel. The corresponding standards for the tests are mentioned.

4.10 REFERENCES

1. ASTM C42-04 (2004), "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete," American Society for Testing and Materials, USA.
2. ASTM C803-03 (2003), "Standard Test Method for Penetration Resistance of Hardened Concrete," American Society for Testing and Materials, USA.
3. ASTM C876-91 (1999), "Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete," American Society for Testing and Materials, USA.
4. ASTM C1046-95 (2001), "Standard Practice for In-situ Measurement of Heat Flux and Temperature on Building Envelope Components", American Society for Testing and Materials, USA.
5. ASTM C1196-04 (2004), "Standard Test Method for In-situ Compressive Stress within Solid Unit Masonry Estimated Using Flatjack Measurements", American Society for Testing and Materials, USA.
6. ASTM C1197-04 (2004), "Standard Test Method for In-situ Measurement of Masonry Deformability Properties Using the Flatjack Method", American Society for Testing and Materials, USA.
7. ASTM C1531-03 (2003), "Standard Test Methods for In-situ Measurement of Masonry Mortar Joint Shear Strength Index", American Society for Testing and Materials, USA.
8. ASTM E518-03 (2003), "Standard Test Methods for Flexural Bond Strength of Masonry", American Society for Testing and Materials, USA.
9. ASTM E748-02 (2002), "Standard Practices for Thermal Neutron Radiography of Materials", American Society for Testing and Materials, USA.

10. ASTM E1004-02 (2002), "Standard Practice for Determining Electrical Conductivity using the Electromagnetic (Eddy-Current) Method", American Society for Testing and Materials, USA.
11. IS 516: 1959, "Methods of Tests for Strength of Concrete", Bureau of Indian Standards.
12. IS 1199: 1959, "Methods of Sampling and Analysis of Concrete", Bureau of Indian Standards.
13. IS 1608: 1995, "Mechanical Testing of Metals – Tensile Testing", Bureau of Indian Standards.
14. IS 2720: 1987, "Method of Test for Soils", Part 26: Determination of pH Value, Bureau of Indian Standards.
15. IS 3664: 1981, "Code of Practice for Ultrasonic Pulse Echo Testing by Contact and Immersion Methods", Bureau of Indian Standards.
16. IS 4032: 1985, "Method of Chemical Analysis of Hydraulic Cement", Bureau of Indian Standards.
17. IS 11216: 1985, "Code of Practice for Permeability Test for Masonry During and After Construction", Bureau of Indian Standards.
18. IS 13311: 1992, "Non-Destructive Testing of Concrete – Methods of Test", Part 1: Ultrasonic Pulse Velocity, Part 2: Rebound hammer, Bureau of Indian Standards.
19. Handbook on Repair and Rehabilitation of RCC Buildings (2002), Central Public Works Department.

5

RETROFIT OF NON-ENGINEERED BUILDINGS

5.1 OVERVIEW

Non-engineered buildings are those, which are not formally designed, but built using traditional techniques. They are made of mud/brick/stone/concrete walls, wooden/bamboo/casuarina posts and thatch/tile/wooden/concrete roofs. Recent earthquakes and cyclones have revealed their vulnerability to lateral loads. Hence, it is necessary to retrofit the buildings to mitigate the disaster possible in such events. This chapter describes only seismic retrofit for the buildings with heavy roof, although some provisions are applicable for cyclone resistance. There is information on the preferred seismic resistant features of non-engineered buildings. The features aim to enhance the integrity of the buildings. The horizontal bands and vertical reinforcement at key locations, proper size and location of the openings and features in typical buildings are highlighted.

The available repair materials and techniques are included in this chapter, although they are applicable for semi-engineered and engineered buildings as well. The advanced repair materials such as shotcrete, epoxy resin, epoxy mortar, quick setting cement mortar, micro-concrete, fibre-reinforced concrete, ferro-cement and a few others are covered to provide some ready information.

The strengthening techniques are broadly grouped as member level and global techniques. Under the member level techniques, the strengthening of roofs, upstairs floors, walls and pillars are described. The strengthening of foundations is separately covered in Chapter 12. Under the global techniques, the introduction of joints, walls, pilasters, buttresses and braces, improvements of the frame and splint and bandage strengthening technique are covered. Several connection details are also provided.

5.2 INTRODUCTION

Many buildings are constructed in the traditional manner without formal design by qualified engineers or architects. The materials include field stone, brick, concrete blocks, rammed earth, wood posts, thatch, tiles, concrete, etc. If blocks are used to make the walls, they are attached together with mud, lime or cement mortar. Sometimes, combination of mortars is also used. Such buildings are built using combination of load bearing walls, piers in masonry, columns in reinforced concrete, steel or wooden posts. The safety of these buildings against earthquakes is of great concern, as huge losses of lives have occurred in such buildings during past earthquakes (Latur, 1993, Kashmir, 2005).

During the survey after the earthquakes, it has been observed that the main reasons for the damage to these buildings are the use of non-engineered construction methods without adequate earthquake resistant features, and the poor quality of workmanship. Hence, Bureau of Indian Standard has published the following codes of practice for earthquake safety of non-engineered and semi-engineered buildings. It is essential that these codes are followed in letter and spirit for the development of sustainable disaster free habitat.

1. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings – Code of Practice".
2. IS 13827: 1993, "Improving Earthquake Resistance of Earthen Buildings – Guidelines".
3. IS 13828: 1993, "Improving Earthquake Resistance of Low strength Masonry Buildings – Guidelines".

5.3 SEISMIC RESISTANT FEATURES

Based on the material, non-engineered buildings can be broadly classified as follows (Guidelines for Earthquake Resistant Non-engineered Construction, 2004).

1. Buildings in brick and other masonry units
2. Stone buildings
3. Wooden buildings
4. Earthen buildings
5. Non-engineered reinforced concrete buildings.

IS 4326: 1993 gives the basic requirements for seismic resistance of buildings with one or two storeys. Some of these requirements are explained along with the seismic vulnerability aspects in Chapter 2. In this section, a few requirements are highlighted.

The different parts of a building should be tied together so that the building acts as one unit under seismic forces. The essential features for resisting seismic forces are as follows.

- The building should be tied to its foundation.
- Horizontal bands of reinforcement are to be provided at the plinth level (plinth band), sill level (sill band), lintel level (lintel band) and roof level (roof band). Vertical reinforcing bars are to be provided at the corners of the building, junctions of walls and around the openings in walls.
- There should be horizontal bracing (bracing in plan) at the roof level for sloping roofs.
- The openings should be small and centrally located in the walls.

The roof band is required when the roof is sloping and the roof is made of flexible material, such as tiles on purlins and rafters. The roof band is not necessary when the roof is made of flat concrete slabs. For better seismic resistance, openings should be located away from the inside of a corner by a certain distance. The width of the pier between two openings should not be less than a minimum value. The total length of openings should not exceed a certain percentage of the span of a wall (distance between consecutive cross walls). The features for seismic resistance as per IS 4326: 1993, IS 13827: 1993 and IS 13828: 1993, are shown in Figures 5.1 to 5.4. The specific limits are available in the codes.

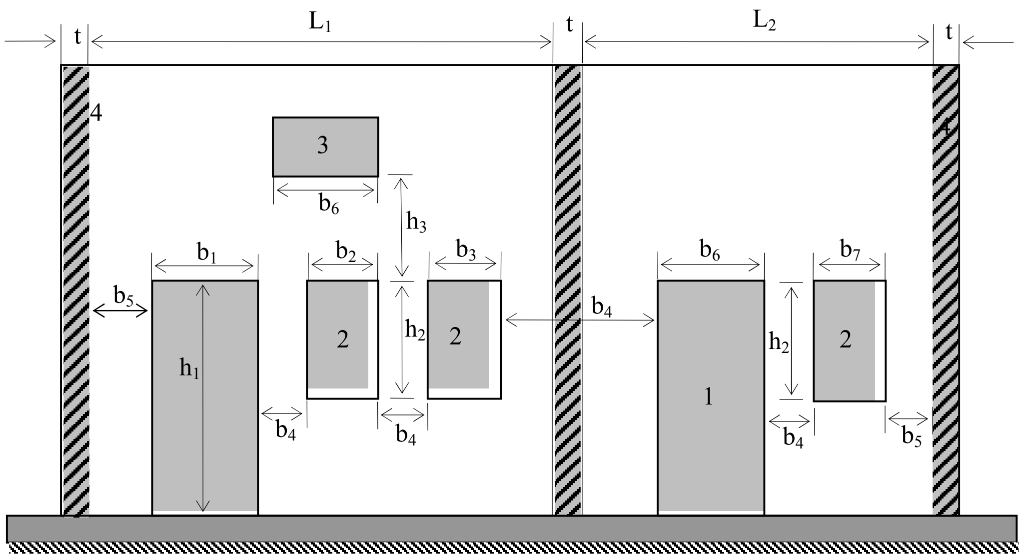
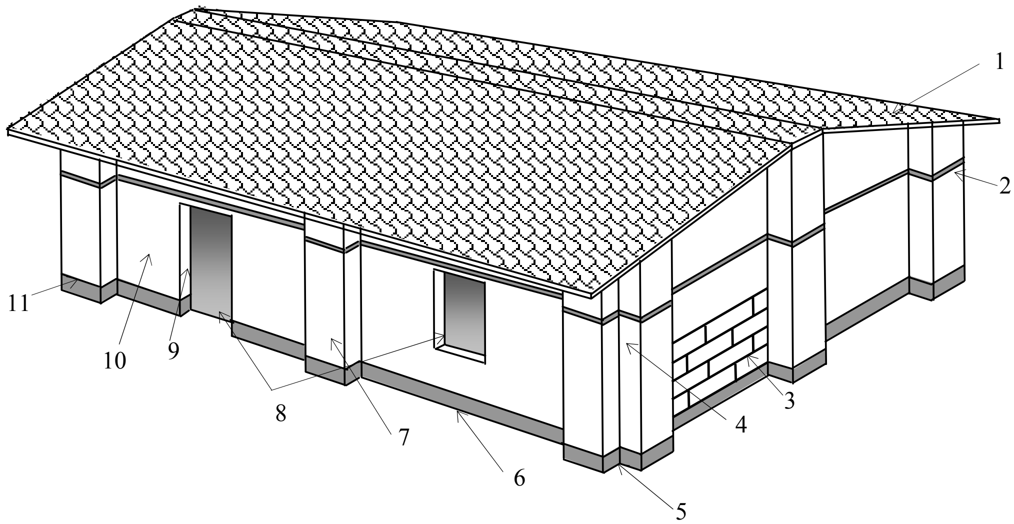
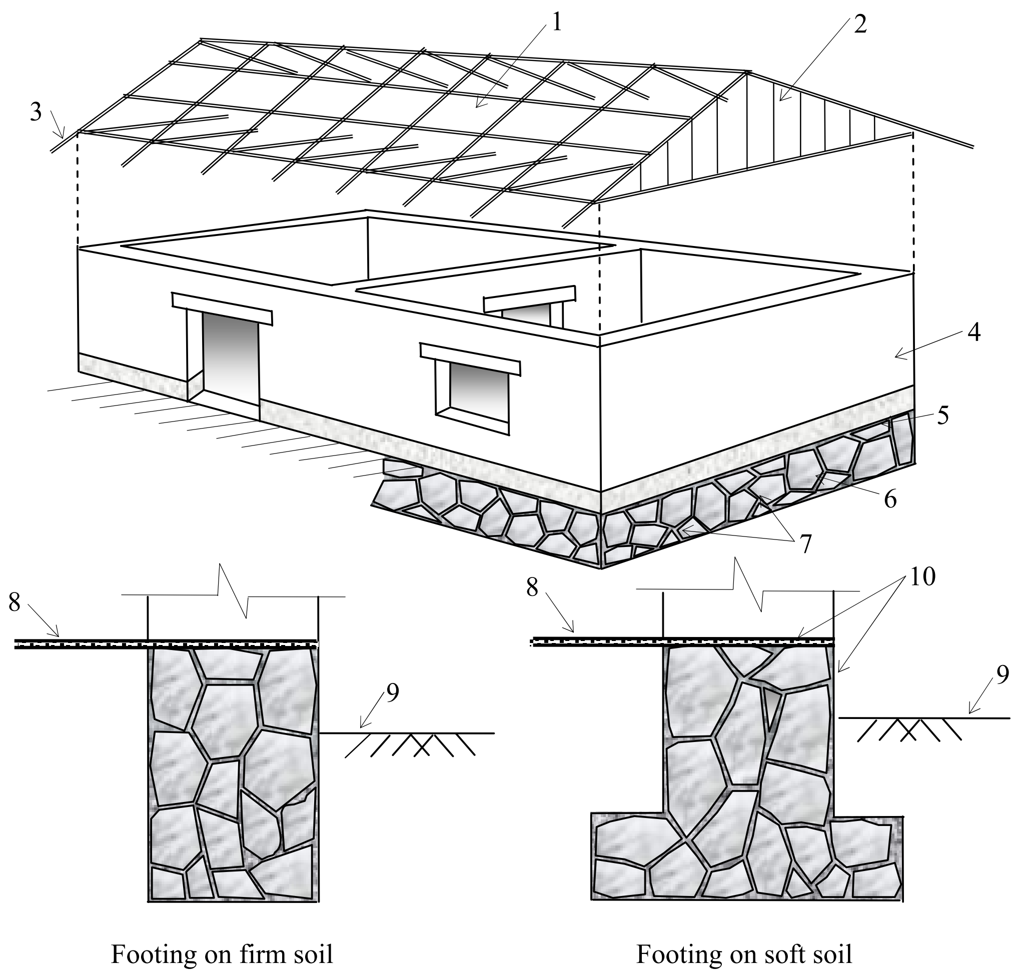


Figure 5.1 Features of openings



1. Sheeting or cemented tiles as roof material
2. Horizontal reinforcement or collar beam
3. Good bond and alternated vertical joint for adobe blocks
4. Proper junction of walls
5. Foundation plinth (300 mm min).
6. Length of wall max. 10 times the thickness
7. Buttress in long walls
8. Small door and window openings
9. Opening 1.2 m from corner
10. Height of walls max. 8 times the thickness
11. Damp proof at plinth level

Figure 5.2 Features in an earthen building



1. Light roof
2. Light gable wall (matting or boarding)
3. Rain protection overhang about 500 mm
4. Stable plaster
5. Plinth height for flood protection
6. Stable foundation
7. Good mortar preferably non-clay
8. Floor level
9. Ground level
10. Water proof layer

Figure 5.3 Details in an earthen building

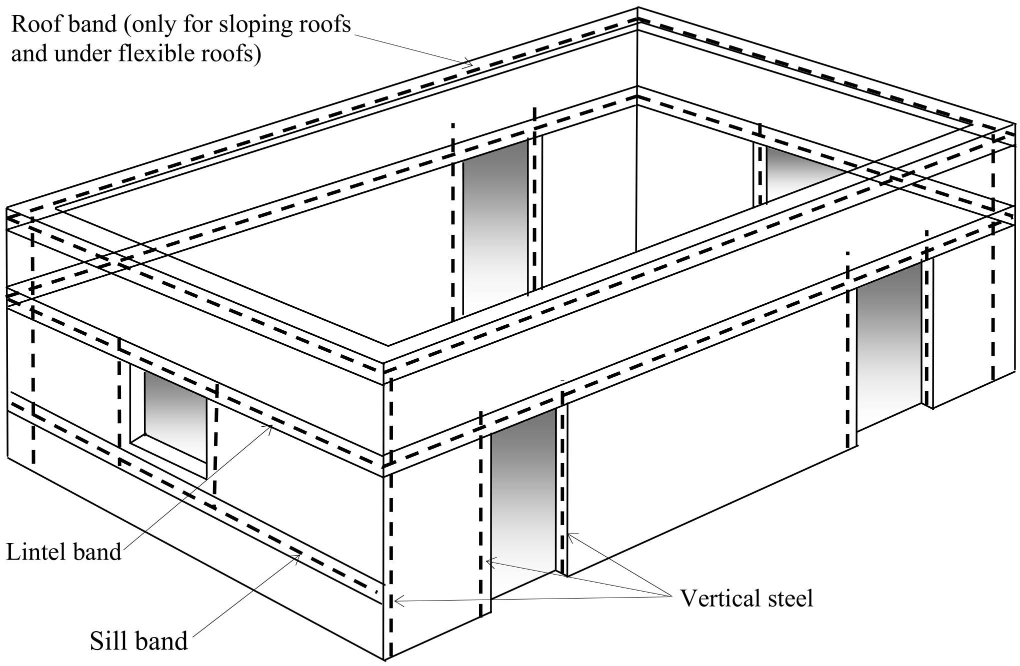


Figure 5.4 Features in a masonry building

5.4 REPAIR MATERIALS

The interventions to enhance the seismic performance of a building can be broadly grouped under the categories of repair, rehabilitation and retrofit. These terms have been explained in Chapter 1. The traditional materials used for repair are mud, lime, cement, sand, brick, stone and steel. Wood, bamboo, casuarina posts are often used for supports and braces. Cement or lime is combined with sand and water to prepare the mortar. Various types of cement with properties such as shrinkage compensation, low heat evolution and sulphate resistance are preferred for specific repair applications. Steel is used in the form of bolts, threaded rods, angles, channels and prestressing strands. A brief summary of a few advanced materials are given below.

5.4.1 Epoxy Resins

Epoxy resins are used for the following purposes.

1. To bond plastic concrete to a hardened concrete surface
2. To bond two rigid materials
3. For patch work
4. For applying a coating over concrete surface to give colour, resistance to penetration of water and chemicals and resistance to abrasion.

Epoxy resins are excellent binding agents. The low viscosity resins can be injected into small cracks. The higher viscosity resins are used as coating and for filling larger openings or holes.

5.4.2 Epoxy Mortar

The epoxy mortar is made using epoxy resin and sand. It has high compressive strength, high tensile strength and low modulus of elasticity. Epoxy resin can be added as a second binder in cement mortar. Here, the polymer particles join and form chain link reinforcement, increasing the tensile and flexural strengths of the cement mortar. There is greater plasticity and reduction in shrinkage stress.

5.4.3 Quick Setting Cement Mortars

These are patented mortars generally having two components and are sold in a pre-packed state. They may be classified as follows.

- Unmodified cementitious
- Polyester or epoxy resin based
- Polymer modified
- Cement/pozzolanic modified

Cementitious mortars such as gypsum cement mortar have limited use for structural purposes and are intended for architectural hand/trowel applications. The use of various types of mortars is given in Table 5.1.

Table 5.1 Applications of mortars

S. No.		Type of mortar	Properties
	Defect		
1.	Minor surface defect	Polymer modified cementitous mortar	• Gives a fair surface. • Good water proofing. • Resists acids and gases.
2.	Surface cavities and honey-combed concrete	Highly adhesive, thixotropic mortar	• Water proof and anti-carbonation finish. • Good resistance to pollution.
3.	Powdery surface	A two component surface stabilizer	• Binds powdery surface. • Evens out absorption characteristics.
4.	Non-structural cracks	Non-shrinking polymer filler	• Easily applied elastic compound. • Eases at low temperatures.
5.	Minor voids of approximate size 100 mm × 100 mm × 50 mm	Rapid curing polymer modified cementitious mortar	• High strength. • Can be compacted in layers.
6.	Major voids of approximate size 200 mm × 200 mm × 150 mm	Heavy duty thixotropic fiber reinforced polymer modified cementitious mortar	• Can be applied up to 100 mm thick without sag. • Easy to mould.
	Other uses		
7.	Surface protection	Resin rich water based co-polymer	• Highly resistant to diffusion. • Self cleaning.
8.	Surface barrier	Water based co-polymer	• Resistant to fungal attack.
9.	Bonding agent	Polymer modified cementitious surface impregnant	• High penetration into porous concrete creating enhanced adhesion.
10.	Protection of steel reinforcement	Two component system of cementitious powder and polymer	• High penetration. • React chemically to generate passivity of steel.

5.4.4 Shotcrete

Shotcrete is a method in which compressed air forces mortar or concrete through a nozzle to be sprayed on a surface of building component such as a wall, at a high velocity. The materials used in shotcrete are generally same as those used for conventional concrete. The reinforcement provided is welded wire fabric or deformed bars tacked on the existing surface.

Shotcrete is applied using either wet or dry process. The wet mix consists of cement and aggregate premixed with water and the pump pushes the mixture through the hose and a nozzle. Compressed air is used at the nozzle to increase the velocity of application. In the dry mix process, compressed air propels premixed mortar and damp aggregate, and at the nozzle end, water is added through a separate hose. The dry mix and water are projected on to the surface through a second hose. In most cases, shotcrete can be applied in a single application for the required thickness. It is a versatile technique as it can also be applied to curved or irregular surfaces. Its strength after application and its good physical characteristics make it ideal for strengthening weak members.

5.4.5 Micro-concrete

Based on hydraulic binders, these ready-made formulations are tailored to produce concrete which is flowable and free of shrinkage. They are applied in complicated locations and in thin sections such as concrete jackets.

5.4.6 Fibre-reinforced Concrete

Fibre-reinforced concrete has better tensile strength as compared to conventional concrete. They also have improved ductility (energy absorption capacity) and durability. They are being increasingly used for structural strengthening.

5.4.7 Fibre-reinforced Polymers

The fibre-reinforced polymer (FRP) composites are made up of a polymer matrix and fibres. The fibres can be of glass, carbon or aramid. They possess high strength-to-weight ratios, high fatigue strength, high wear resistance, vibration absorption capacity, dimensional stability, high thermal and chemical stability and corrosion resistance. They are manufactured in long lengths by the pultrusion process. FRP wraps can be used to strengthen structural members. The details of the use of FRP are given in Chapter 13.

5.4.8 Ferro-cement

Ferro-cement is constructed of cement mortar reinforced with closely spaced layers of small diameter wire mesh. The mesh may be made of steel or other suitable material. The mortar should be compatible with the opening size and weight of the mesh. The mortar may contain discontinuous fibres. The technique of ferro-cement overlay is explained in Chapter 6, Retrofit of Masonry Buildings. The use of ferro-cement can be economical even for non-engineered buildings.

5.5 REPAIR TECHNIQUES

The applicability of repair techniques varies depending on the problems to be addressed. The associated techniques are presented under the specific problems.

5.5.1 Small Cracks

Cracks reduce the strength of load bearing members especially when they are not reinforced, as in masonry or plain concrete. Hence, they should be marked carefully and the critical ones repaired. Cracks that are small in width ($\leq$ 0.75 mm) can be effectively repaired by pressure injection of epoxy (IS 13935: 1993).

The surfaces are thoroughly cleaned of loose materials. Injection ports are placed along the length of the cracks on both sides, at intervals approximately equal to the thickness of the member (Figure 5.5). Low viscosity epoxy resin is injected into the ports sequentially, beginning at the port at the lowest level and moving upwards one by one. The resin is pushed through the packer till it is seen flowing from the other end or from a port higher than where it is injected. The port is closed at this juncture and the packer is moved to the next higher port.

Larger cracks will require larger packer spacing depending on thickness of the member. Vacuum injection has a typical fill level of 95 percent and can fill cracks as small as 0.025 mm. A similar technique can be applied to strengthen weak walls (Figure 5.6).

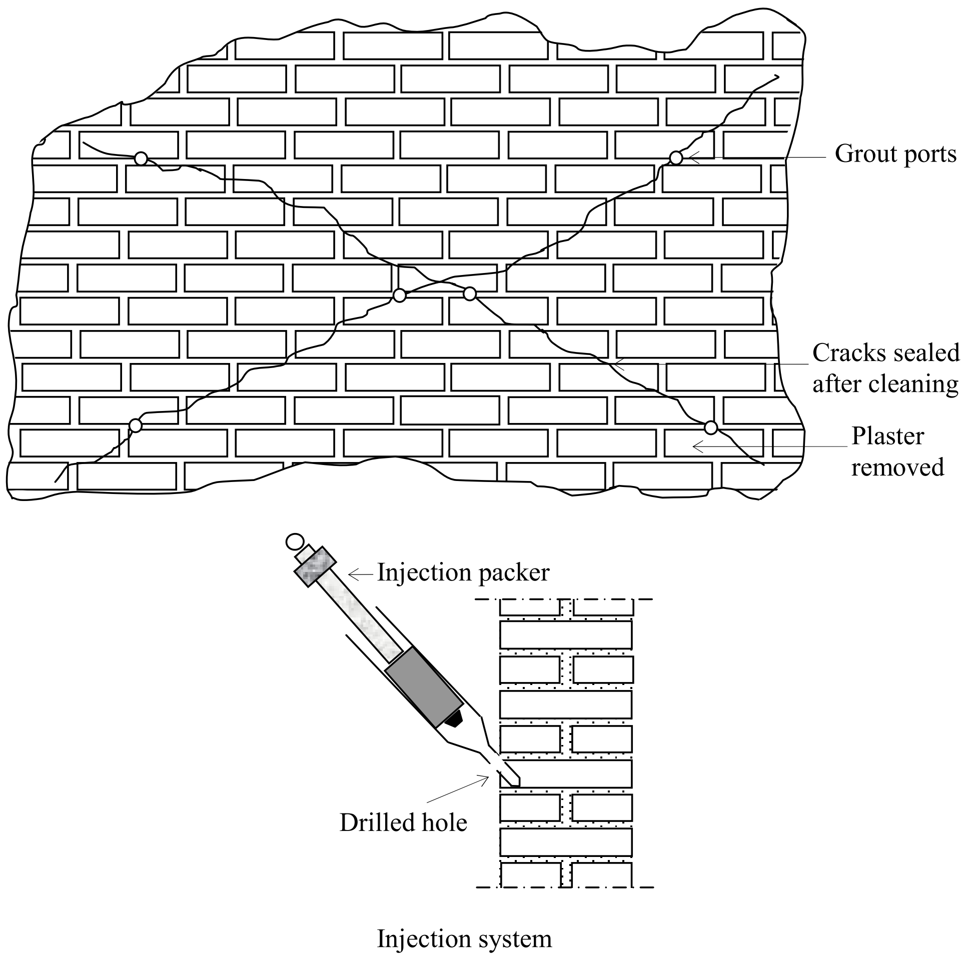


Figure 5.5 Grout or epoxy injection in cracks

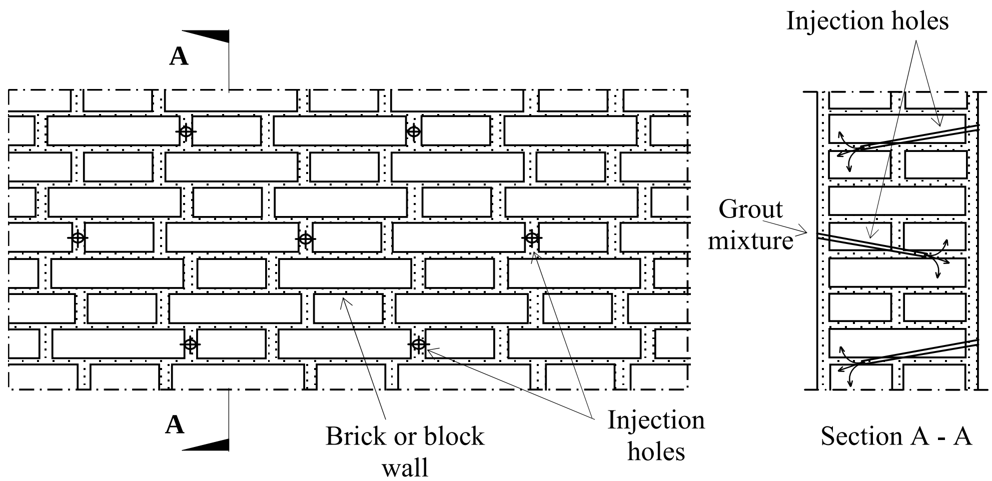


Figure 5.6 Grout or epoxy injection in weak walls

5.5.2 Large Cracks and Crushed Material

For cracks with width larger than 6 mm or in regions were brickwork or concrete is crushed, the following procedure is suitable.

1. Loose material in the crack is removed and any of the repair mortar mentioned in Section 5.4 is filled.
2. If necessary, the crack is dressed to have a V grove.
3. At wide cracks, fillers like flat stone chips can be used.
4. To prevent widening of the cracks, they can be stitched (Figure 5.7)

The stitching consists of drilling small holes of diameter 6 to 10 mm on both sides of the crack, cleaning the holes, filling up these with epoxy mortar and anchoring the legs of stitching dogs (U-shaped steel bars of diameter 3 to 6 mm with short legs). The stitching dogs can have variable length and orientation. The spacing of the reinforcement should be reduced at the ends of the crack. Stitching will not close the crack, but it prevents further propagation and widening of the crack. The stitching will stiffen the area near the vicinity of the crack.

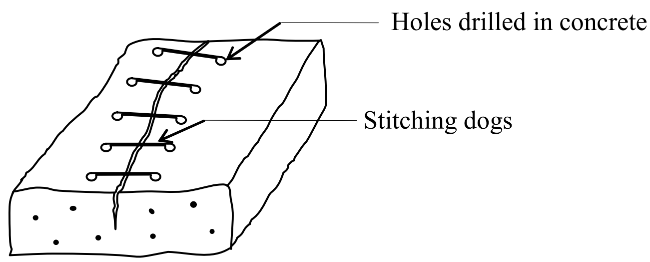


Figure 5.7 Repair by stitching the cracks

5.5.3 Damaged Reinforcement

After an earthquake, the reinforcement in a reinforced concrete element can be severely damaged, showing signs of either buckling or elongation due to yielding. The element can be repaired by attaching new bars with the old bars by either lap or butt welding. Additional stirrups are added at the locations of damage and then covered with concrete to provide confinement of the existing concrete.

5.5.4 Decayed Wooden Members

Decayed areas of timber usually occur at the bottom chord members of trusses because they are subjected to tensile forces, which tend to pull the joint apart. Surgery is performed on decayed areas by sawing and gouging out material. Reinforcement in the form of steel bar; carbon fiber or glass fiber is inserted into predrilled holes filled with resin grout. The area is then shuttered and filled with resin and allowed to cure (Figure 5.8)

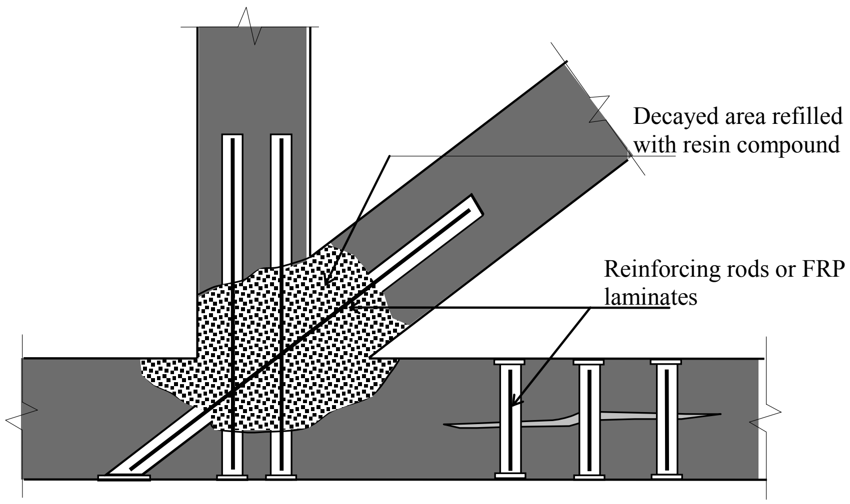
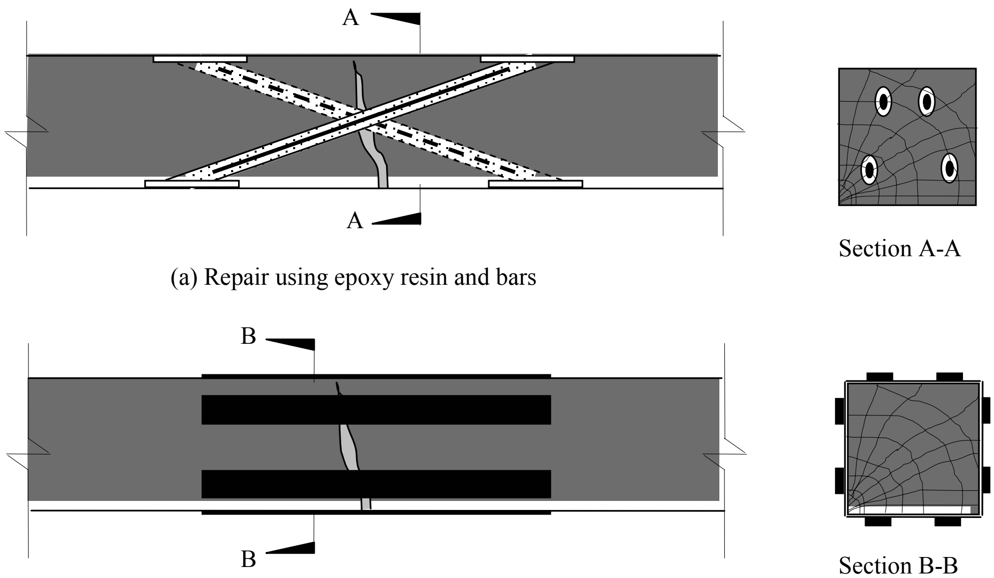


Figure 5.8 Repair of a defective timber joint

If a member is cracked, split or rotted right through, it can be repaired by filling the void with epoxy resin and reinforcing bars in two planes from opposite surfaces as shown in Figure 5.9a. Alternatively, FRP strips can be bonded on the surface with a timber laminate rendering the repair work invisible (Figure 5.9b).



(a) Repair using epoxy resin and bars

(b) Repair using FRP strips

Figure 5.9 Repair of a fractured timber member

The problem of decaying ends of timber members such as stringers and joists can be addressed using steel bars in conjunction with epoxy resin. The decayed section is cut off and replaced with a new preformed section, with protruding epoxy bonded reinforcing bars. The bars are placed in a slot cut in the good timber adjacent to the cut end. Epoxy mortar is then poured into the slot to attach the bars to the timber (Figure 5.10).

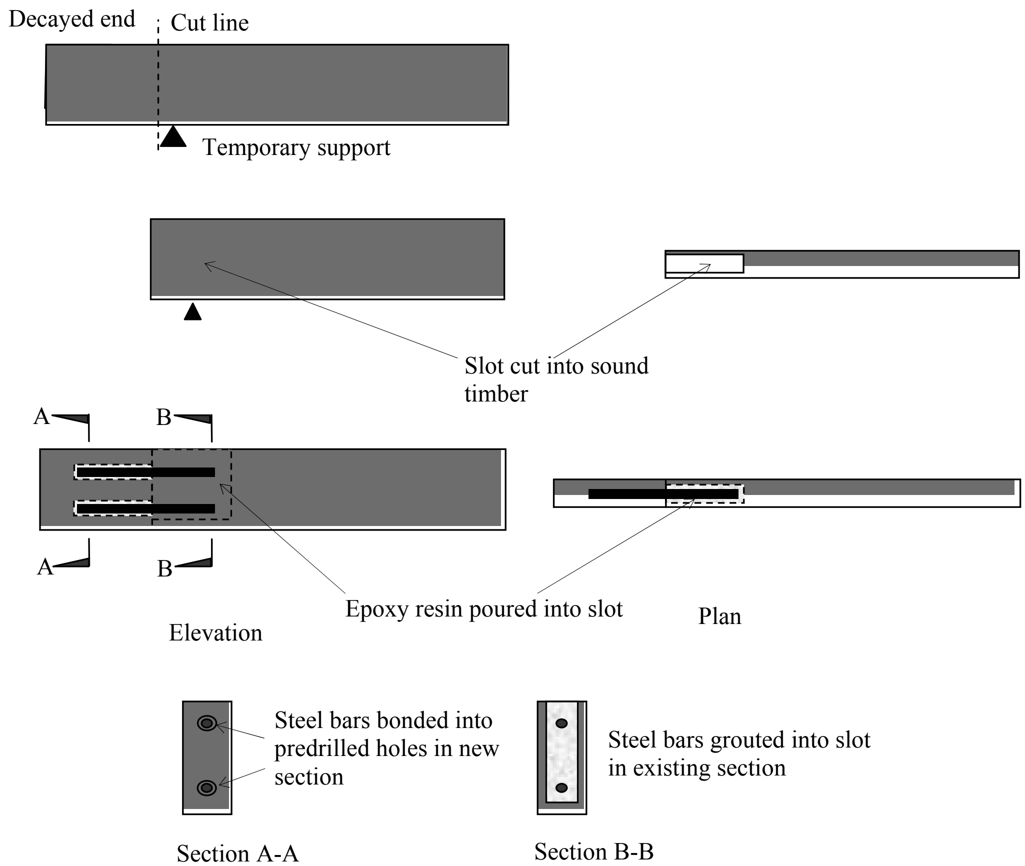


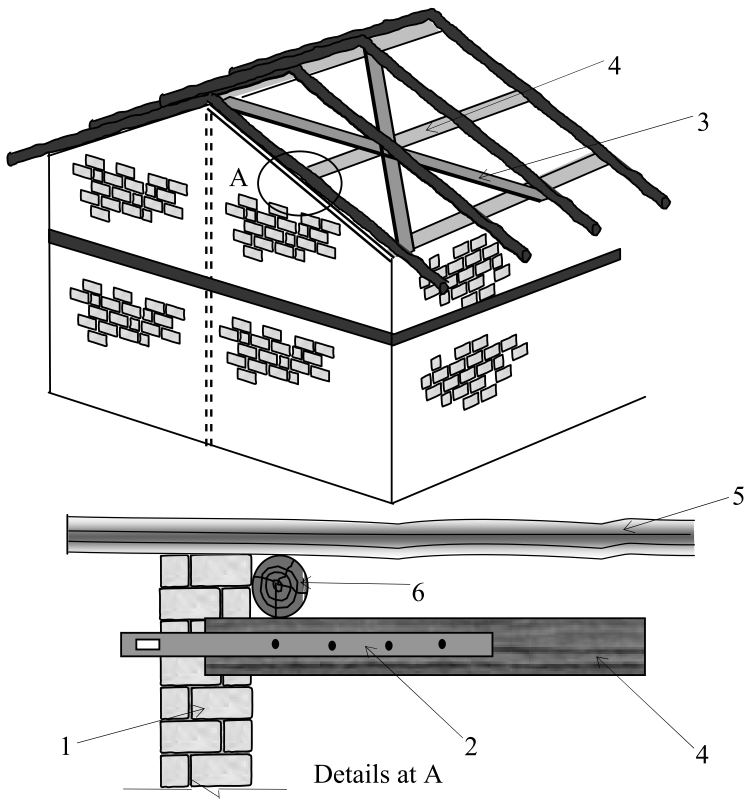
Figure 5.10 Replacement of the decayed end of a timber member

5.6 STRENGTHENING OF ROOFS

The roof should effectively distribute the lateral load generated due to earthquake, to the walls. In other words, the roof should act as a horizontal diaphragm. There are varieties of roofs in non-engineered buildings. The most common types of roofs are considered here.

5.6.1 Timber Truss Roofs

A timber roof truss usually supports tiles which are brittle and tend to get dislodged. The tiles can be replaced with galvanised iron corrugated sheets. A secondary (false) ceiling of brittle material is hazardous. Ductile material like bamboo matting or light foam type material can be used to replace brittle false ceiling. Anchors of roof trusses to supporting walls should be improved and the roof truss thrust on the walls should be minimized or eliminated. This can be achieved by providing new bracing as shown in Figure 5.11 (IS 13935: 1993).



1. Existing gable wall (tympanum)
2. Steel strips bolted to new ties (4) and wall (1)
3. New planks for diagonal bracing
4. New planks for ties
5. Roof covering
6. Existing roof rafters

Figure 5.11 New bracing in a timber truss roof

5.6.2 Terrace Roofs

Where the roof consists of prefabricated units or channels, integration of individual units is a must. The units can be integrated by a cast-in-situ reinforced concrete topping and a band. The technique is similar to that shown in Figure 5.15. The detail can also be adopted for Madras terrace roof, which is discussed in Chapter 6.

5.6.3 Roofs of Light Weight Sheets

Although roofs made of asbestos or galvanized iron / aluminium sheets do not cause heavy damage to the building because of their light weight, there is damage in the roofs during earthquakes. This can be reduced by close spacing of the hold down bolts, which connect the sheets to the purlins.

5.6.4 Thatched Roofs

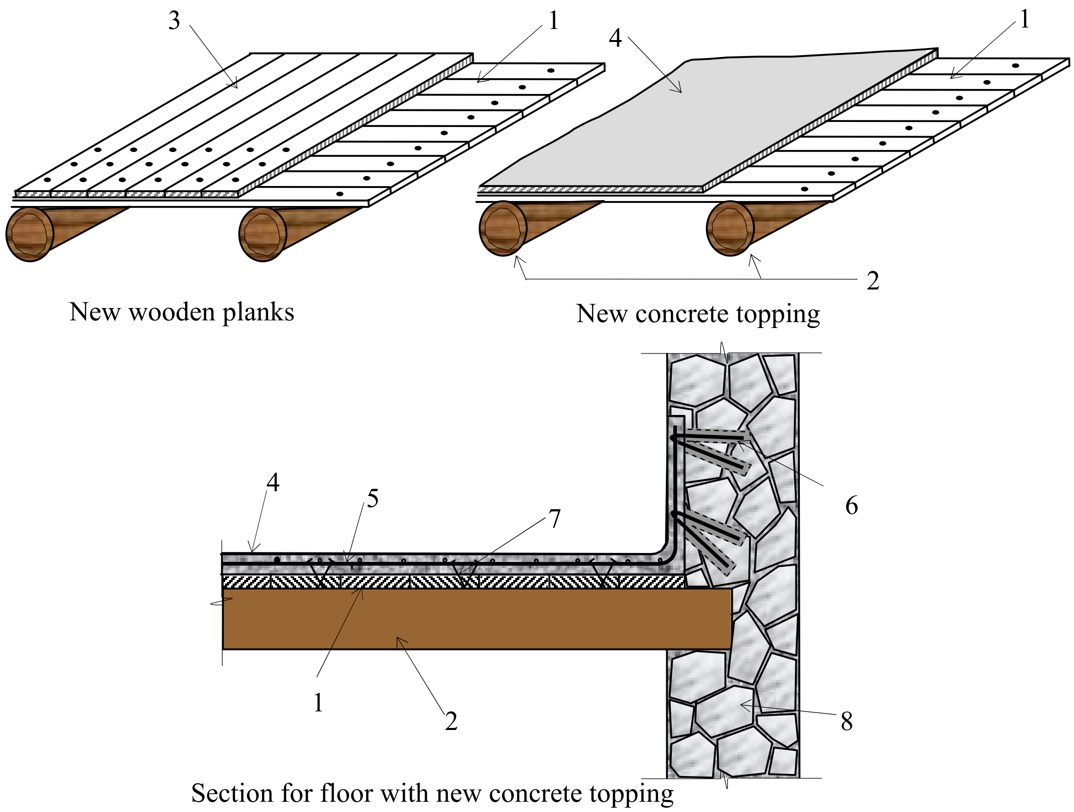
Thatch made from leaves of palm and coconut trees is widely used as roofing material in rural areas. Thatched roof are blown off during cyclones, damaged during earthquakes and are also vulnerable to fire. The performance of the thatched roof can be significantly improved by holding it down using ropes in a diagonal fashion. Periodical fire retardant spray will ensure fire safety. This is a must if loss of life due to fire is to be avoided.

5.7 STRENGTHENING OF UPSTAIRS FLOORS

Like the roof, an upstairs floor in a two-storeyed building also acts as a horizontal diaphragm. Strengthening of the floor involves two issues. First, the lateral load will generate in-plane forces in the floor. For this, the integrity of the floor has to be improved by providing braces and ties at the periphery. Second, the connections between the walls and the floor have to be improved, so that the seismic forces can be transferred from the floor to the walls. These issues are described for floors made of wooden planks or prefabricated units.

5.7.1 Wooden Floors

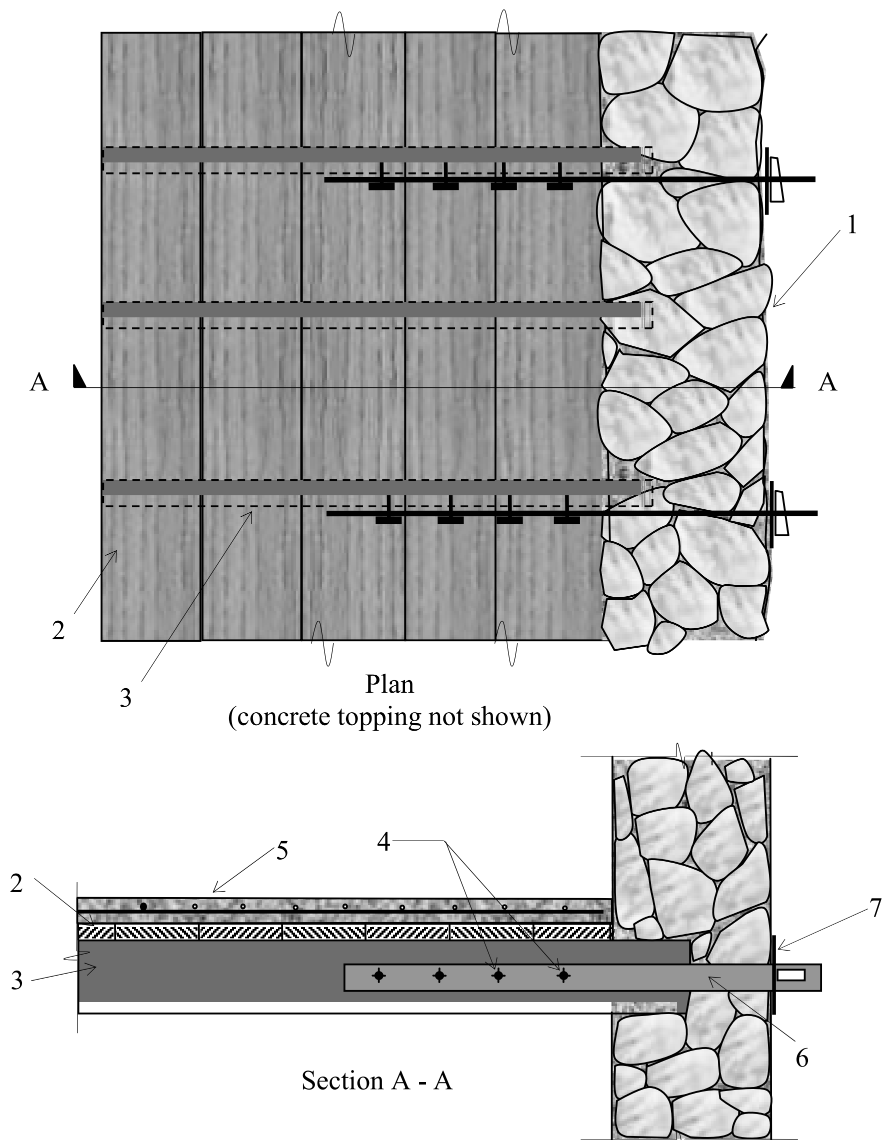
Strengthening of a wooden floor can be achieved by nailing planks perpendicular to the existing planks or by providing a thin reinforced concrete topping over the floor. For the connection with the wall, a grid of reinforcement is nailed to the planks and inserted into the wall with steel anchors, as shown in the following figure.



1. Existing planks
2. Existing beams
3. New planks
4. New concrete topping
5. Steel mesh reinforcement
6. Steel anchors grouted in drilled holes
7. Nails to fix mesh
8. Existing wall

Figure 5.12 Strengthening of wooden floor

It is essential to have a proper connection between the floor and the wall. Two techniques are shown in Figures 5.13 and 5.14, for the existing beams of the floor perpendicular and parallel to the walls, respectively. The connection consists of steel flats nailed to the beams. The holes drilled in the walls to anchor the flats should be filled with grout.



1. Existing wall
2. Existing planks
3. Existing beam
4. Nails
5. New concrete topping
6. Steel flats
7. Anchor plates

Figure 5.13 Connection of floor to wall (beams perpendicular to wall)

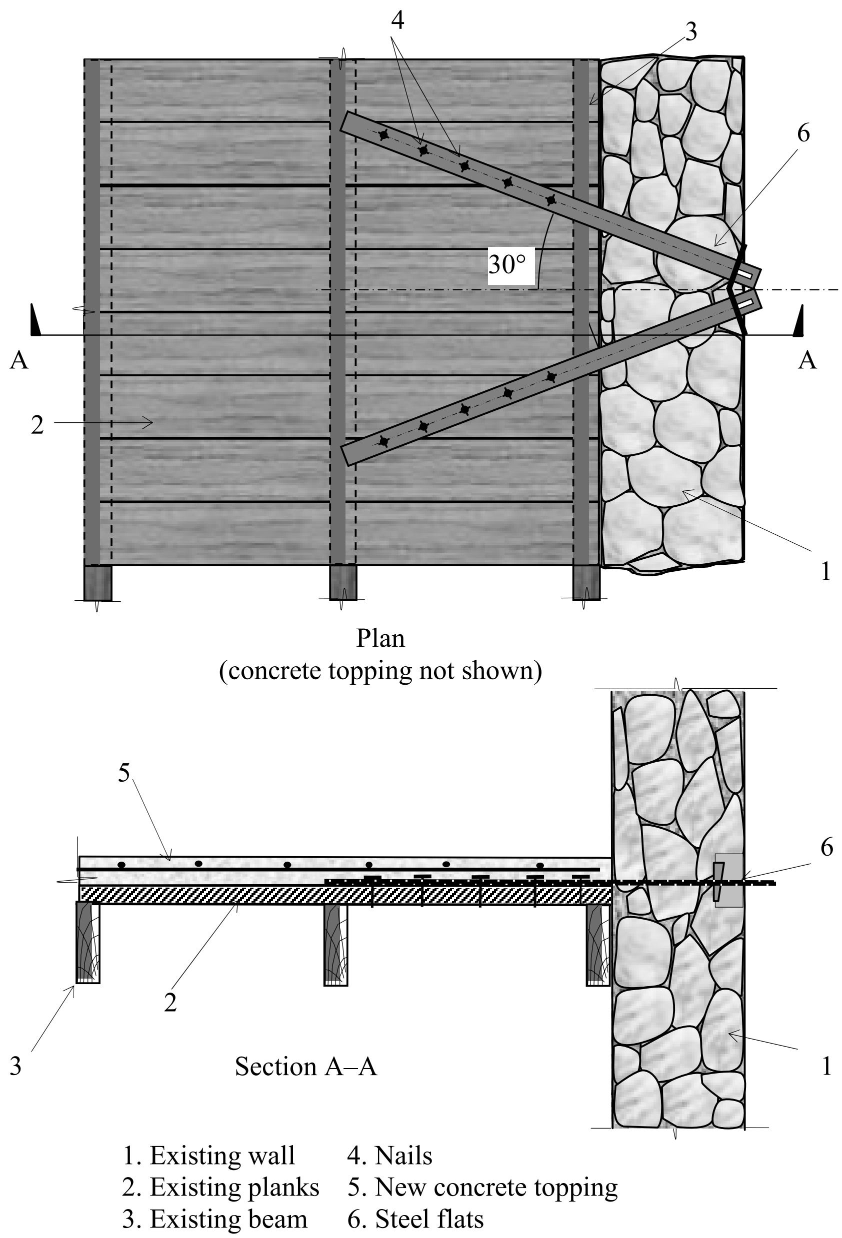


Figure 5.14 Connection of floor to wall (beams parallel to wall)

5.7.2 Floors of Prefabricated Units

Figure 5.18 shows the details for strengthening a floor made of prefabricated units or channels (IS 13935: 1993). The units are integrated with a reinforced concrete topping. A reinforced concrete band with keys is provided at the junction of the floor and the wall. The keys can be spaced at an interval of 3m.

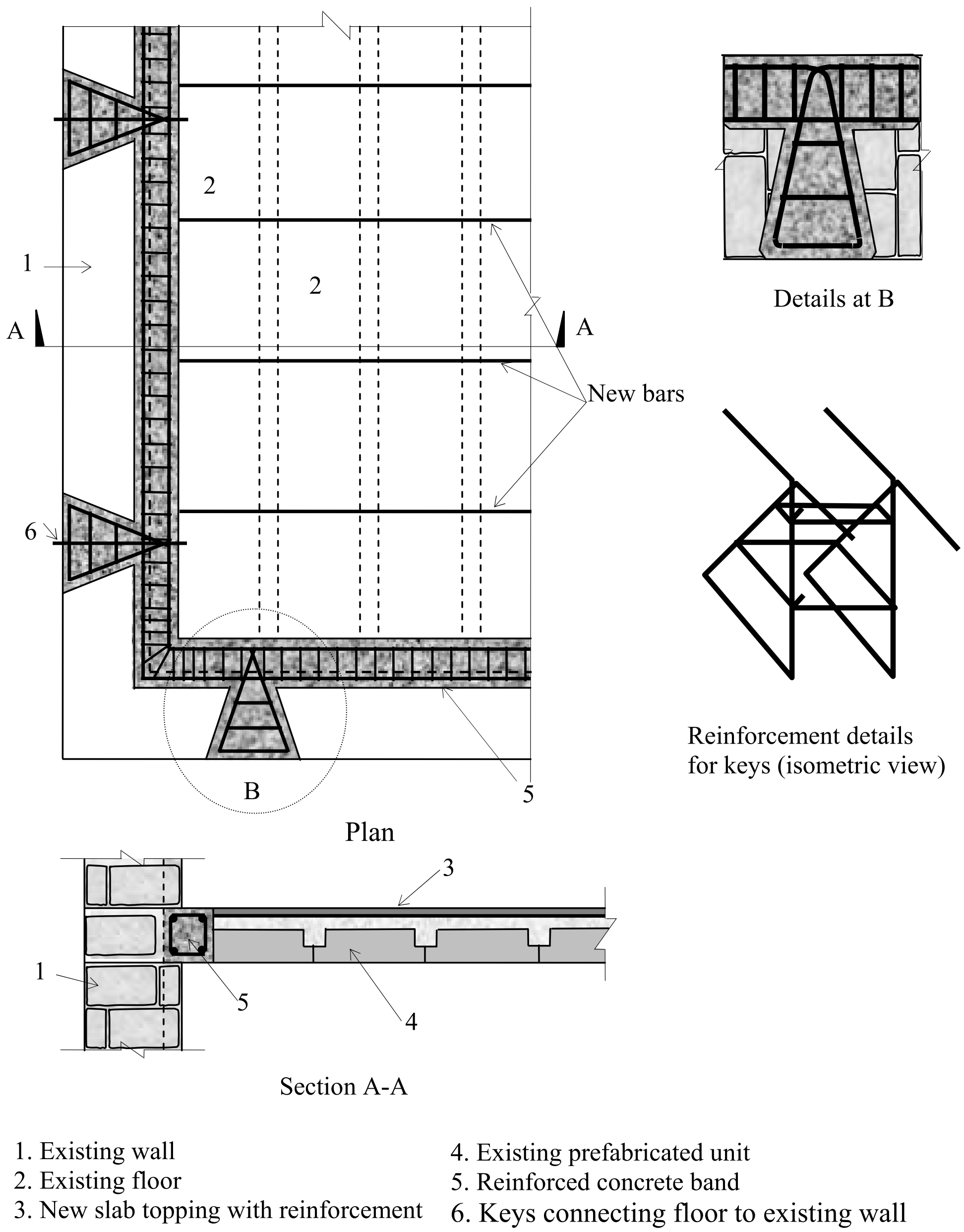


Figure 5.15 Strengthening of floor made of prefabricated units

5.8 STRENGTHENING OF WALLS

The walls support the roof and upstairs floor, as well as provide resistance to lateral load. The walls can be strengthened by the following methods.

a) Grouting
b) Containment reinforcement
c) Use of ferro-cement
d) Use of fibre reinforced polymer.

The method of grouting is discussed under repair techniques. The method of using containment reinforcement is described next. The other methods are covered in Chapter 6.

Containment Reinforcement

In this method, first grooves are made in the mortar joints. Next, horizontal, vertical and cross bars are inserted in the grooves which are subsequently covered by mortar.

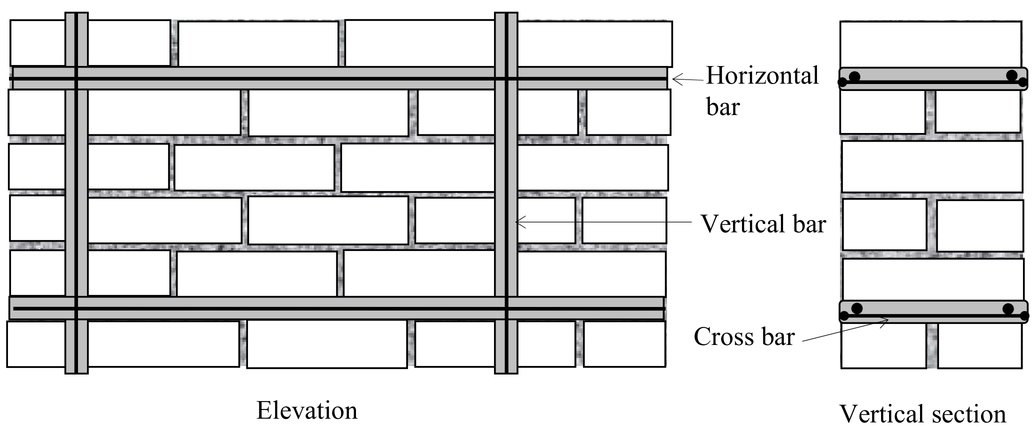
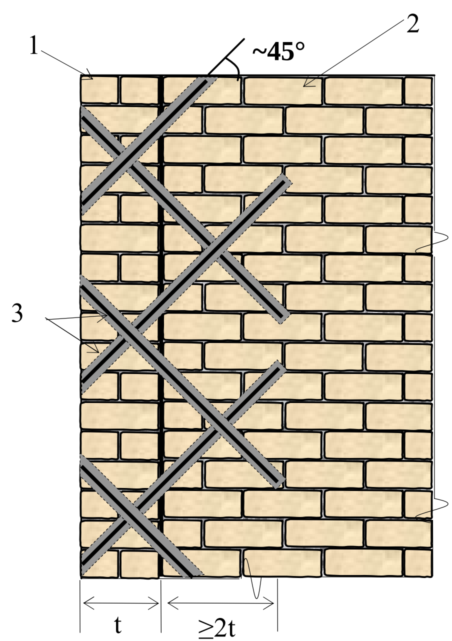


Figure 5.16 Containment reinforcement

Corners and T-junctions in perpendicular walls can be connected by effective ‘stitching’ of the walls. In this method, first, holes are drilled in an inclined pattern. Next, steel bars are placed in the holes. Subsequently, grout is injected to form the bond between the bars and the wall, as well as to provide protection against corrosion of the bars. Generally, 8 to 10 mm diameter bars of Grade Fe 415 are used for stitching. In multi-leaf walls, the inner filling can be strengthened by injection of grout. A collar band can be provided at the junction of two perpendicular walls at the lintel level to enhance the structural integrity (Figure 5.18). In many buildings, the partition walls are the first ones to collapse during an earthquake. To avoid failure, such a wall should be integrated by stitching with the two walls at its ends.



1. Longitudinal wall
2. Transverse wall
3. Bars in holes filled with grout

Figure 5.17 Connection of two existing perpendicular walls

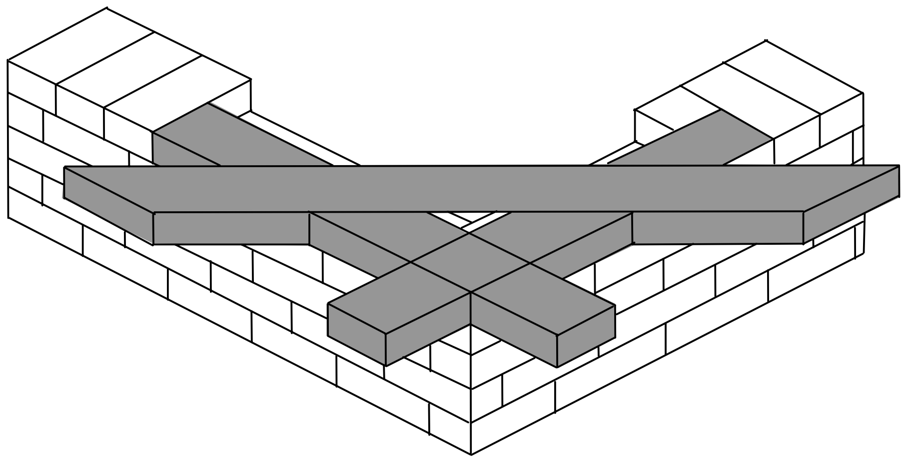


Figure 5.18 Collar band at the junction of walls at lintel level

5.9 STRENGTHENING OF PILLARS

In a barrack type of building, pillars made of bricks are placed along the corridor. These pillars support the roof trusses. In absence of any reinforcement, the pillars are weak under lateral forces, leading to failures (Figure 5.19). Such pillars can be strengthened by concrete jacketing, as shown in Figure 5.20. The additional stirrups can be inserted from the top of the pillars after lifting the truss by a temporary support. Else, each stirrup has to be made of two pieces as shown under column jacketing in Chapter 9, Retrofit of Reinforced Concrete Buildings. The trusses should be properly anchored to the pillars.

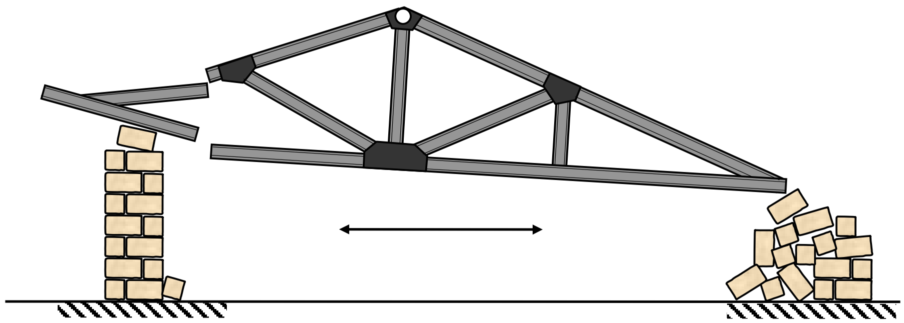


Figure 5.19 Collapse of a truss resting on pillars

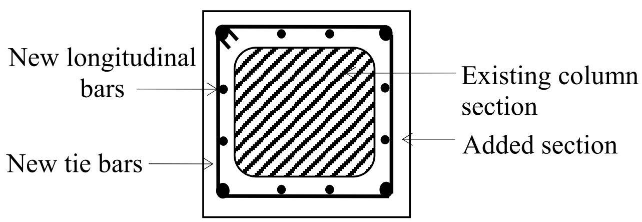


Figure 5.20 Strengthening of pillars by concrete jacketing

5.10 TECHNIQUES FOR GLOBAL STRENGTHENING

Global strengthening aims to improve the lateral load resistance of the building as a whole. This will relieve overloaded members and ensure better seismic behaviour.

5.10.1 Introduction of Joints

The buildings having large aspect ratio (length-to-breadth ratio) or projections in plan, behave badly during earthquakes. Such buildings can be separated into blocks with a crumple section in between them (IS 4326: 1993). New walls are inserted and tied with the existing walls to avoid long length of walls and to reduce the distance between centre of mass and centre of rigidity for each block (Figure 5.21).

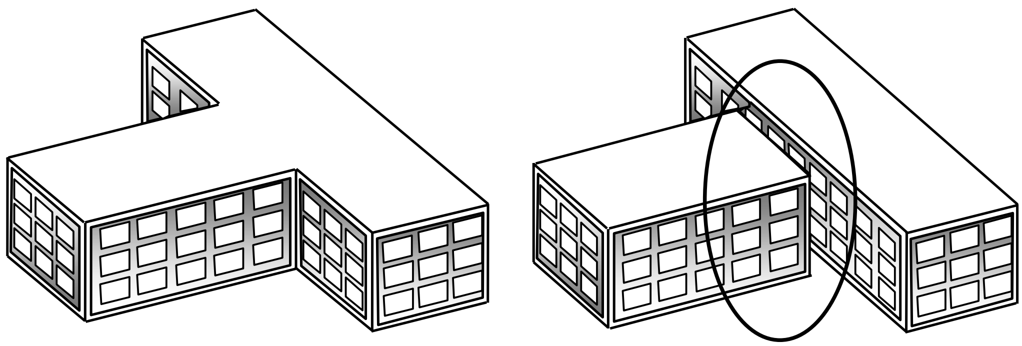


Figure 5.21 Separation of blocks by introducing joints

5.10.2 Introduction of Walls

Long walls are vulnerable to out-of-plane collapse. For a wall of thickness *t* and cross wall spacing *a*, the ratio *a*/*t* should not exceed 40. To ensure this cross walls can be added as indicated in Figure 5.22. The spaces can be utilised appropriately without significantly altering the functionality of the building. This type of intervention is possible in long barrack type buildings used for schools, dormitories or other purpose. The junction between the new wall and the old wall will be a T-junction. Figure 5.23 and Figure 5.24 show the typical details to be adopted for tying the new and old walls. Some details of the tying reinforcement are given in IS 13935: 1993.

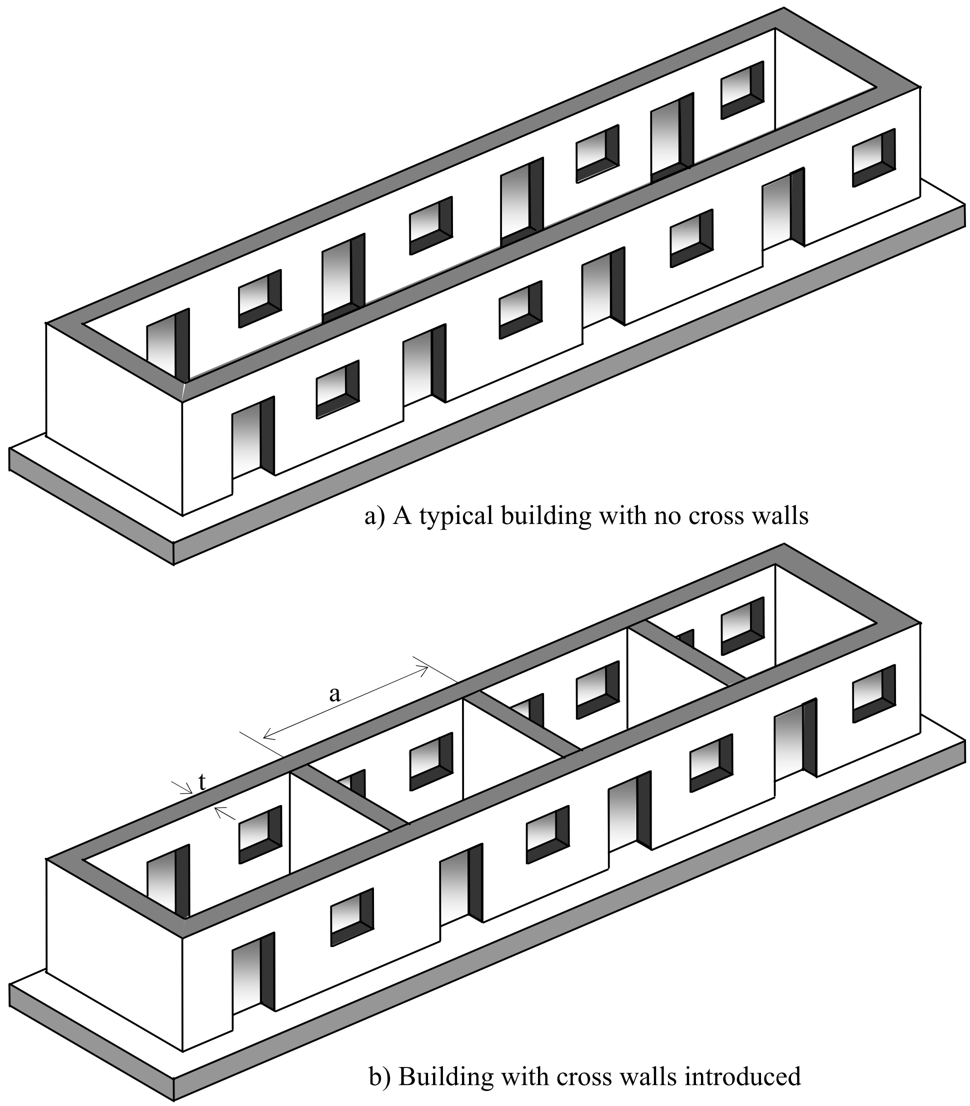
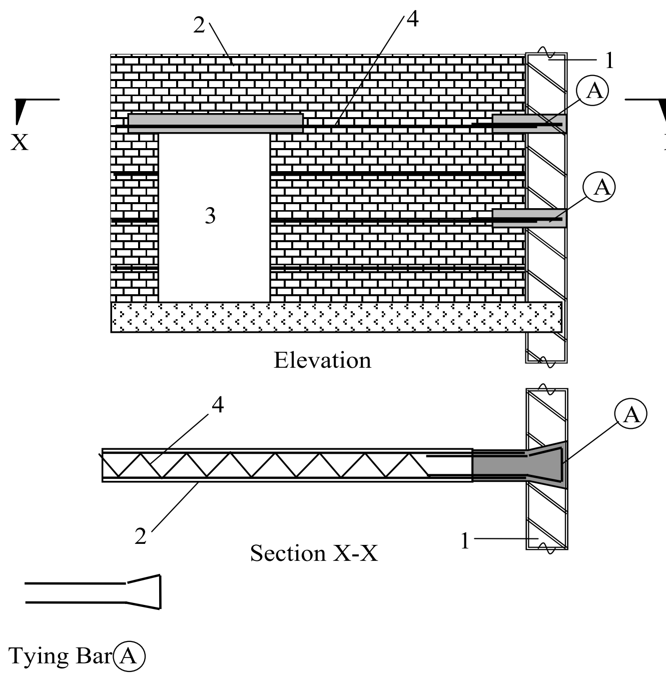
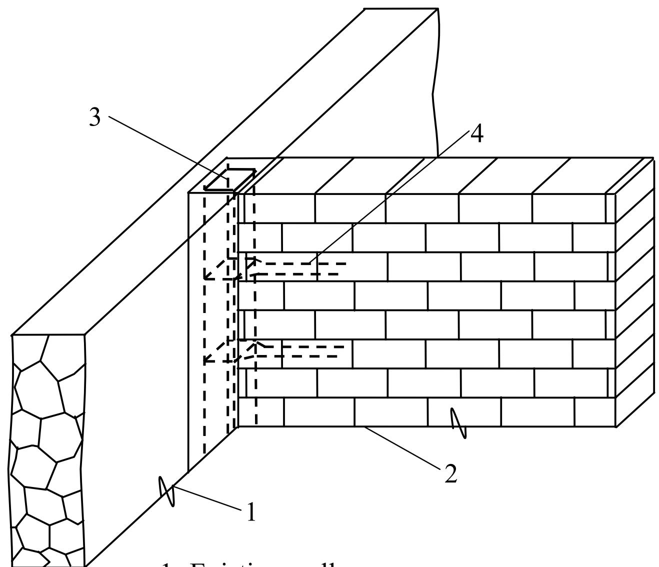


Figure 5.22 Introduction of cross walls



1- Existing wall
2- New wall
3- Door opening
4- Reinforcement in new wall (example of truss system shown)

Figure 5.23 Connection of a new brick wall with an existing brick wall

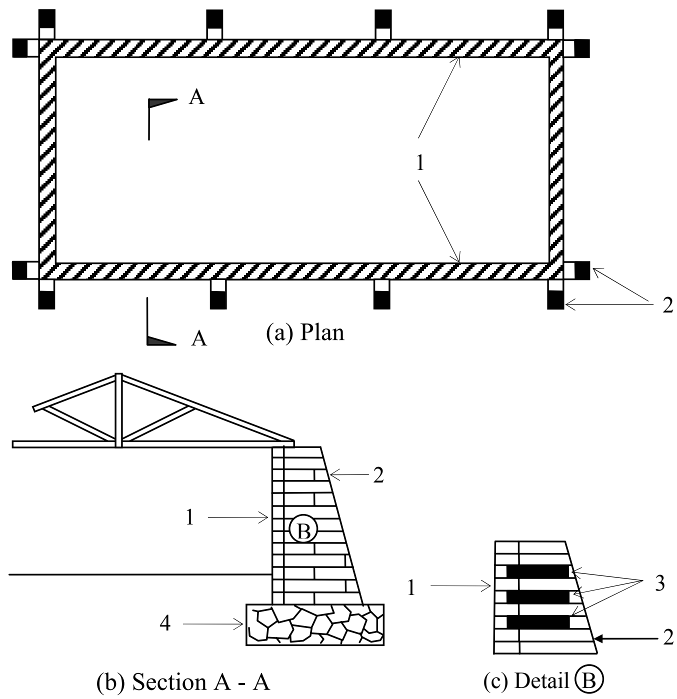


1- Existing wall
2- New wall
3- Concrete column
4- Tying bars, every fourth course

Figure 5.24 Connection of a new brick wall with an existing stone wall

5.10.3 Introduction of Pilasters or Buttresses

In the case of longitudinal walls of the barrack type buildings, pilasters or buttress walls added externally will enhance lateral load resistance (Figure 5.25). The buttresses need to be integrated with the existing wall by key stones (IS 13935: 1993).



1 Existing wall
2 Buttress or pilaster
3 Key stones
4 Foundation

Figure 5.25 Strengthening of long walls by buttresses

5.10.4 Improvement of the Frame

Most of the non-engineered constructions do not have adequate lateral load resistance. The rural hut made with casuarina and bamboo posts is a typical example. Two modes of collapse of the frame are shown in Figure 5.26. Figure 5.27 shows how the introduction of braces in the periphery stabilizes the frame. Provision of doors can be accommodated by providing braces on either side as shown in the second sketch of the same figure. There can be cross braces or knee braces in the vertical planes of the walls or in the horizontal plane at the top of the walls, as shown in Figure 5.28.

Instead of using ropes for tying the rafters and braces, metal straps and mild steel wires of suitable gauge will make the system behave better. The connection details are shown in Figure

5.29. The vertical post should be properly anchored with cross sticks tied firmly below ground, as shown in Figure 5.30. This provides good anchorage for the post. The portion of the post below ground level should be protected by tar coat.

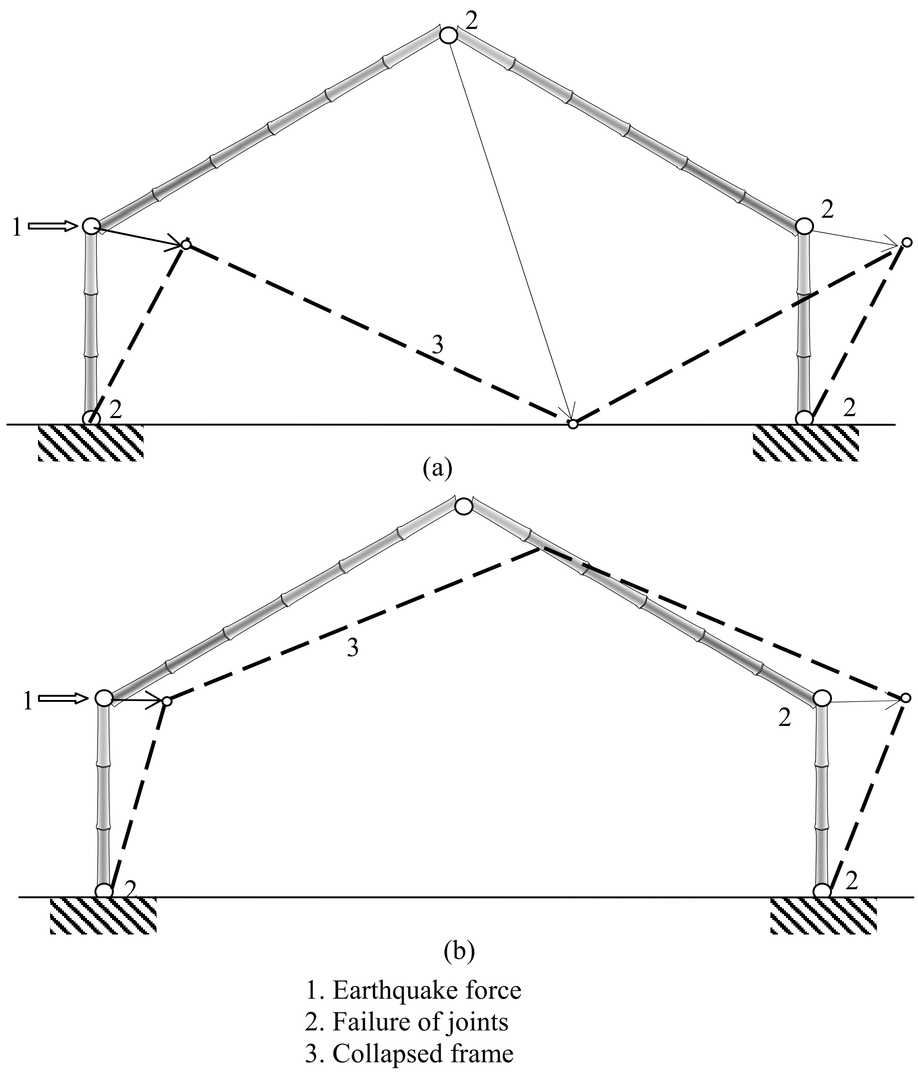


Figure 5.26 Collapse modes of a typical hut

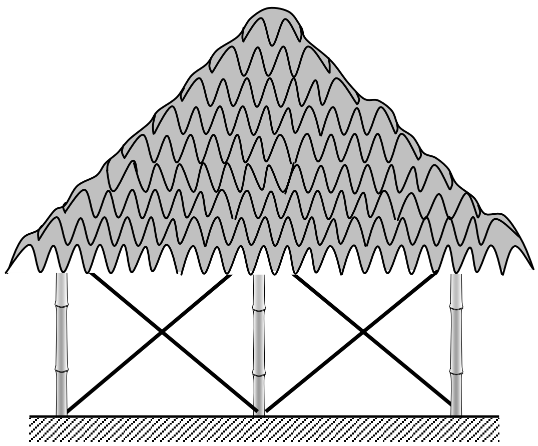


Figure 5.27 Introduction of braces in a hut

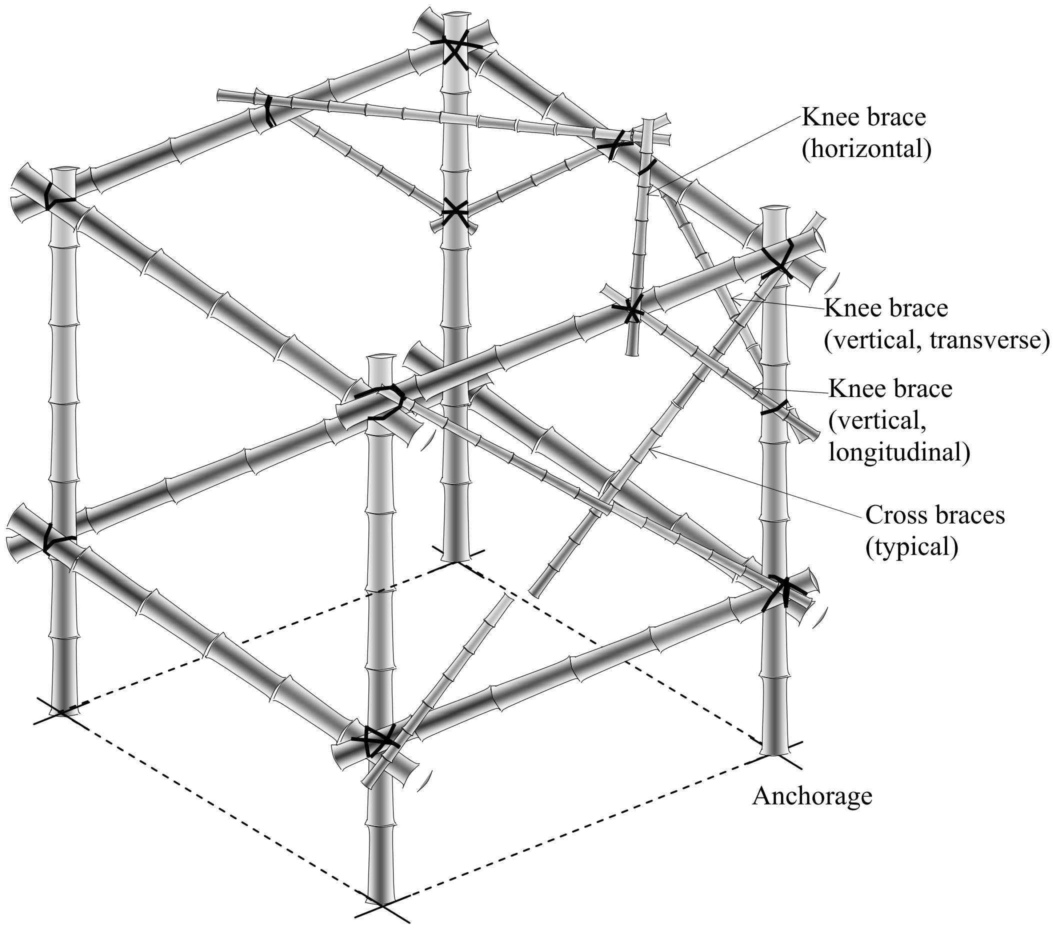


Figure 5.28 Introduction of braces

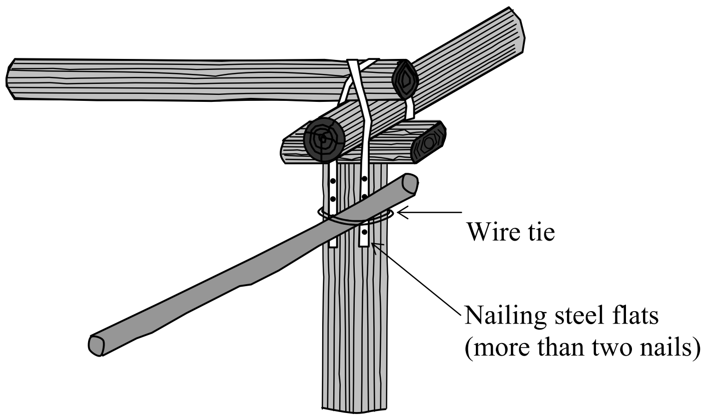
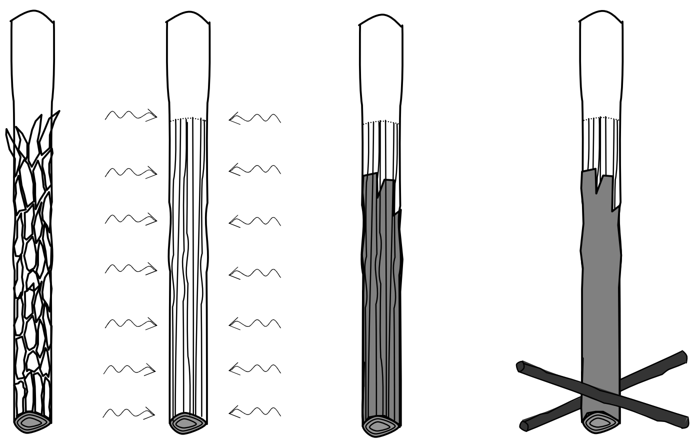


Figure 5.29 Connection details



(a) (b) (c) (d)

(a) Remove bark completely
(b) Apply heat at the bottom so that there is a layer of carbon
(c) Apply tar for portions being embedded in soil, including cross sticks
(d) Tie two sticks on either side of trunk

Figure 5.30 Improvement of wooden posts

5.10.5 Splint and Bandage Strengthening Technique

To economise in the retrofit cost while targeting to enhance the integrity of the building, ferro-cement with galvanised welded steel wire mesh can be added on the exterior of the walls in the form of vertical splints adjacent to openings and horizontal bandages over the openings, as shown in Figure 5.31.

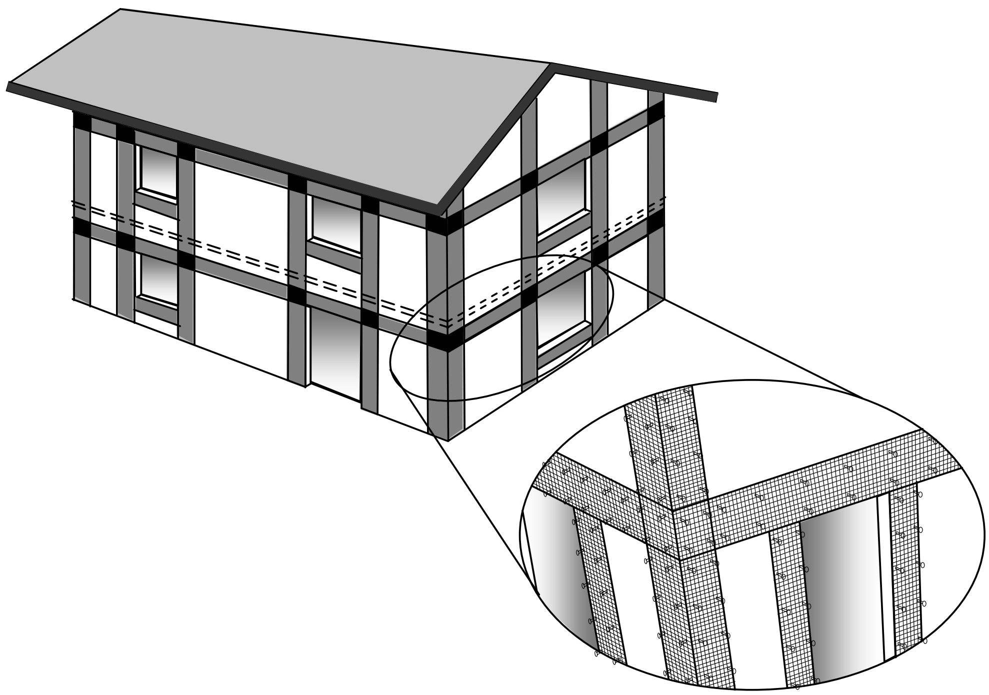


Figure 5.31 Splint and bandage strengthening technique

5.11 SUMMARY

This chapter summarises the various methods of retrofitting non-engineered buildings. First, the preferred seismic resistant features of such buildings are explained. Next, a brief description of various repair materials and techniques is provided. The strengthening methods for roofs, upstairs floors, walls and pillars are described. The techniques for global strengthening cover the introduction of joints, walls, pilasters, buttresses and braces, improvements of the frame and splint and bandage strengthening technique.

The techniques covered in this chapter are intended to provide various options for retrofitting non-engineered buildings. However, given the wide variety of non-engineered buildings, differences in the detailing for each technique are expected to be based on the local condition and availability of materials, skills and funds. Hence, importance should always be placed on the detailing while designing the most appropriate repair and retrofit scheme for a building.

5.12 REFERENCES

1. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings – Code of Practice", Bureau of Indian Standards.

e) IS 13935: 1993, "Indian Standard for Repair and Seismic Strengthening of Buildings – Guidelines", Bureau of Indian Standards.

f) IS 13827: 1993, "Improving Earthquake Resistance of Earthen Buildings – Guidelines", Bureau of Indian Standards.

g) IS 13828: 1993, "Improving Earthquake Resistance of Low strength Masonry Buildings – Guidelines", Bureau of Indian Standards.

h) "Guidelines for Earthquake Resistant Non-engineered Construction", (2004), Published by the International Association for Earthquake Engineering and National Information Centre of Earthquake Engineering.

i) "Guidelines for Reconstruction of the Houses affected by Tsunami in Tamil Nadu", (2005), Published by the Government of Tamil Nadu.

j) Madhava Rao, A. H. and Ramachandra Murthy, D.S. (1999), "Appropriate Technologies for Low Cost Housing", Oxford & IBH Publishing Co. Pvt. Ltd.

k) "Technologies for Retrofitting of Existing Buildings and Structures to make them Earthquake Resistant", (2003), sponsored by TIFAC, Department of Earthquake Engineering, IIT Roorkee.

6

RETROFIT OF MASONRY BUILDINGS

6.1 OVERVIEW

Masonry buildings refer to those with load bearing walls made of fired clay bricks, stone blocks or concrete masonry units. The buildings that are covered in this chapter are expected to be adequately designed for gravity loads. The buildings which are informally constructed in the traditional manner are covered in Chapter 5, Retrofit of Non-engineered Buildings. The masonry buildings are vulnerable to seismic loads because of their relatively high mass and lack of ductility. This chapter deals with the methods of strengthening of masonry buildings. The information on the available codes of practice is provided.

Any building first requires an evaluation of the existing condition. The chapter provides the procedure of seismic analysis specific to masonry buildings. The method of piers, the model for the analysis and the essential equations are explained.

The common defects observed in masonry buildings and the deficiencies in resisting seismic forces are listed. It is essential to diagnose the deficiencies in a building before undertaking retrofit. Also, the highlighting of deficiencies is expected to create awareness for future construction. Next, the repair and retrofit techniques are presented. Some of the repair materials and techniques covered in Chapter 5 are also applicable for masonry buildings. These are not repeated in this chapter. The retrofit techniques are grouped under strengthening of members and global techniques. The strengthening of members cover roofs, floors, walls and pillars. The global retrofit techniques cover introduction of frames and braces and strengthening by post-tensioning. The importance of maintenance after undertaking retrofit cannot be overemphasised. Hence, the primary actions for maintenance are highlighted. Finally, a case of implementation of retrofit is illustrated.

6.2 INTRODUCTION

Many masonry buildings made of fired clay bricks, stone blocks or concrete masonry units, fall under the category of non-engineered buildings, which are informally constructed in the traditional manner without formal design by qualified engineers or architects. The strengthening of such masonry buildings is covered in Chapter 5, Retrofit of Non-engineered Buildings. Some buildings can be considered to be semi-engineered. Although they were designed for gravity loads, their resistance to seismic forces was ignored. These buildings require strengthening for seismic forces. The engineered buildings which were designed for seismic forces may also require strengthening. This chapter covers the retrofit of semi-engineered and engineered masonry buildings. Emphasis is placed on the analysis for seismic forces.

The existing masonry buildings are mostly un-reinforced, that is, they do not have embedded reinforcing bars. The vulnerability of un-reinforced masonry to seismic forces arises due to its very low tensile and shear strengths, lack of ductility and energy absorbing capacity. The following publications of the Bureau of Indian Standards provide the basic guidelines for the design of masonry buildings and incorporation of seismic resistant features.

1. IS 1905: 1987, "Code of Practice for Structural Use of Un-reinforced Masonry"
2. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings – Code of Practice"
3. IS 13828: 1993, "Improving Earthquake Resistance of Low Strength Masonry Buildings – Guidelines"
4. IS 13935: 1993, "Indian Standard for Repair and Seismic Strengthening of Buildings – Guidelines".

While undertaking retrofit, it is important to comply with the stipulations of the above codes. The interventions during retrofit can also improve the architectural appearance, acoustics, thermal insulation and fire resistance.

6.3 SEISMIC ANALYSIS

The seismic analysis of a masonry building can be done based on the method of piers or based on the more sophisticated finite element analysis. Here, only the method of piers is explained. The reader may refer to specialised literature for advanced modelling and assessment of masonry buildings.

> In a pier analysis, the building model constitutes of vertical wall piers resisting the seismic forces along a direction, predominantly by shear. For the wall piers, only the resistance to in-plane lateral loads is considered. Any resistance to out-of-plane lateral loads is neglected. The portions of the walls above the doors, windows and arch openings are neglected.

In a pier analysis, the roofs and floor slabs made of concrete or concrete over wooden rafters can be assumed to act as rigid diaphragms, which mobilise the wall piers (subsequently referred to as walls) to act together like parallel springs. The seismic forces get distributed as per the lateral stiffness of each wall. The walls can be considered rotationally fixed at the diaphragms. The roofs and floors made of wooden planks can be assumed to act as flexible diaphragms. In this case the walls act independently like individual springs. The seismic forces get distributed as per the tributary area of each wall. The walls can be considered to be rotationally free (pinned) at the diaphragms.

The end condition at the bottom of a ground storey wall depends on the type of foundation. If there is no spread wall foundation or if the spread foundation does not have any dowels, such as in an un-reinforced masonry building, the bottom of the wall can be assumed to be pinned. If there are dowels between the foundation and the wall, such as in reinforced masonry buildings, the bottom can be assumed to be fixed. Figure 6.1 shows a schematic representation of the model for pier analysis along a direction.

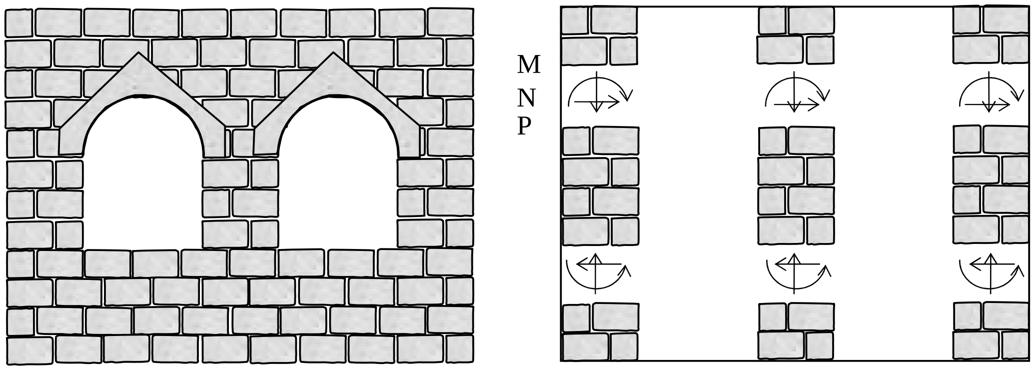


Figure 6.1 Model for a pier analysis

In the analysis, first the seismic weight of the building is calculated from the dead loads and live loads at the roof and upstairs floors. Next, the base shear and its distribution along the height are determined. The storey shears are calculated from the applied lateral forces. The effect of torsion in a floor is evaluated based on the eccentricity of the centre of mass with respect to the centre of rigidity. These are explained in Chapter 8, Structural Analysis for Seismic Retrofit. Finally, the stresses in each wall is determined and compared with the allowable values.

6.3.1 Calculation of Stiffness of a Wall Pier

Due to an in-plane lateral load, a wall deflects as a result of both bending deformation and shear deformation. The total deformation (δ) due to a unit load is given as follows.

For a wall fixed at one end and free at the other end

$$\delta = \left(\frac{H^3}{3EI}\right) + \left(\frac{1.2H}{GA}\right) \quad \textbf{(6.1)}$$

For a wall fixed at both the ends

$$\delta = \left(\frac{H^3}{12EI}\right) + \left(\frac{1.2H}{GA}\right) \quad \textbf{(6.2)}$$

Here,

A = cross-sectional area of the wall

E = elastic modulus for the wall
G = shear modulus for the wall
= $E/2(1 + \nu)$
H = height of the wall
I = moment of inertia of the wall
ν = Poisson's ratio for the wall.

The expression of lateral stiffness (k) is the inverse of the deflection.

6.3.2 Calculation of Stresses in a Wall Pier

The axial compressive stress (f_a) in the j^{th} wall can be determined from the load (N_j) from the tributary areas of the roof and supported floors. The bending and shear stresses are determined from the moment (M_j) and shear force (V_j), respectively. The moment is related to the shear force based on the fixity conditions at the top and bottom. For a wall fixed at both the ends, $M_j = V_jH/2$, where, H is the height of the wall.

The shear force in a wall (V_j) in a storey consists of two components. One is due to the storey shear and the other is due to the storey torsion. The storey shear is the sum of the lateral loads acting on all the floors above. The storey torsion is the sum of the torques acting on all the floors above. The torque in a floor is the product of the lateral load and eccentricity of the design centre of mass and centre of rigidity. The shear force (V_{Dj}) in the j^{th} wall of the i^{th} storey due to storey shear is given as follows.

$$V_{Dj} = V_i \frac{k_j}{\sum_1^n k_j} \tag{6.3}$$

Here,
V_i = storey shear in the i^{th} storey
k_j = stiffness of the j^{th} wall
n = number of walls along the direction of the shear.

The shear force (V_{Tj}) in the j^{th} wall of the i^{th} storey due to storey torsion is given by the following equation.

$$V_{Tj} = T_i \frac{k_j r_j}{\sum_{j=1}^n k_j r_j^2} \tag{6.4}$$

Here,

T_i = storey torsion in the i^{th} storey
r_j = radial distance of the j^{th} wall measured from the centre of rigidity.

6.3.3 Calculation of Allowable Stresses

The allowable stresses of the walls are calculated as per IS 1905: 1987. First, the basic compressive stress for masonry (f_{bc}) is evaluated considering the crushing strength of bricks and the type of mortar (Table 8). Next, the stress is modified by the reduction factor for slenderness (k_s) (Clause 5.4.1.1), the area reduction factor for small wall (k_a) (Clause 5.4.1.2) and the shape modification factor (k_p) (Clause 5.4.1.3). Thus, the allowable stress under direct compression is given as $F_a = k_s\, k_a\, k_p f_{bc}$. The allowable compressive stress under bending can be taken as F_b = 1.25 F_a.

When both direct compressive stress (f_a) and flexural compressive stress (f_b) act, the following interaction formula must be satisfied.

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \le 1.0 \tag{6.5}$$

Finally, the shear demand-to-capacity ratio for a wall can be calculated from the shear stress acting (demand) and the allowable shear stress (capacity). The ratio should be less than 1.0. As per IS 1905: 1987, the allowable shear stress is the least of a) 0.5 MPa, b) 0.1+0.2 f_a and c) 0.125 $\sqrt{f_m}$. Here, f_a is the direct compressive stress in the wall and f_m is the crushing strength of the bricks.

6.4 BUILDING DEFICIENCIES

6.4.1 General Defects

The typical problems that are frequently encountered due to the deterioration of existing masonry buildings are as follows.

1. Cracking
2. Spalling
3. Staining
4. Moisture ingress
5. Deterioration of mortar and loose components

6. Corrosion
7. Differential settlement of the foundation
8. Blistering of coating
9. Design and construction defects.

Most of these defects are due to improper construction, lack of quality control during construction, over-emphasis on reducing cost at the expense of durability and safety, and lack of maintenance. Many of the problems listed above occur even in relatively new buildings, which require repair within 5 years after construction. Of late, even buildings which have survived for long periods have shown problems because of changes in the environment, such as industrial and traffic pollution. A few of the above problems are elaborated here.

<u>Cracking</u>

Cracking is the most common visually detectable distress encountered in a building, needing repair or retrofit. The cracking may be minor such as those due to restraint to shrinkage. Else, the cracking may be major due to any one of the following causes.

a) Over-loading
b) Differential settlement of the foundation
c) Thermal movement
d) Load transfer from beams and columns in a framed building
e) Vibration
f) Corrosion of reinforcing bars in a reinforced masonry building.

Cracks of smaller width are of aesthetic concern and hence, need cosmetic treatment. Cracks of width greater than 1 to 2 mm signify structural problems. Proper location of expansion joints can avoid cracks due to thermal movement. It is necessary to classify whether a crack is active or dormant. Active cracks propagate and hence, separate inspections of the same crack can reveal if the crack is active or dormant. The cracks may be horizontal, vertical, diagonal or stepped depending upon the cause and the relative strength of the masonry units and the mortar. Figure 6.2 shows examples of cracks witnessed in masonry buildings.

Figure 6.2a Horizontal crack

Figure 6.2b Vertical crack

Spalling

The delamination of surface of brick or mortar or plaster is called spalling. Spalling can occur due to internal stresses or due to external actions. Concentrated eccentric load causes highly stressed narrow compression zone which encourages spalling. Spalling also occurs due to freeze-thaw effect of entrapped water, chemical effect, efflorescence and repeated wetting and drying in coastal areas (Figure 6.3). Observation of the location of spall gives an indication of the cause.

Figure 6.3 Deterioration of brick masonry due to spalling

Staining

Staining of masonry walls is caused by absorption of water containing salts and subsequent efflorescence. Efflorescence is defined as the deposition of water soluble salts on the surface after evaporation of the water. For efflorescence to occur, there should be a source of water and water soluble salts. The efflorescence can disrupt the wall because of internal crystallization of salts.

In reinforced masonry walls, rust staining may occur due to absorption of water. Because of increase in volume due to the formation of rust, spalling and cracking occur. If unattended, it can lead to faster corrosion of the steel bars and deterioration of the wall. Thus, to check corrosion and efflorescence staining, the problem of absorption of water has to be addressed.

Moisture ingress

The moisture ingress depends on several factors such as the porosity of the bricks, the mortar joints, the pointing, cracks in the wall and the plastering. Water seepage causes wetness and encourages the growth of mould, fungi and vegetation. It can degrade the quality of the wall. Penetration of rain water and its pathways can be detected through visual observation of wet areas and patches following rainy days.

Differential settlement of the foundation

If parts of the ground are made of fill or are susceptible to consolidation or swelling, differential settlement of the wall foundation can cause cracks. The cracks tend to widen with time.

Construction defects

The following problems are primarily due to deficient construction and defective workmanship.

1. Use of poor quality bricks
2. Use of poor construction procedure, such as not soaking the bricks before construction
3. Defective bond and flashing
4. Use of incorrect wall thickness
5. Out-of-plumb wall
6. Defective and misaligned joints of walls
7. Lack of movement joints for expansion and contraction
8. Plugged weep holes.

Most of these defects can be detected by visual inspection.

6.4.2 Deficiencies in Seismic Resistance

A typical small masonry building is a box system with multiple openings of doors and windows. A rigid roof slab transfers the seismic load to the various walls. The deformation and typical damage to the walls due to the load are shown in Fig. 6.4. The behaviour depends on how the walls are interconnected and anchored with each other and with the roof. The walls perpendicular to the load are subjected to out-of-plane bending, causing vertical cracks at the middle. The walls in the direction of the load develop diagonal cracks due to shear forces. The cracks form X-patterns due to the reversible nature of the seismic load.

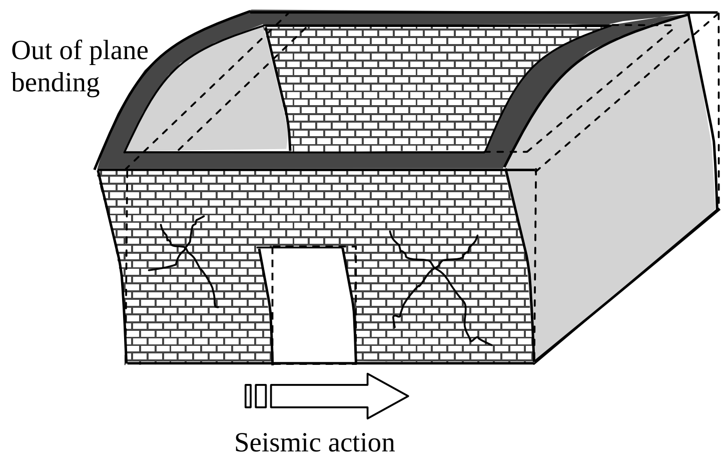


Figure 6.4 Deformation and damages to walls in box type building

The deficiencies of masonry buildings for resisting seismic load are listed below.

1. Absence of horizontal bands of reinforcement at different levels such as plinth band, sill band, lintel band and roof band.
2. Non-existent or improper connections between walls, roof and floor.
3. Large openings resulting in reduced wall stiffness or storey stiffness.
4. Out-of-plane failure of walls due to lack of cross walls.
5. Roof slabs at different levels or large opening in the roof slab (the opening may be covered by jack arch roof), leading to poor diaphragm action. There is absence of reinforcement around the openings and the edges of the slabs.
6. Plan asymmetry and eccentric mass from water tanks causing torsion.
7. Re-entrant corners in U-, L- and T-shaped buildings, without adequate reinforcement in the slab at the corners.

8. Inadequate anchorage of parapet walls and sunshades.
9. Inadequate gap between buildings or across expansion joints, which can lead to pounding.
10. Inadequate foundation.

The above deficiencies lead to inadequate lateral strength, stiffness and structural integrity.

6.5 REPAIR TECHNIQUES

If the stress demand exceeds the capacity of the walls, retrofit is required. The retrofit includes repair of existing defects and increasing the lateral strength, stiffness and structural integrity. Some of the repair materials and techniques explained in Chapter 5, Retrofit of Non-engineered Buildings, are also applicable for masonry buildings. The retrofit techniques can be broadly classified as local (those for improving the strength of a member) and global (those for improving the performance of the building). The repair and local retrofit techniques specific to masonry buildings are first described.

6.5.1 Repointing

For repointing, first the wall should be made wet and all loose debris cleared. The joints that are to be repointed should be raked to a depth of 2 times the joint height. Next, fresh mortar should be placed by trowels. The mortar should be non-shrinking type. The repointed portion should be cured properly.

6.5.2 Grout Injection

Grout can be injected to strengthen the walls. The method is covered in Chapter 5. The strengthening is useful for historical buildings. The grout mixture should be chosen based on the following requirements.

1. High water retention
2. Minimum or no shrinkage, preferable to have slight expansion.
3. Highly flowable without any segregation
4. High tensile strength
5. Good bond with the existing masonry.

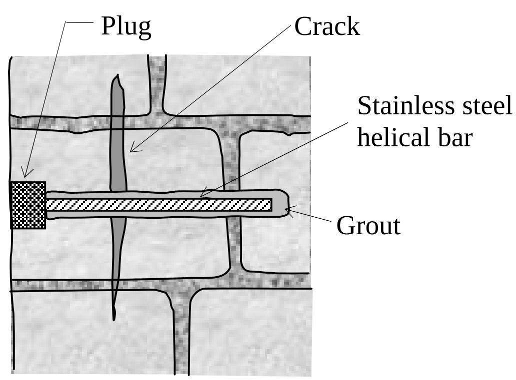
After grout injection, the improvement in properties may be assessed using non-destructive tests. Small cores can be made through the thickness of a wall and the flow of grout can be assessed.

6.5.3 Grout Filling

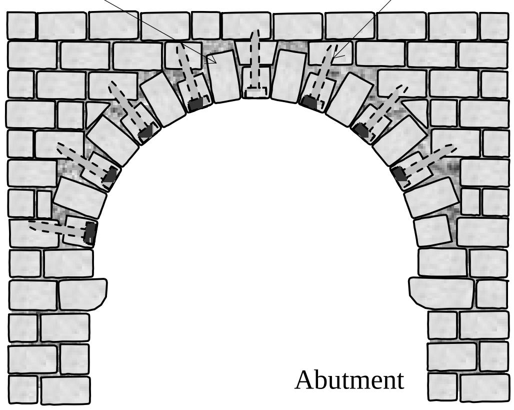
Selected cells in a hollow block masonry wall can be filled with grout. Filling the voids with grout will increase the compressive strength and make the wall more impermeable to water penetration. The inside of the cavity should be pre-wetted, then drained prior to grouting.

6.5.4 Crack Stitching

It is possible to introduce internal ties in a masonry wall by drilling a hole, placing a bar and finally grouting the hole. A similar ‘pinning technique’ can be used for stitching cracks in the walls and strengthening the arches (Figure 6.5).



a) Crack stitching



b) Strengthening a masonry arch

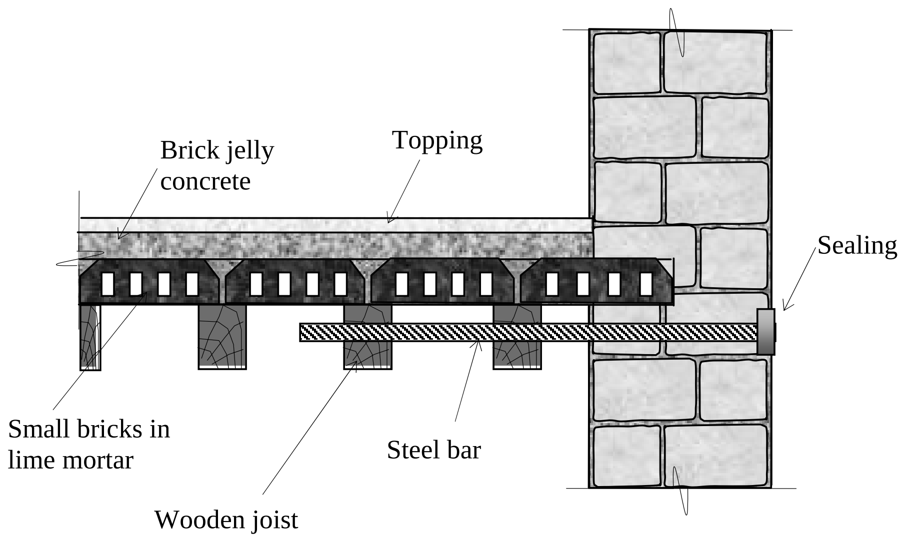
Figure 6.5 Pinning technique

6.6 STRENGTHENING OF ROOFS AND UPSTAIRS FLOORS

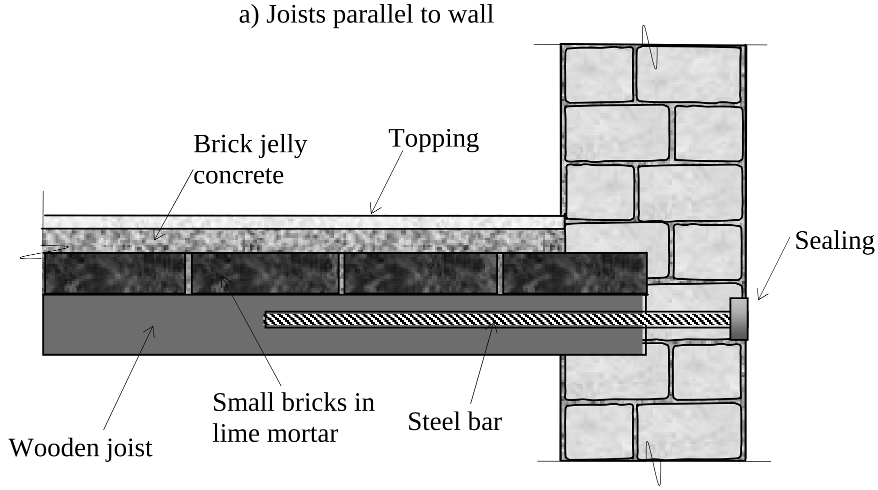
Strengthening of roofs and floors involves two issues. First, the integrity of the slab has to be improved by providing adequate thickness or braces for the in-plane shear capacity. Second, the connection between the slab and the walls has to be improved so that the seismic load can be transferred to the walls. The strengthening of two typical types of roof is elaborated.

6.6.1 Madras Terrace Slab

Madras terrace slab exists in old masonry buildings as roof or upstairs floors. It is generally constructed of wooden joists spanning between the walls. Small size bricks are laid across the joists in a brick-on-edge fashion and are then topped with a layer of lime or cement mortar. Brick jelly concrete and topping (tiles or floor finish) are laid subsequently. In order to improve the diaphragm action, the pinning technique can be adopted as shown in Figure 6.6. The steel bars (pins) ensure integration of the walls with the roof (or floor) for the transfer of seismic load.



a) Joists parallel to wall



b) Joist perpendicular to wall

Figure 6.6 Pinning technique for Madras terrace slab

6.6.2 Jack Arch Roof

Jack arch roofs are common in old masonry buildings for spanning larger distance between walls. Tie rods can be provided between the springing of the arches (Figure 6.7). This will relieve the walls from the thrust of the arches and the load transferred to the walls will be vertical.

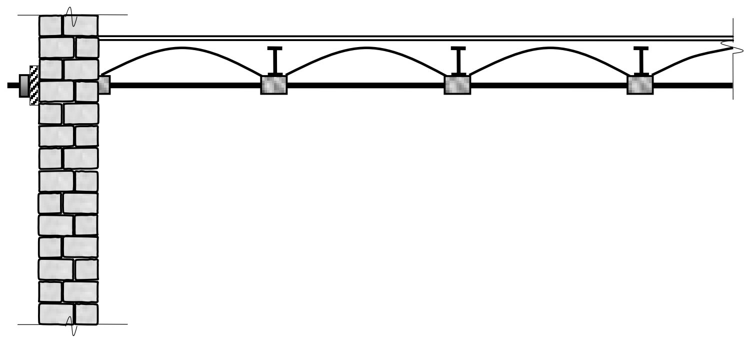


Figure 6.7 Strengthening of jack arch roof by ties

6.7 STRENGTHENING OF WALLS

The walls support the roof and upstairs floor, as well as provide resistance to lateral load. The walls can be strengthened by the following methods.

(a) Grouting
(b) External reinforcement
(c) Use of steel plates
(d) Use of fibre reinforced polymer
(e) Internal reinforcement.

The method of grouting is discussed under repair techniques and in Chapter 5. The other four methods are described next. Besides these, the anchoring of the walls and reinforcing the openings are also presented.

6.7.1 External Reinforcement

An external reinforcement overlay is an effective strengthening technique for existing masonry walls. The overlay can be done by one of the following methods.

1. Ferro-cement with wire mesh
2. Reinforcement mat with shotcrete

In ferro-cement strengthening, first the plaster is removed and mesh reinforcement (welded wire fabric with a mesh size of 50 mm × 50 mm) is placed on one or both sides of the wall (Figure 6.8). The mesh reinforcement is tied in place by steel wires at 500 mm to 700 mm interval and the holes are grouted. Finally, a 25 to 40 mm thick layer of micro-concrete or plaster is applied on the reinforcement. The sandwiched wall is expected to behave better during an earthquake (IS 13935: 1993).

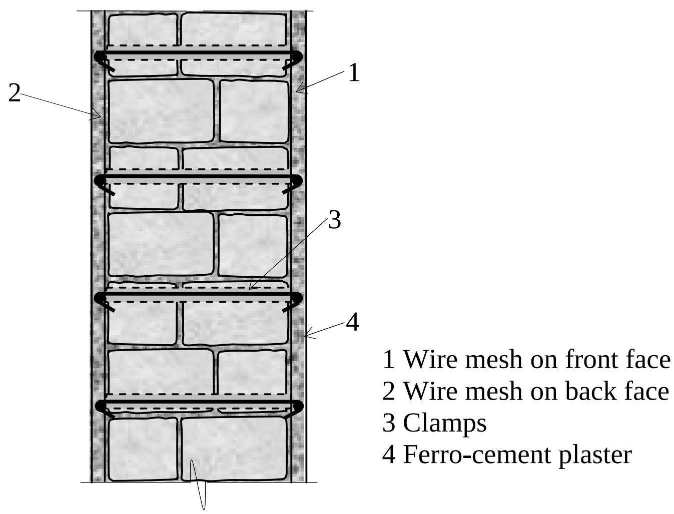


Figure 6.8 Strengthening of wall with ferro-cement

For substantial strengthening, a reinforcement mat can be attached to the wall with dowels. The vertical bars are anchored in the foundation. The reinforcement can be designed for adequate in-plane shear capacity and out-of-plane bending capacity of the wall. Shotcrete can be applied over the reinforcement mat. The method of shotcrete is briefly explained in Chapter 5.

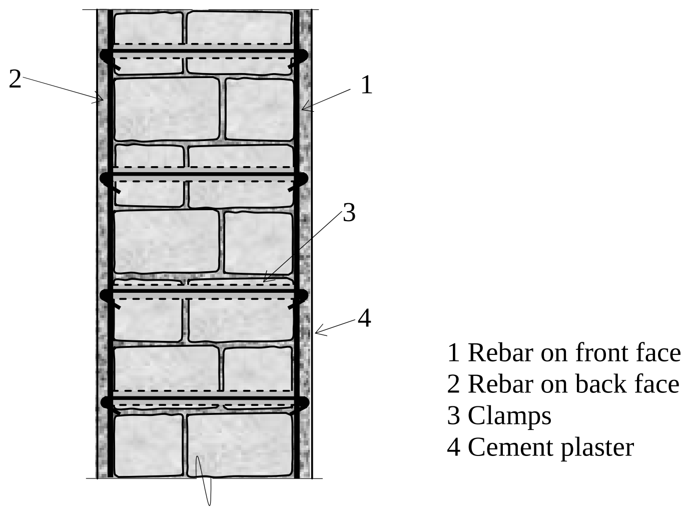


Figure 6.9 Strengthening of wall with reinforcement mat

6.7.2 Use of Steel Plates

Steel plates or angles can be used to strengthen masonry walls. A typical strengthening strategy is shown in Figure 6. 10.

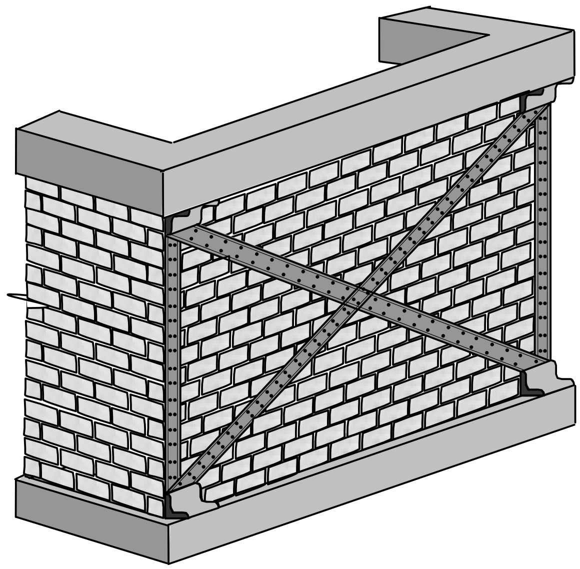


Figure 6.10 Strengthening of wall with steel plates

6.7.3 Use of Fibre Reinforced Polymer

Fibre reinforced polymer (FRP) composites are tailor-made, flexible, easy to apply and can be made architecturally pleasing. Hence, they can be extensively used in retrofit. Three types of strengthening techniques namely, surface mounting of FRP bars, surface mounting of FRP strips and overlay of FRP wraps, can be applied to enhance the out-of-plane bending strength of a wall.

Surface mounting of FRP bars

In this technique grooves are cut through mortar bed joints horizontally and vertically if needed, about 25 mm deep. The grooves are half filled with epoxy adhesive. Small diameter (about 6 mm) FRP bars are inserted and covered with another layer of adhesive (Figure 6.11).

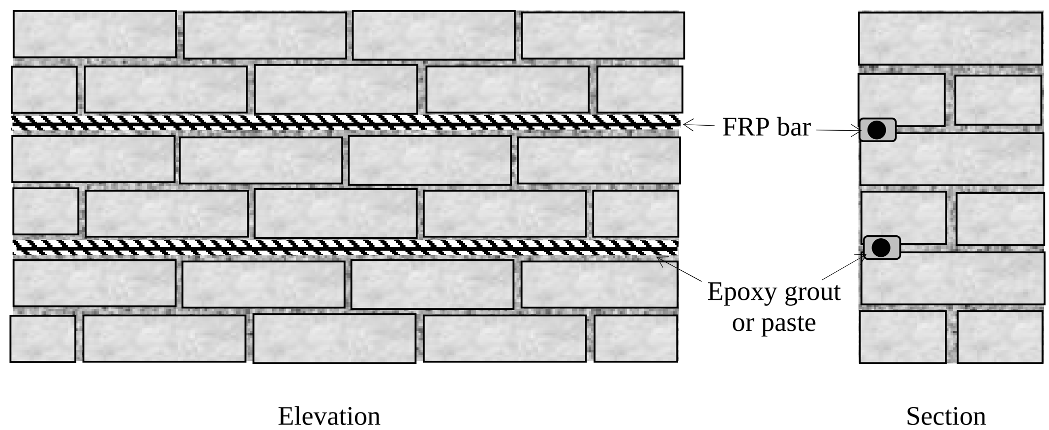


Figure 6.11 Surface mounting of FRP bars

Surface mounting of FRP strips

FRP strips are applied to walls vertically or diagonally to improve their out-of-plane bending capacities. The diagonal strip application enhances the in-plane shear capacity by acting similar to a tension chord of a diagonal brace.

Overlay of FRP wraps

This technique involves applying a layer of epoxy to the surface of the wall, impregnating the FRP wrap with epoxy, placing the overlay on the wall surface, and finally finishing with another layer of epoxy. The bond is a key factor that determines the success of an FRP overlay. An undesirable mode of failure is peeling off due to insufficient anchorage. Figure 6.12 shows a typical anchorage for an FRP wrap by embedded FRP bars.

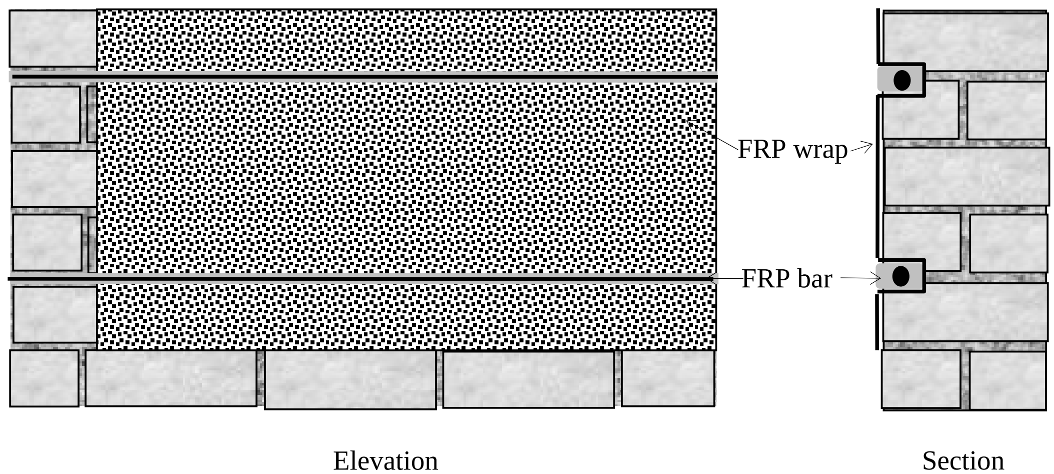


Figure 6.12 Anchoring of FRP wrap using FRP bars

6.7.4 Internal Reinforcement

Reinforcing bars can be placed inside a masonry wall to maintain its exterior appearance. A hole is drilled from the top of the wall up to the foundation. For hollow block masonry units, the drilling can be aligned with the space in the units. Reinforcing bars are placed and the hole is grouted. The grout covers the voids around the hole. The reinforcement can be designed for adequate in-plane shear capacity and out-of-plane bending capacity of the wall.

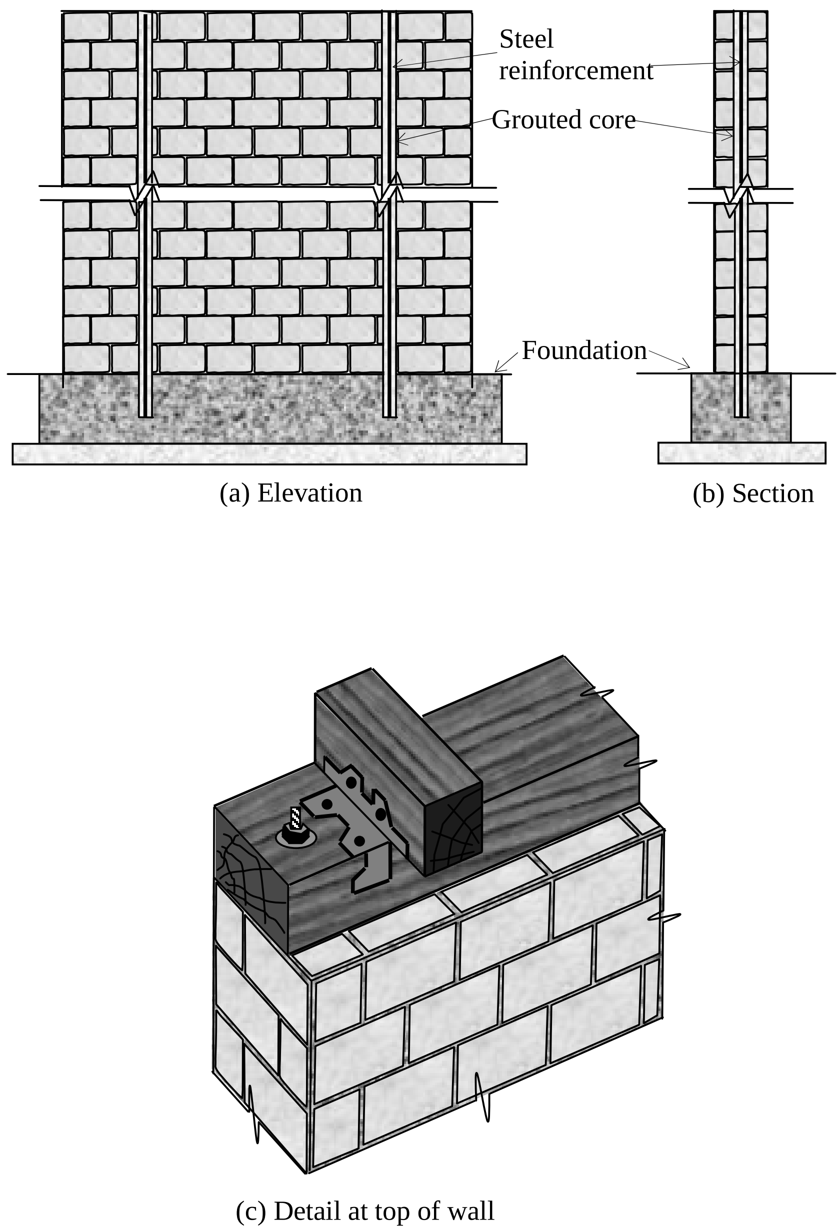


Figure 6.13 Strengthening of wall with internal reinforcement

6.7.5 Anchoring the Walls

Anchoring a wall to the roof, floors and other walls enhances the box action. The various types of wall connections in a multi-storeyed masonry building are shown in Figure 6.14. To ensure the lateral support of the wall at the top, the anchorage to the roof is essential. The schemes shown under strengthening of Madras terrace slab provides anchorage to the walls. A few other schemes of anchorage are shown here.

The anchors can be external or internal. Internal anchors can be used in situations were external anchors are not acceptable for either aesthetic or functional reasons. Figure 6.15 shows the use of external anchors such as steel angles. Parapet walls collapse during earthquake if they are not anchored properly. For a partition wall that is not designed for load bearing, a gap is to be maintained above the wall and slotted angles can be provided for anchorage.

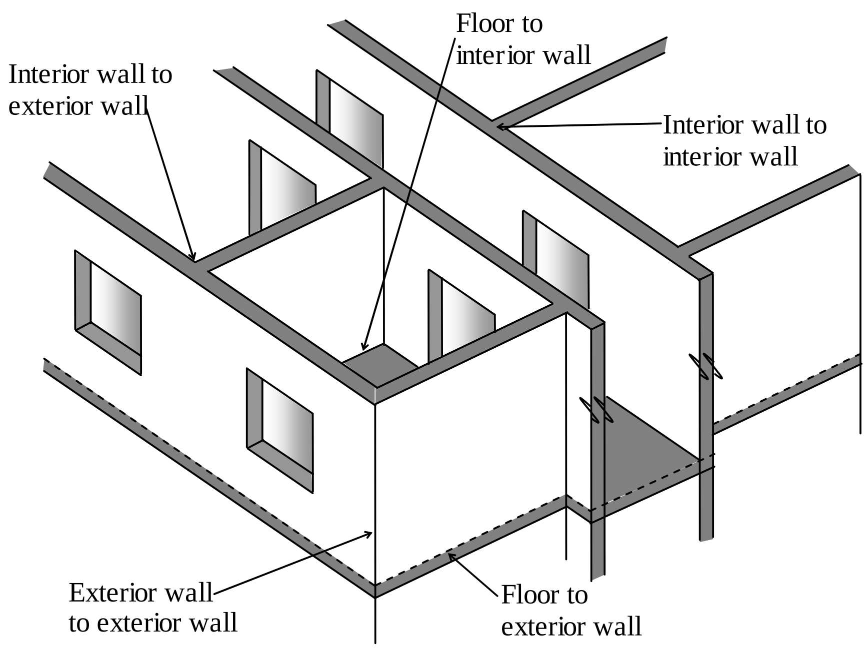


Figure 6.14 Various wall connections in a multi-storeyed masonry building

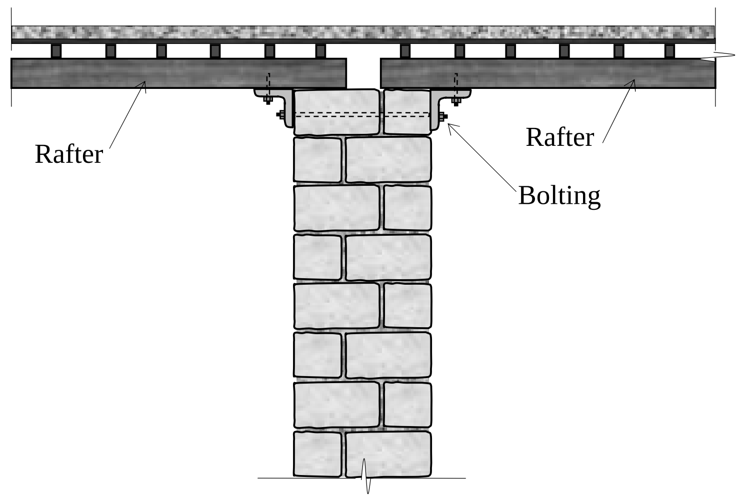


Figure 6.15 Anchoring of walls.

The internal anchoring can be done by dowel bars at the corners and junctions of the walls. An example of stitching two existing perpendicular walls is shown in Chapter 5. Else, dowels can be inserted at regular intervals of 500 mm and taken into the walls to sufficient length so as to develop the full bond strength (Figure 6.16).

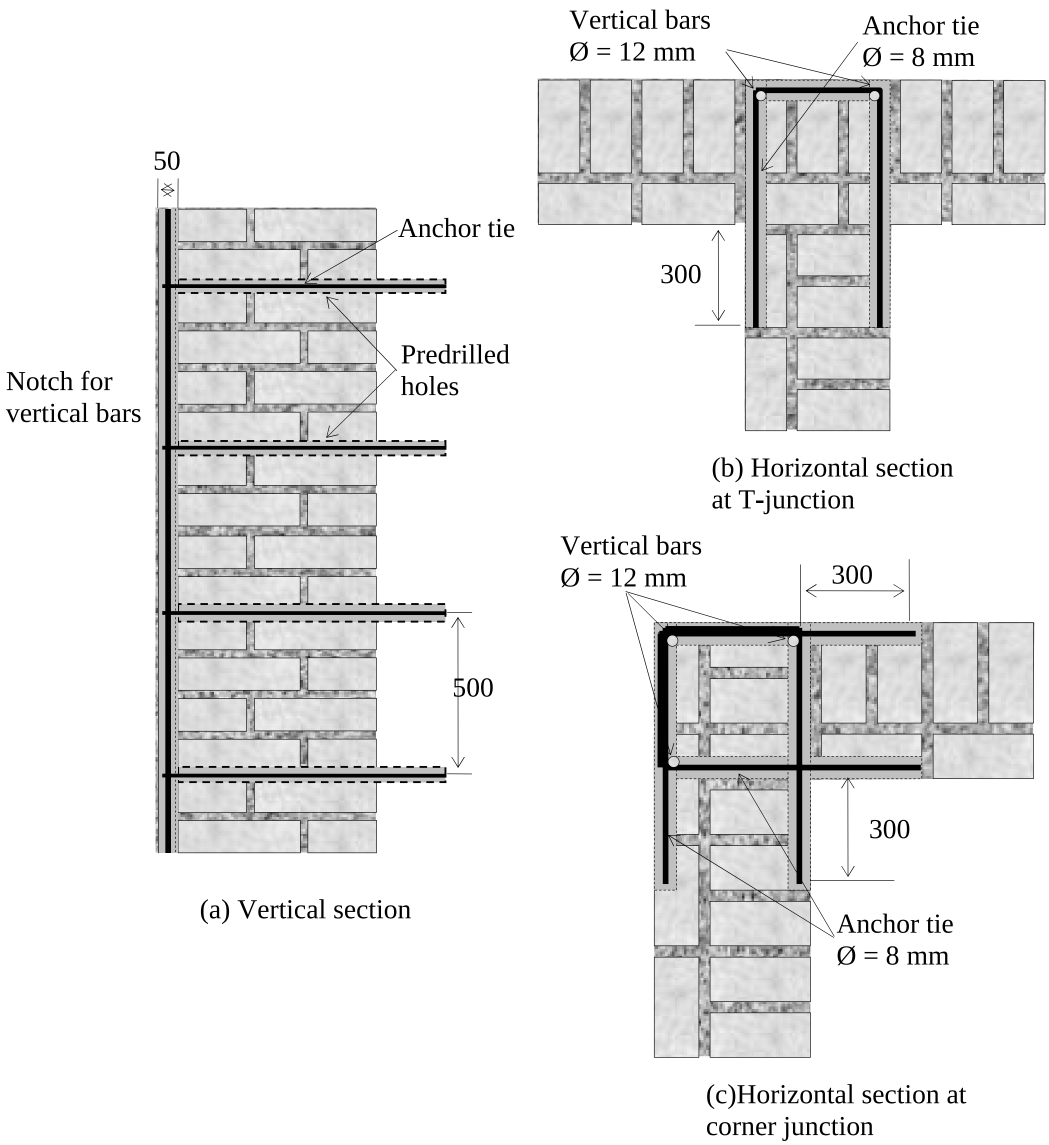
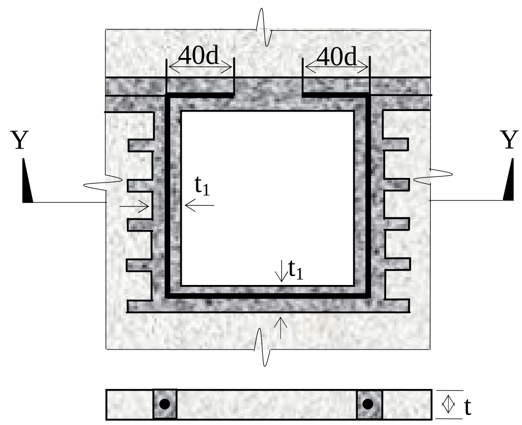


Figure 6.16 Insertion of dowel bars at corners and T-junctions (dimensions in mm)

6.7.6 Reinforcing the Openings

When the openings do not comply with the requirements of IS 4326: 1993, they can be either closed or reinforcement bars can be provided in the jambs (Figure 6.17).



t - thickness of the wall
t_1 - thickness of RC band
d - diameter of reinforcing bar

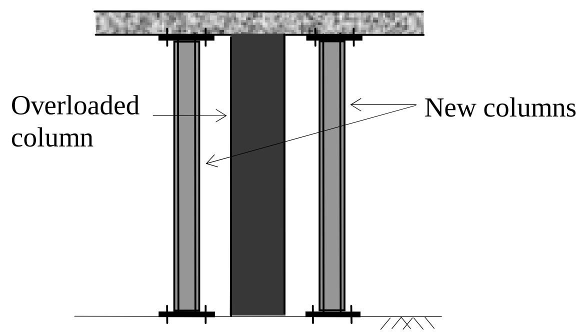
Figure 6.17 Strengthening masonry around window opening

6.8 STRENGTHENING OF PILLARS

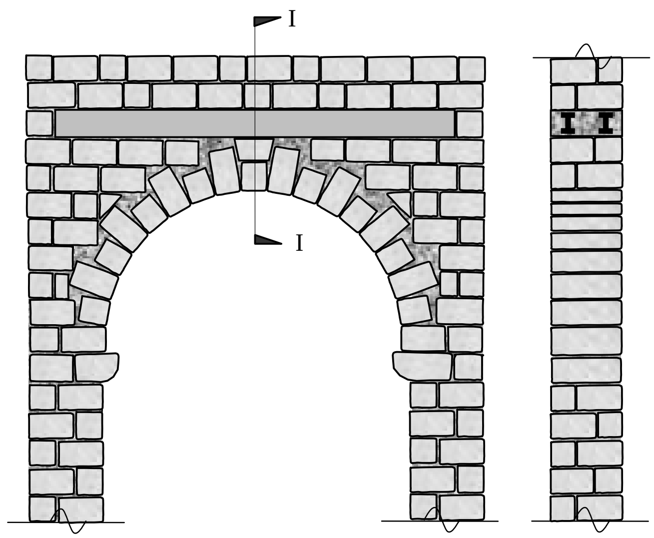
In absence of any reinforcement, the masonry pillars are weak under lateral forces. Such pillars can be strengthened by concrete jacketing, as shown in Chapter 5.

6.9 STRESS RELIEVING TECHNIQUES

This technique involves insertion of a new structural member in order to relieve an either overloaded or damaged component. A brick pillar or pier which is overloaded or damaged can be relieved by constructing columns on either side of the damaged pier as shown in Figure 6.18a. A weak brick arch can be relieved by a lintel consisting of steel beams, inserted just above the arch (Figure 6.18b). The piers below the arch are relieved of the lateral thrust from the arch.



(a) Insertion of new columns



(b) Insertion of lintel

Section I-I

Figure 6.18 Stress relieving by insertion of new members

6.10 GLOBAL RETROFIT TECHNIQUES

Global retrofit aims to improve the lateral load resistance of the building as a whole. Some of the techniques are described in Chapter 5. Here, these are listed.

1. Introduction of joints
2. Introduction of walls
3. Introduction of pilasters or buttresses
4. Splint and bandage strengthening technique.

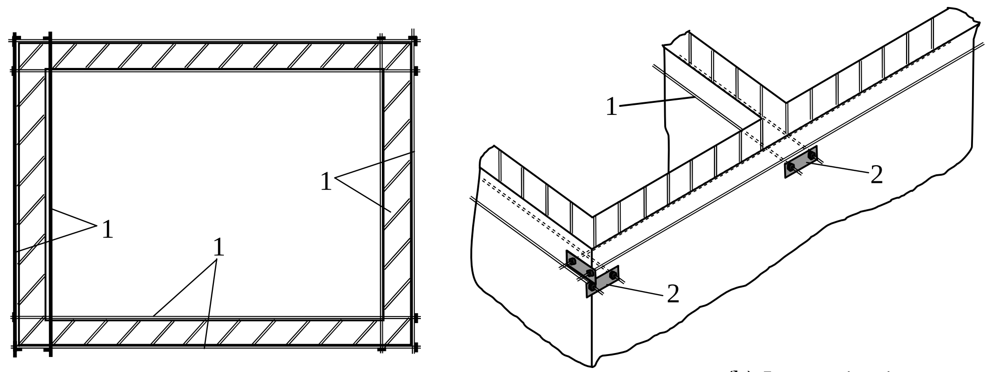
A few other techniques are explained.

6.10.1 Introduction of Frames and Braces

To relieve an overloaded masonry wall, a reinforced concrete frame may be inserted. Braces can be introduced to increase the lateral load resistance of the building, as well as to reduce the hazard due to out-of-plane collapse of a wall.

6.10.2 Strengthening by Post-tensioning

An unreinforced masonry wall develops tension due to bending or shear. Prestressing creates compression in the wall which counteracts the tension. The prestressing is introduced in an existing building by post-tensioning of external tendons. Horizontal tendons can be inserted in pairs on opposite sides of a wall so that out-of-plane bending is not introduced (Figure 6.19).



(a) Plan

(b) Isometric view

1- Tendons for prestressing
2- Anchor plates

Figure 6.19 Post-tensioning by external tendons

6.11 MAINTENANCE

Maintenance of masonry buildings is important for long term durability, even after retrofitting. With deterioration, serviceability and safety problems manifest. Cleaning of stains from efflorescence and carbonation is to be done as a part of regular maintenance. The stains can be brushed and washed with water. The roof should be accessible for inspection and maintenance. Any growth of vegetation and clogging of drainage spouts have to be cleared. To overcome the problem of water seepage, coatings such as aluminium stearate and alkyl trioxy silane can be applied. Condensation of water is a problem in bathrooms and toilets. To reduce the amount of condensation, proper ventilation or exhaust fans can be installed at appropriate locations.

6.12 IMPLEMENTED CASE

The following photos depict the retrofit of a factory building. The building had long and high walls. Continuous horizontal bands and vertical columns were introduced to enhance the structural integrity. Figure 6.20a shows the scaffolding during construction. Figure 6.20b shows the building after the completion of the project.

a) During construction

b) After completion

Figure 6.20 Retrofitting of a factory building

6.13 SUMMARY

This chapter gives an overview of the repair and retrofit techniques for masonry buildings. The procedure for evaluation of masonry buildings for seismic forces is provided. The building deficiencies are highlighted to create awareness for future construction. The retrofit techniques cover strengthening of individual members such as roofs, floors, walls and pillars, and global techniques. The importance of maintenance is highlighted. Finally, a case of implementation of retrofit is illustrated.

6.14 REFERENCES

1. IS 1905: 1987, "Code of Practice for Structural Use of Un-reinforced Masonry" , Bureau of Indian Standards.
2. IS 4326: 1993, "Earthquake Resistant Design and Construction of Buildings – Code of Practice", Bureau of Indian Standards.
3. IS 13828: 1993, "Improving Earthquake Resistance of Low Strength Masonry Buildings – Guidelines", Bureau of Indian Standards.
4. IS 13935: 1993, "Indian Standard for Repair and Seismic Strengthening of Buildings – Guidelines", Bureau of Indian Standards.
5. "Technologies for Retrofitting of Existing Buildings and Structures to make them Earthquake Resistant", (2006), Indian Institute of Technology Roorkee. Project sponsored by Technology Information, Forecasting and Assessment Council (TIFAC).
6. Hamid, A. A. and Drysdale, R. G. (1993), "Masonry Structures: Behavior and Design", Prentice Hall.
7. Arya, A. S. (1992), "Masonry and Timber Structures Including Earthquake Resistant Design", Nem Chand & Bros.
8. Dayaratnam, P. (1987), "Brick and Reinforced Brick Structures", Oxford & IBH Publishing Co. Pvt. Ltd.
9. Tomazevic, M. (1999), "Earthquake-resistant Design of Masonry Buildings", Imperial College Press, UK.

7

RETROFIT OF HISTORICAL AND HERITAGE STRUCTURES

7.1 OVERVIEW

A vast majority of historical buildings in India is constituted by stone and brick masonry structures and hence, the emphasis of retrofitting techniques in this chapter is for such structures. First, excerpts from the internationally endorsed principles and recommendations of heritage conservation advocated by the International Council on Monuments and Sites (ICOMOS) are stated. Next, the techniques for condition assessment appropriate for masonry structures are listed. The strengthening of components includes walls, arches, vaults, domes, towers and spires. For structure level intervention, the reduction of forces by base isolation and energy dissipation, the strengthening of soil and the foundation are briefly covered. The respective chapters for further information are mentioned. Finally, there is a section on archaeological reconstruction.

The retrofit techniques detailed here are those which were successfully applied to historical structures in European countries and relevant to the Indian scenario as well, where masonry is the predominant building material.

7.2 INTRODUCTION

The principle difference between a heritage structure and a regular structure is that retrofitting techniques cannot be indiscriminately applied with the sole aim of improving structural response to earthquakes. Judicious selection and application of retrofitting techniques, respecting the authenticity and the heritage value of the structure in its entirety is paramount. Use of new materials for repair must be dealt very cautiously. The aim of the retrofit is to preserve the historical structure for generations to come. In this context, it is appropriate to recall that even a powerful material such as reinforced concrete has a rather short life.

7.2.1 Structural Restoration and Seismic Retrofit

Seismic retrofit of historical and heritage structures is part of the complex, multidisciplinary science of conservation engineering. Structural assessment and remedial interventions on structural systems of historical buildings requires special considerations to retain the architectural integrity and historical authenticity.

Earthquakes pose a serious threat to the architectural heritage of the nation. Historical structures are predominantly composed of unreinforced masonry which is far more vulnerable to earthquakes than reinforced masonry, concrete or steel structures. Masonry displays complex, non-linear mechanical behaviour characterised by low tensile strength and high stiffness. Understanding the behaviour of masonry and subsequently prescribing remedial strategies for historical structures is complicated by factors such as use of local materials and techniques of construction, level of workmanship, age of the structure, weathering and alterations throughout its history and effects of previous earthquakes.

The techniques reported in this chapter are with reference to two categories of buildings: **traditional buildings** (residential clusters) within heritage areas of cities or towns and **monumental structures** such as royal palaces, victory towers and religious structures notable for their cultural, historical and architectural value.

7.2.2 Status of Monuments in India

The earthquake in Bhuj, Gujarat, on 26th January, 2001, brought down a number of monuments. Prior to this, the earthquake in Bhuj in 1819 was responsible for the loss of many monuments. A temple built in the 10th century at Kera, Kutch, is a classic example of the cumulative effects of the earthquakes. The structure was damaged in the 1819 event. The monolithic *kalasa*[*] that once crowned the spire sits today on the ground, covered in patina, where it had fallen two centuries ago. The temple almost completely collapsed during the event of 2001.

Appreciable advances in research on structural retrofit techniques, numerical modelling and prediction of structural response under seismic loads, advent of power tools and equipment have barely impacted the restoration and retrofit of monuments in India. The present scenario of restoration with semiskilled labourers using traditional tools and techniques for addressing serious structural problems is not adequate for retrofit.

7.2.3 Basic Principles of Structural Restoration

> Structures of architectural heritage present a number of challenges in restoration and retrofit, which limit the application of modern codes and building standards. Recommendations are desirable and necessary to ensure rational methods of analysis and repair methods appropriate to the cultural context. They are intended for those involved in conservation and restoration, but cannot in anyway replace specific knowledge acquired from cultural and scientific texts (ICOMOS, 2003).

This sub-section enumerates the excerpts of a few important recommendations of the International Council on Monuments and Sites (ICOMOS).

<u>ICOMOS, 1964</u>

1. The restoration of monuments must have recourse to all the techniques which can contribute to the safeguarding of the architectural heritage. (Article 2)
2. The intention in conserving and restoring monuments is to safeguard them no less as works of art, than as historical evidence. (Article 3)
3. The aim of restoration is to preserve and reveal the aesthetic and historical value of the monument and is based on the respect for original material and authentic documents. (Article 9)
4. Where traditional techniques prove inadequate, the restoration of a monument can be achieved by the use of any modern techniques of construction, the efficacy of which has been shown by scientific data and proved by experience. (Article 10)

[*] A traditional pot made of metal/stone generally found on the top of temple towers.

5. The valid contributions of all periods to the building of a monument must be respected, since unity of style is not the aim of a restoration. When a building includes the superimposed work of different periods, the revealing of the underlying state can only be justified in exceptional circumstances. (Article 11)
6. Replacements of missing parts must integrate harmoniously with the whole, but at the same time must be distinguishable from the original, so that the restoration does not falsify the artistic or historical evidence. (Article 12)
7. In all works of preservation or restoration, there should always be precise documentation, illustrated with drawings and photographs. Every stage of the work of clearing, rearrangement and integration, as well as technical and formal features identified during the course of the work, should be included. (Article 16)

ICOMOS, 2003

1. The removal of the inner structures maintaining only the façades does not fit the conservation criteria. (Article1.3)
2. No action should be undertaken without having ascertained the achievable benefit and harm to the architectural heritage, except in cases where urgent safeguard measures are necessary to avoid the imminent collapse of the structures (for example, after seismic damages). Those urgent measures, however, should not be irreversible. (Article1.7)
3. Therapy should address root causes rather than symptoms. (Article 3.1)
4. Safety evaluation and an understanding of the significance of the structure should be the basis for conservation and reinforcement measures. (Article 3.3)
5. The choice between “traditional” and “innovative” techniques should be weighed up on a case-by-case basis and preference given to those that are least invasive and most compatible with heritage values. (Article 3.7)
6. At times the difficulty of evaluating the real safety levels and the possible benefits of interventions may suggest “an observational method”, that is an incremental approach, starting from a minimum level of intervention. (Article 3.8)
7. Where possible, any measures adopted should be “reversible” so that they can be removed and replaced with more suitable measures when new knowledge is acquired. Where they are not completely reversible, interventions should not limit further interventions. (Article 3.9)

7.2.4 Degrees of Intervention

> Strengthening of historical buildings is a difficult compromise between requirements of structural theory and conservation principles. Intervention must be "*as much as necessary, but as little as possible*" and reversible to give room for better solutions in the future. Temporary interventions must be carried out to prevent catastrophic collapse if such a situation exists. Temporary interventions can ensure safety against collapse during the post-earthquake investigation (from aftershocks) and before the final retrofit procedure is tested and arrived at.

For a monumental building, eight degrees of interventions in the ascending order of intrusion are possible:

1. Prevention of deterioration
2. Preservation of existing state
3. Consolidation of the fabric
4. Restoration
5. Rehabilitation
6. Reproduction
7. Reconstruction
8. Translocation

7.3 CONDITION ASSESSMENT

7.3.1 Non-destructive Tests

Estimation of the expected strength of existing structural members requires the evaluation of the strengths of various materials in place, the actual dimensions of the members, and an extensive knowledge of the presence of cracks, cavities, open joints and other defects or discontinuities. Intrusive testing may in most cases be prohibitive not only due to possible architectural damages, but also due to risk of local collapse in critical regions of the structure that may trigger global instability. Therefore, non-destructive tests come forth as appropriate tools. A few non-destructive and partially intrusive techniques suitable for historical structures are listed. Brief descriptions of some of these tests are given in Chapter 4, Condition Assessment of Buildings.

1. Thermal method (infrared thermography)
2. RADAR technique
3. Ultrasonic pulse velocity test
4. Ambient vibration test
5. Endoscopy

7.3.2 Intrusive tests

Normally, in the case of heritage structures, intrusive testing procedures may not be permissible. If indispensable and if permissible, the choice of the technique should guarantee minimal intervention. On the other hand, fairly large specimens from debris of a structural collapse may well be used to perform destructive tests. The different tests are listed.

1. Core test
2. In-situ shear test
3. Bond-wrench test
4. Test of masonry prisms

7.3.3 Numerical Techniques

Numerical modelling is important for understanding the behaviour of the structure. Constraints for advanced modelling are the cost, the need for experienced engineers, the level of accuracy required and the availability of input. The need for validation of the model and results against in-situ observations is a key issue without which the results of a complex analysis may be rendered useless. Non-linear analysis is the most powerful method, capable of tracing the complete response from the elastic range, through cracking and crushing up to complete failure. Simplified modelling such as limit analysis using kinematic methods could be a useful too. General recommendations in the numerical modelling of historical structures are as follows.

1. It is better to model structural parts than complete structures.
2. Do not use full-structure three-dimensional modelling unless it is necessary.
3. Avoid using linear elastic calculations for historical structures.

7.4 STRENGTHENING OF MASONRY WALLS

Various techniques are available for strengthening different types of masonry walls. The type and quality of the masonry material and the structural integrity of the building are the main criteria to be considered when choosing the method of strengthening. Some of the methods are described in Chapter 6, Retrofit of Masonry Buildings.

7.4.1 Repair of Cracks

The cracks can be repaired grout injection. In case of excessively damaged walls, in addition to grout injection, the area around the cracks can be coated with a cement concrete coating reinforced with a wire mesh (Figure 7.1). Table 7.1 provides recommendations for various crack widths.

Table 7.1 Recommended repair procedures for various crack widths

Crack width	Recommended procedure	Remarks
< 1.0 mm	Injection with epoxy	It may be costly if the damaged area is large.
0.3 to 3 mm	Injection with cement grout that contains shrinkage reducing admixture.	
> 10 mm	Reconstruction of damaged area with new brick units.	Cracks may be sealed with mortar if the wall thickness is relatively small.

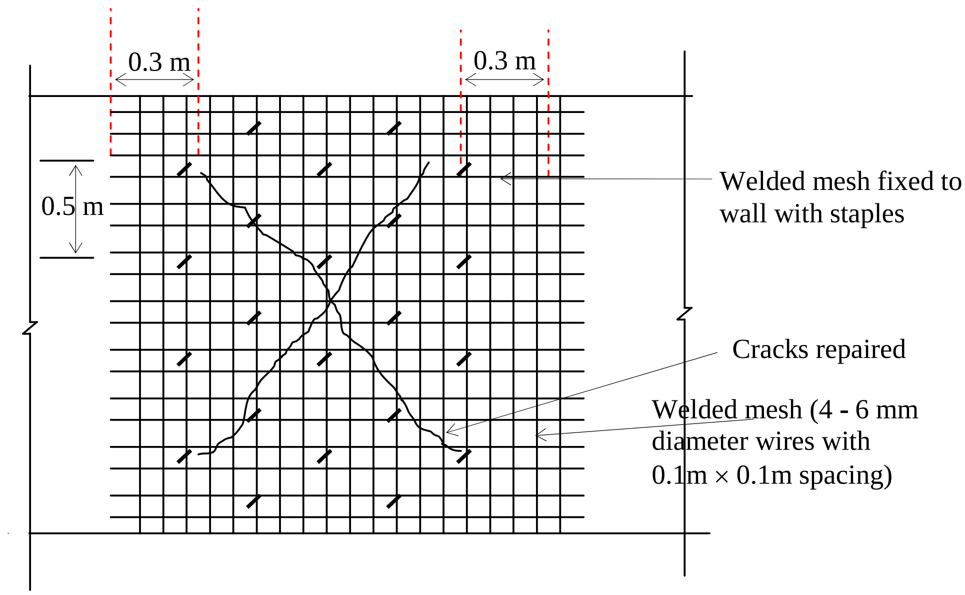
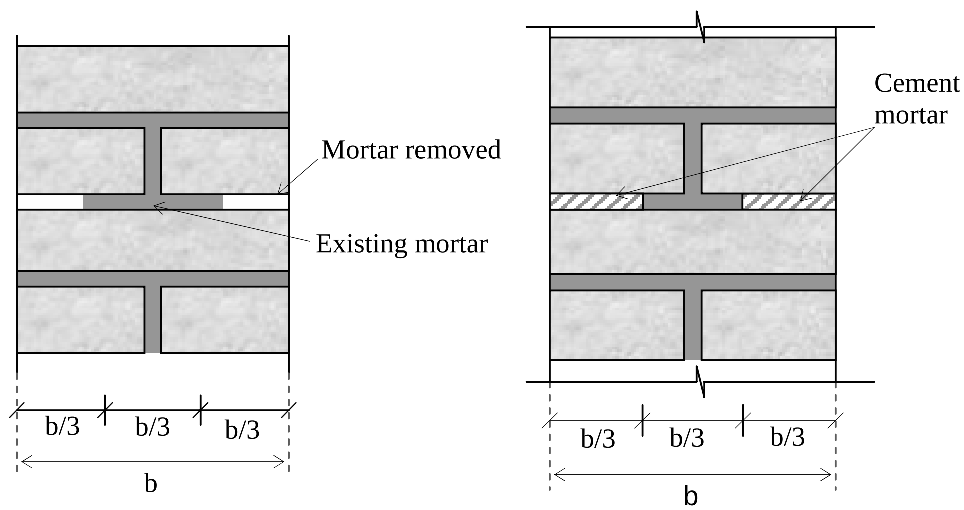


Figure 7.1 Repair of heavily cracked masonry wall using welded mesh and staples

7.4.2 Repointing of Bed-joints

The resistance of a wall to lateral and vertical loads can be considerably improved by replacing parts of the existing deteriorated mortar in bed-joints with mortar of better quality. This is applicable where bed-joints are level, the mortar is of poor quality and the masonry units are good.

As shown in Figure 7.2, existing mortar up to one third of the walls thickness is removed from the joints on one or both sides of the wall using clamps or electric chippers. To maintain vertical stability of the wall, repointing is completed on one side and then the other side is repointed. Once the existing mortar has been removed, the surface of the bed-joint is cleaned thoroughly and moistened with a water jet. The procedure of repointing is repeated on the other side of the wall once the fresh mortar has attained sufficient strength.



(a) Removal of existing bed joint

(b) Re-pointing with cement mortar

Figure 7.2 Section of wall strengthened by re-pointing of bed-joints

Placing steel bars along the bed-joints improves the ductility and energy dissipating capacity of the wall. As shown in Figure 7.3, 6 mm bars are placed at 0.3 to 0.5 m interval along the height of the wall, adequately anchored at the ends of the wall. The joints are then repointed with cement mortar. Stainless steel bars, fibre reinforced plastic bars and other synthetic ropes are alternative to steel bars.

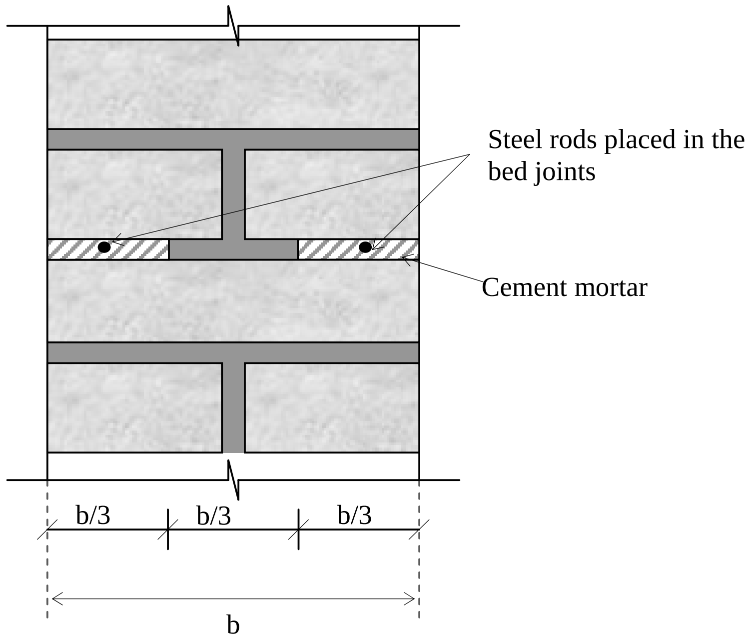


Figure 7.3 Re-pointing of bed joint with steel bars

7.4.3 Reinforced Concrete Jacketing

In the case of heavily damaged brick masonry walls, or where there is a need to strengthen the entire wall, the application of external reinforcement and concrete on both sides of the wall is a way of improving its lateral strength and energy dissipation capacity. The jacket can be in the form of ferro-cement with wire mesh or reinforcement mat with shotcrete, as illustrated in Chapter 6.

In case of stone masonry walls, plaster and loose stone pieces are removed first and all the cracks are grouted and sealed. Next, 6 to 8 mm diameter bars are placed on either side of the wall. At regular intervals stones are removed from the wall and a reinforcement cage is placed in the void. The void is filled with concrete to create a shear connector (Figure 7.4). This ensures efficient transfer of forces between the existing wall and the new concrete.

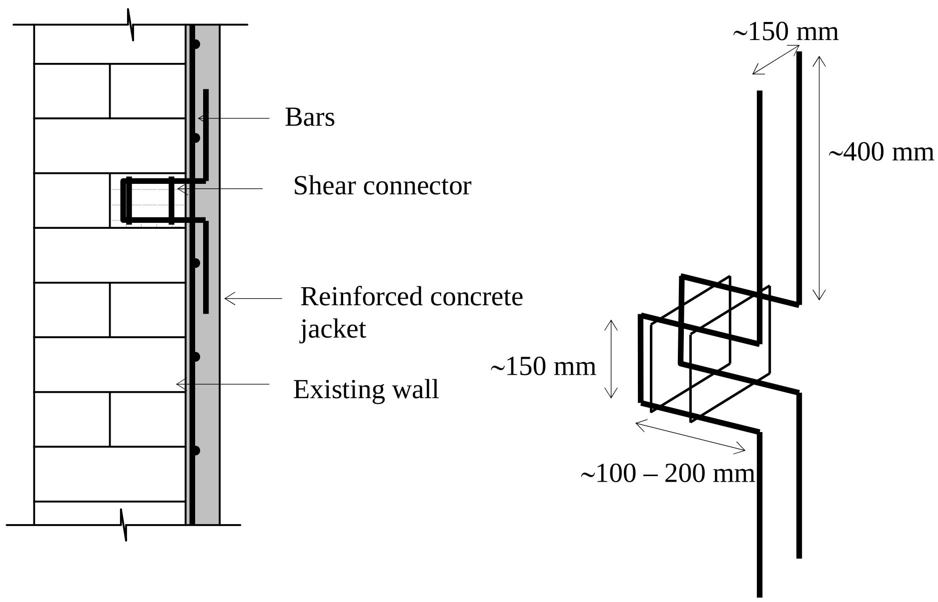


Figure 7.4 Reinforced concrete jacketing with shear connector

7.4.4 Grout Injection

Stone, mixed stone and brick masonry are frequently characterised by two outer leaves of masonry with an infill of smaller pieces of stone (Figure 7.5). Lime mortar, often of relatively poor quality, is used as the bonding material. Due to poor methods of construction and agents of weathering, many voids are formed within, drastically weakening their resistance to lateral loads.

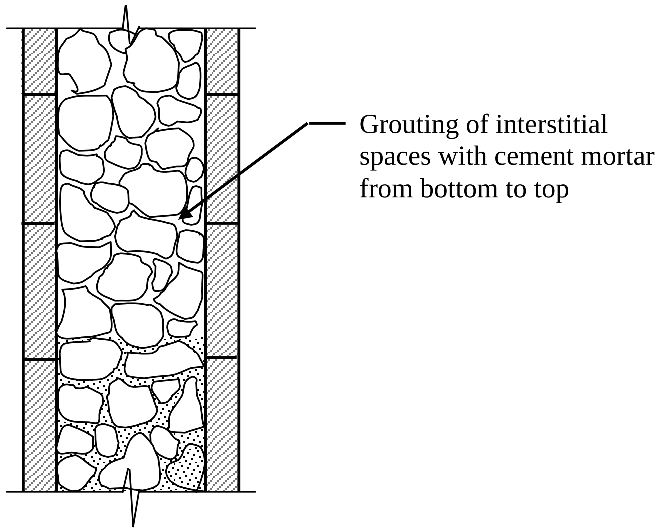


Figure 7.5 Grouting of a masonry wall

Systematically filling the voids by cementitious grout injection is an efficient method of strengthening. After hardening, the injected grout will bond the loose parts together into a solid homogeneous structure. A cementitious mix (90% Portland cement and 10% pozzolans) is injected into the wall through injection tubes and nozzles built into the joints between the stones, uniformly over the entire wall surface at 0.5 to 1.0 m interval. The grouting should proceed from the bottom to the top. The advantage of this intervention is that it is invisible.

> Grout mixes have to be appropriately modified with hydrophobic (water repellent) additives to prevent capillary activity of the hardened cement grout that can damage frescos and surface decorations on historical masonry.

7.4.5 Prestressing

Prestressing can be introduced in an existing structure by post-tensioning. Tendons can be inserted in the holes that are drilled through the central part of a wall at uniform spacing. After prestressing a uniform compression is induced in the wall, thereby increasing its lateral load resistance. The tendons can be removed from the wall in case a better solution for strengthening is developed.

7.4.6 Wall Reconstruction

There may be cases where heavily damaged brick and stone masonry walls or parts of walls cannot be repaired or strengthened. If the remaining structure is retainable, then careful dismantling and reconstruction of the wall is the only option. During reconstruction, materials compatible with the original masonry but of improved quality should be used. Special stitching units should be incorporated at uniform interval to provide good connection between the new and existing masonry.

In case of stone masonry, reconstruction becomes necessary where the wall has bulged excessively or collapsed, as shown in Figure 7.6. Complete reconstruction can be avoided if one of the layers is stable enough to be used as the formwork for reconstruction. Connecting stones must be used at intervals of 1.0 m to bridge the two layers. After the reconstruction, grouting should be carried out to ensure filling up of all the voids and to increase the homogeneity of the wall.

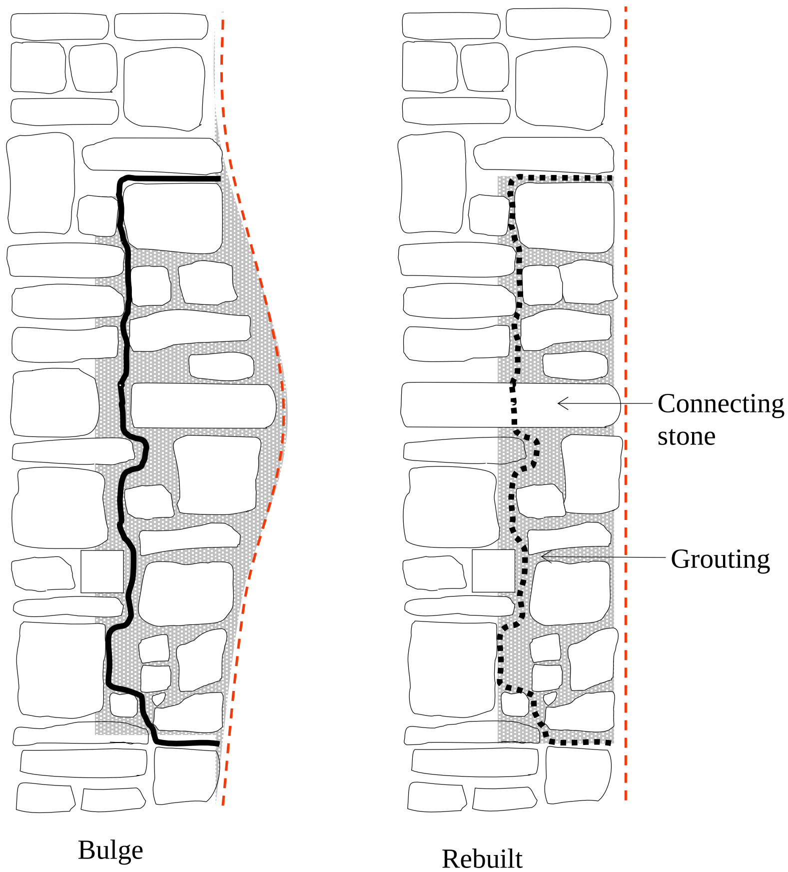


Figure 7.6 Reconstruction of wall (Tomazevic, 1999)

7.4.7 Strengthening using Fibre Reinforced Polymer

The use of fibre reinforced polymer (FRP) is non-invasive and reversible, a favourable feature for historical buildings. Chapter 13 describes the properties of FRP and the basic calculations for design. FRP sheets or bars can be used to strengthen masonry walls as illustrated in Chapter 6, Retrofit of Masonry Buildings. Figure 7.7 illustrates the steps of using FRP strips on cracked walls. In the case of masonry domes, FRP strips may be used as horizontal bands on the extrados of the domes.

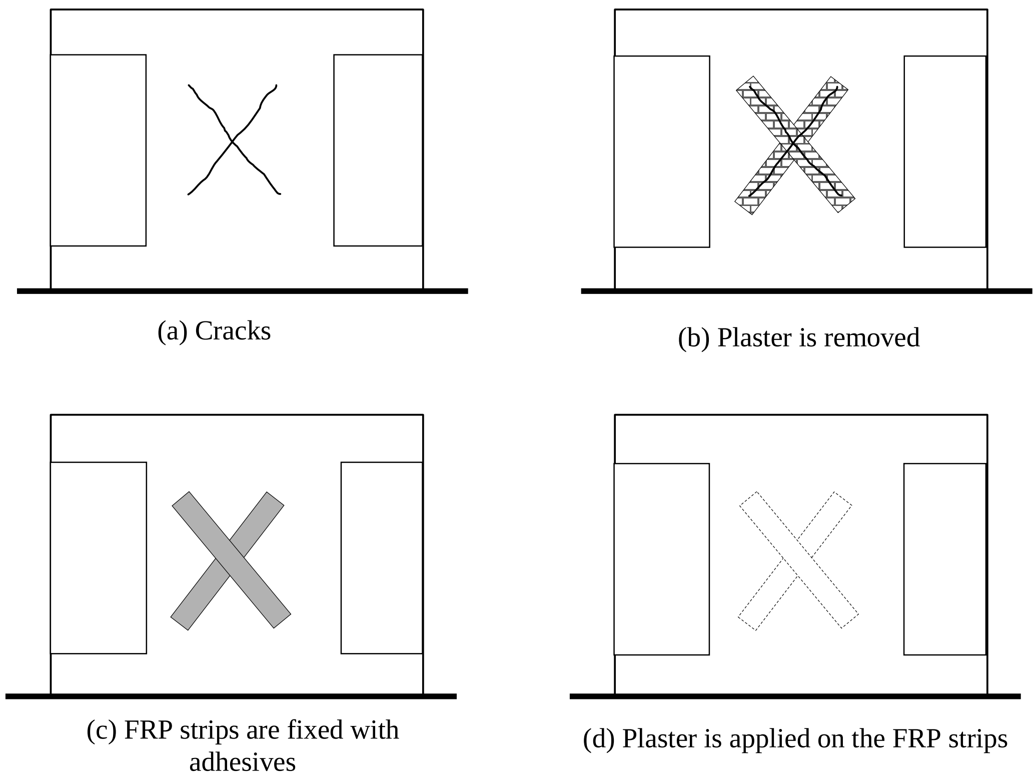


Figure 7.9 Steps in repairing cracks in a masonry wall using FRP strips

7.4.8 Strengthening using Shape Memory Alloys

Shape Memory Alloys (SMA) are metal alloys (nickel–titanium) endowed with very unusual thermo-mechanical properties due to reversible crystallographic phase transformation. The ability to recover large deformations in loading-unloading cycles is known as the super-elastic behaviour. SMAs are particularly suited for historical buildings. They may be used as wires or strands in association with conventional steel strands, and as parts of devices capable of increasing both in-plane and out-of-plane capacities of masonry walls.

Figure 7.10 shows a bracing system consisting of diagonal steel bars with a dissipating device on top of each. A nickel–titanium wire is wound up on a pulley. One end of it is connected to a steel anchor plate and the other end is connected to a diagonal steel bar. The anchor plates on each side of the wall are bolted together securely. A steel beam at the base provides anchorage for the steel bar. This system provides permanent compression on the wall. It also satisfies the requirements of non-invasive nature and reversibility of intervention for historic buildings.

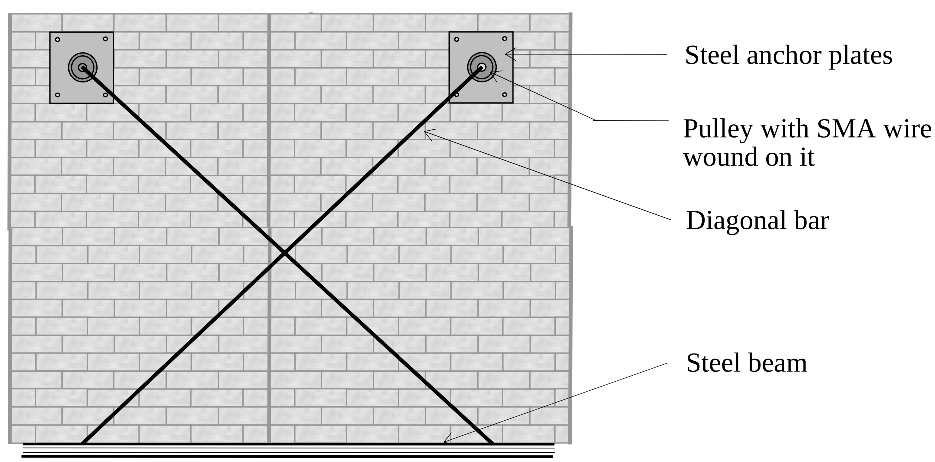


Figure 7.10 Bracing with shape memory alloy wires

7.4.9 Repair of Wall Corners and Intersections

The corners and intersections of walls are frequently damaged during earthquakes. They can be strengthened by stone stitching or metal stitching (Figure 7.11). In case of stone stitching, new stones diagonally connecting the intersecting walls are placed with sufficient bearing and embedded in cement mortar, at intervals 500 to 750 mm. In case of metal stitching, steel strips welded to anchor plates at their outer ends are placed at the intersection.

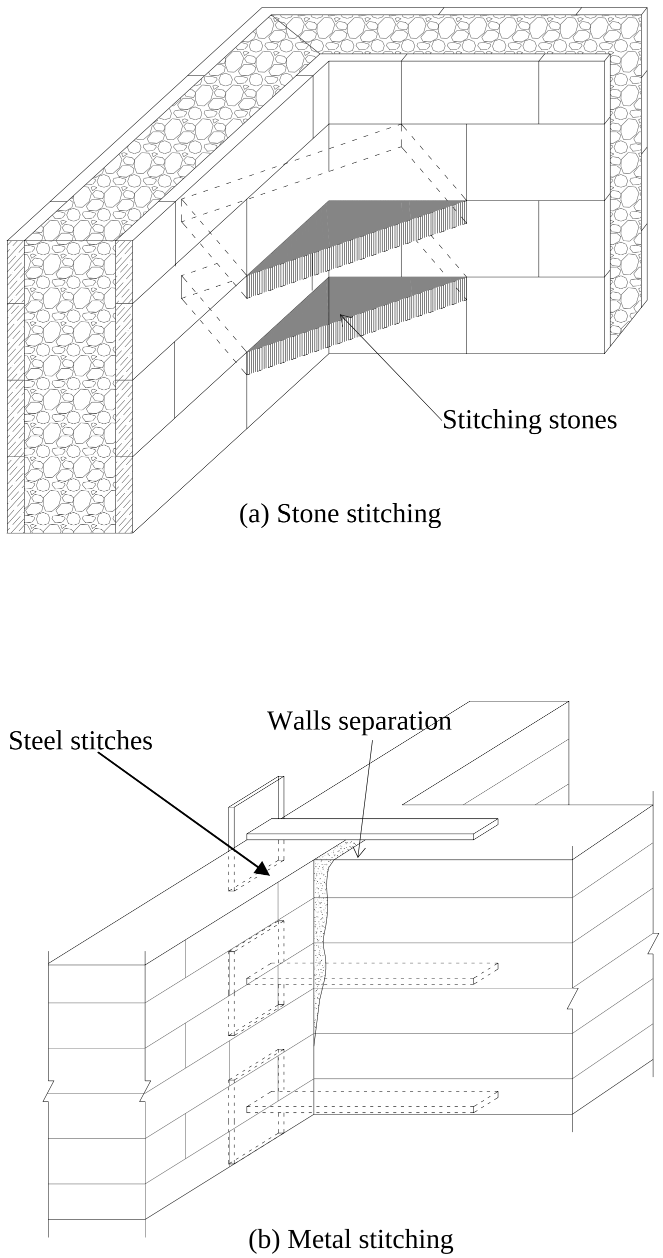


Figure 7.11 Repair of wall intersections using stone or metal stitching

7.4.10 Improving Connections of Secondary / Non-structural elements

Repair to secondary elements like cornices, parapets, merlons, sunshades, facing stone slabs, etc., primarily involves establishment of positive connections. Techniques such as insertion of pivots, nails, dowels, clamps, stirrups, anchorages, etc. can be applied to establish the connections. Grouting of the holes (to accommodate the connectors) will improve the effectiveness of the connections.

7.5 STRENGTHENING OF ARCHES, VAULTS AND DOMES

Dry stone masonry offers very high strength in compression, but their joints provide limited shear and tensile resistance as they depend purely on friction. A positive connection between the stone blocks can be achieved using dowels, cramps or special tie bars or structural connections inserted through specially prepared holes in the joints, without being visible from outside (Figure 7.12).

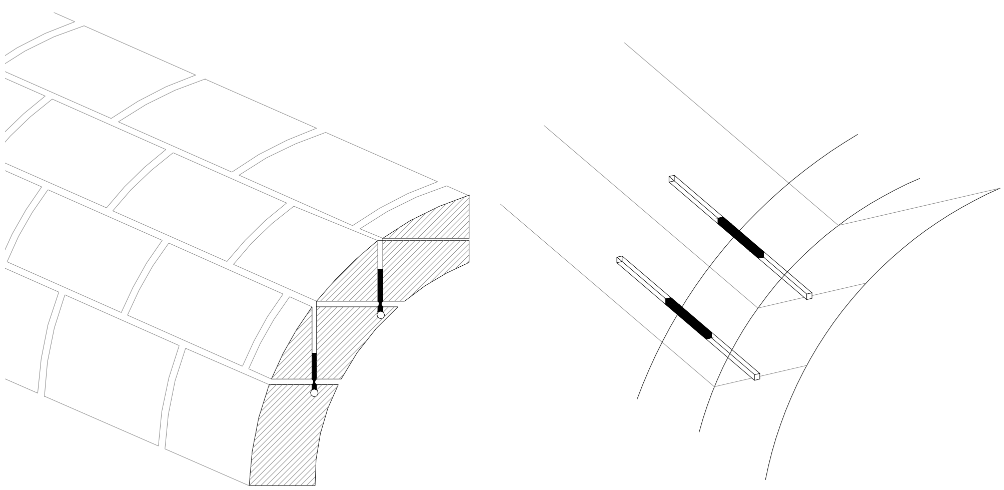
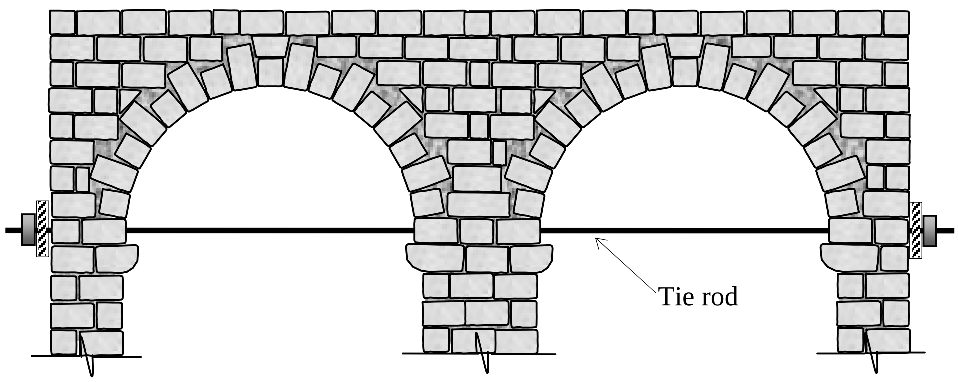
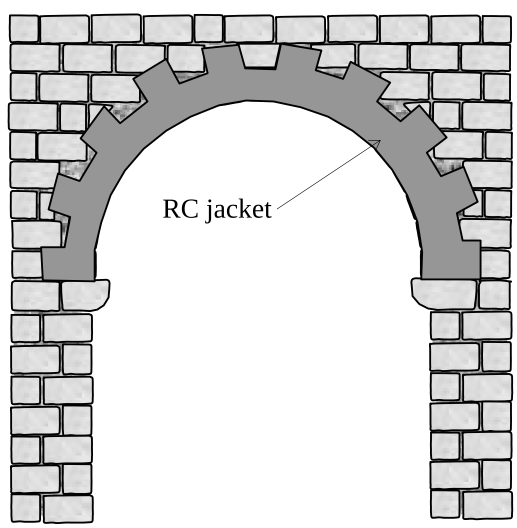


Figure 7.12 Connecting stone blocks with dowels (Croci, 1998)

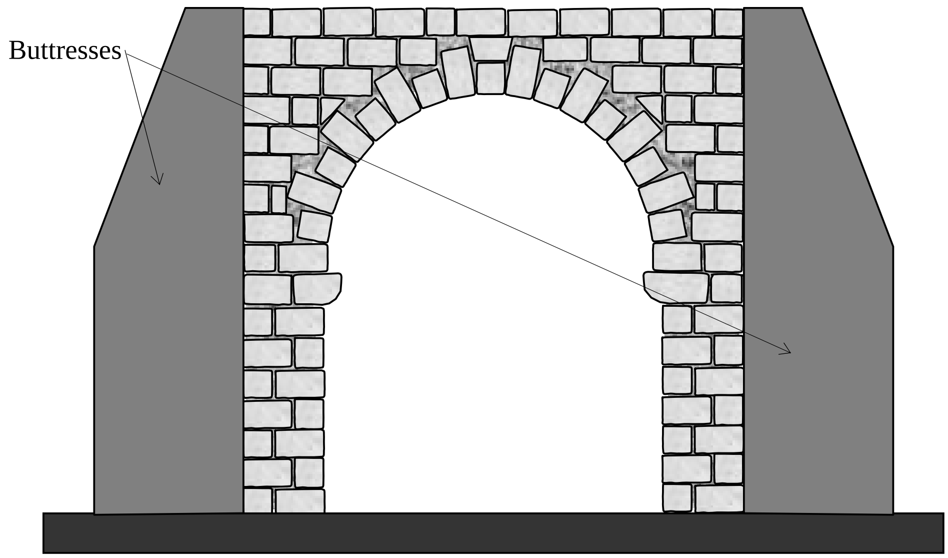
Damage to arches is generally caused by lack of thrust at the springers. The thrust can be achieved by steel ties or by large masonry masses like buttresses at the outside (Figure 7.13). The ties can be post-tensioned for better effectiveness. A reinforced concrete jacket can be added on the intrados of an arch. In corbelled arches, the shear forces between the blocks may be exceeded, leading to slippage of the blocks and possible collapse of the system. The shear resistance can be improved by tying each block to its neighbour with dowels or clamps.



(a) Use of tie rods



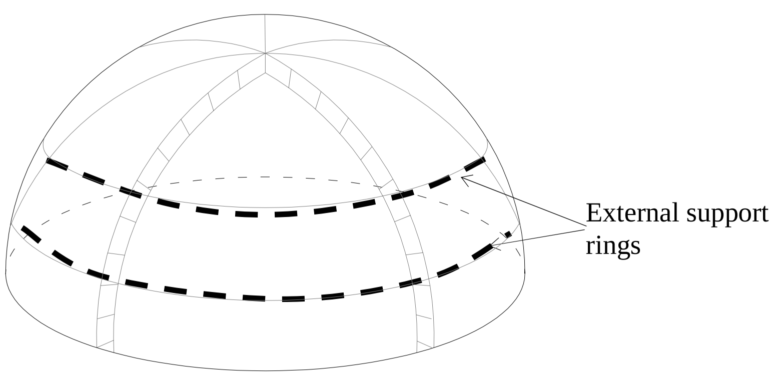
(b) RC Jacket on the intrados



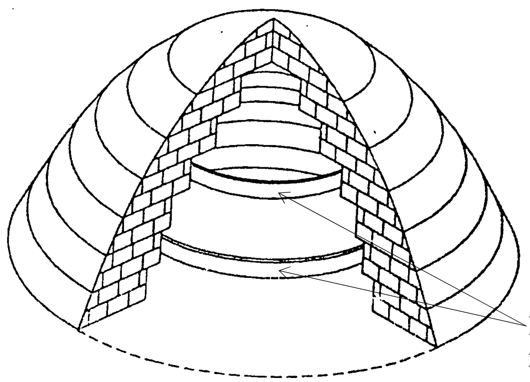
(c) Counteracting buttresses

Figure 7.13 Strengthening of arches

In domes, external support rings can provide compression to enhance the integrity. In corbelled dry block vaults, an inwards collapse can occur due to loosening of the blocks. Internal support rings or bracing placed in the interior surface can counter the damage (Figure 7.14).



a) Dome



b) Corbelled dome

Figure 7.14 Strengthening of domes (Croci, 1998)

7.6 STRENGTHENING OF TOWERS AND SPIRES

Towers and minarets have the problems similar to that of pillars and walls. Their form, height and slenderness imply a lack of redundancy for redistribution of stresses and lack of energy dissipation capacity. This results in concentration of stresses at the base and brittle failure. Repair and strengthening of towers mainly consists of containment of lateral expansion. Reinforcements and prestressed tendons may be required in horizontal and vertical directions.

An effective, short-term reversible intervention to stabilise old masonry towers would be strengthening under dry construction with tie rods. As illustrated in Figure 7.15, steel diaphragms are provided at the centre of the cross section to prevent inward movement of the walls of a tower. The diaphragms also serve as anchorages for radially-arranged tie rods that protrude through putlog holes in the walls. This prevents the walls from bulging outwards. Moderate post-tensioning of the tie rods can be favourable. Vertical prestressing of towers may improve the overall flexural capacity of towers, but it can drastically change (at times for the worse) the dynamic behaviour of the tower.

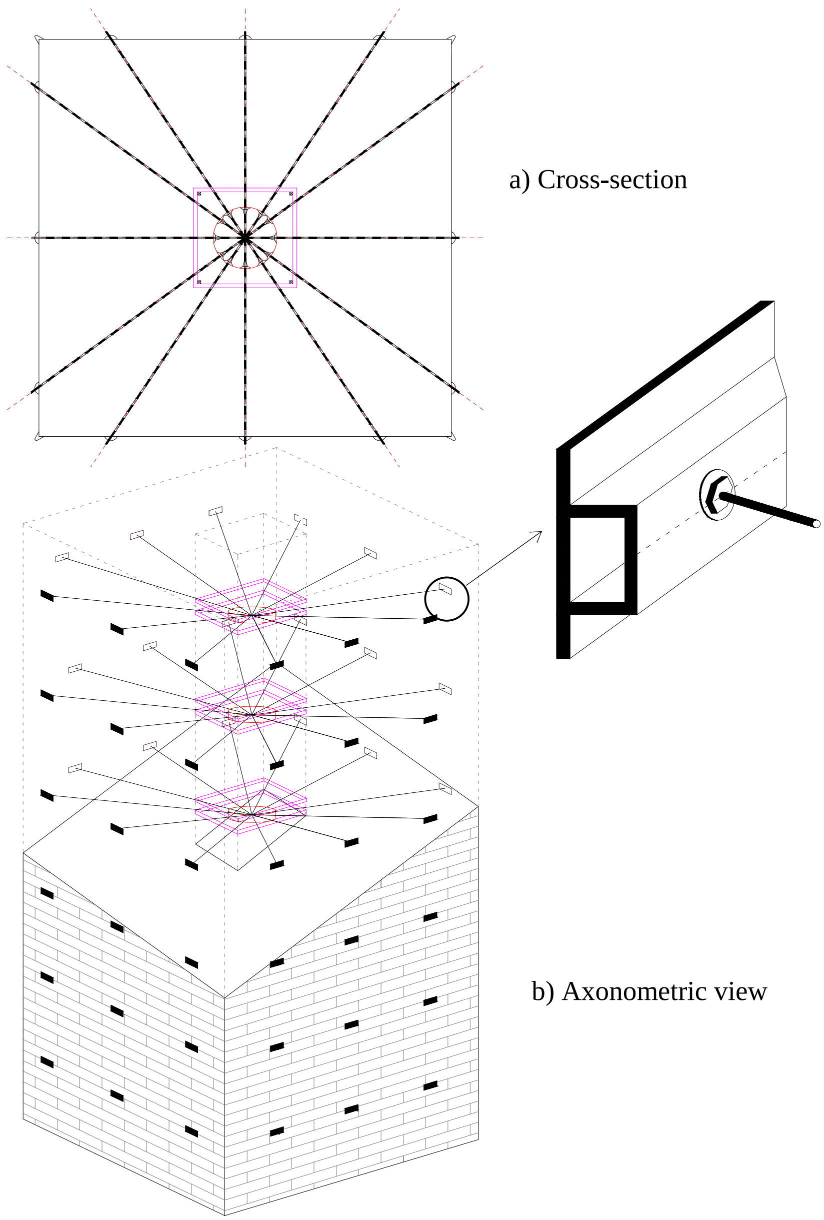


Figure 7.15 Strengthening of towers (Ballio, 1993)

7.7 REDUCTION OF SEISMIC EFFECTS ON STRUCTURE

7.7.1 Reduction of Forces

If the structure can be 'isolated' either at the foundation level or at the ground floor level from the portion below, then the forces in the members due to ground motion are reduced. The remedial action consists of cutting the structure to create a joint and to allow relative movement of the super- and sub-structures. This technique is referred to as base isolation.

Another method to reduce forces is the energy dissipation. Special devices that are capable of following a particular hysteresis loop during each cycle of ground acceleration, dissipate part of the energy imparted by the earthquake. Limiting the forces to the structure reduces the risk of damage to the contents. This is particularly relevant for museum buildings and buildings with heritage items. The techniques of base isolation and energy dissipation are described in Chapter 14.

7.7.2 Reduction of Mass

A reduction of induced forces can be achieved by reducing mass, especially at upper levels. Extreme situations may require some amount of demolition to reduce the loading.

7.8 STRENGTHENING OF SOIL AND FOUNDATION

As observations of earthquake damage indicate, existing foundations are rarely the reason for inadequate seismic behaviour of historical masonry structures. Failures due to soil liquefaction have to be addressed by adequate geotechnical measures. The methods of strengthening the soil or foundation of historical buildings can be grouped into the following (Przewlocki *et al.*, 2005).

1. Increasing the area of foundation, lowering the foundation level and strengthening the existing foundation.
2. Inclusion of structural elements such as piles or micropiles, and underpinning.
3. Modifying the effective stress in the soil by drainage or consolidation.
4. Improving the subsoil by chemical or cement grouting, or electro-osmosis.
5. Replacing the entire sub-structure.

The geotechnical considerations are described in Chapter 11, Geotechnical Seismic Hazards. The retrofit of foundations is covered in Chapter 12.

7.9 ARCHAEOLOGICAL RECONSTRUCTION

7.9.1 Principles

Archaeological reconstruction is a viable option for a heritage structure destroyed in a seismic event. Archaeological principles give a basis for the reconstruction of structures that have been destroyed by natural calamities (earthquakes, volcanoes, and storms), extensive weathering action or even human activities (wars), from original salvaged materials.

Reconstruction depends on effective use of earlier architectural drawings, verbal documentation, sketches, paintings and photo-documentation of the structure under consideration. Reuse of salvaged building material to the maximum extent is imperative to retain authenticity. Therefore, the process relies extensively on documentation of the remains of the structure and their correlation with drawings. In case of missing components, new materials compatible in structural behaviour and similar in appearance to the parent material should be used in the reconstruction. Modern tools for documentation, structural analysis, communication (for example, multimedia) and construction practices help the process of archaeological reconstruction.

7.9.2 Procedure of Rubble Clearance

The following is a course of action for archaeological clearance of rubble.

1. Determining and documenting the surroundings of the location where the object is found with the clear reference. Classification of the place with clear orientation and designation of the area of the find.
2. Evaluation of individual pieces found; determining whether they are worth recording.
3. Clear labelling of the objects found and recording all information pertaining to the find.
4. Description, illustration and graphical representation of the piece found. Decision on storage or filing.
5. Documentation of all stages of work. Preparing drawings to map out the place where the object is found in the ground plan and the sectional drawing.
6. Identification of the piece found based on all knowledge gained up to the present.
7. Archaeological evaluation of the knowledge gained towards reconstruction of the structure. Preparation for reutilisation of parts/pieces for the new structure.

The aim of reconstruction is to secure as much original material as possible for the reconstruction work. The objective of clearing the rubble is to obtain information with regard to the following points.

1. Original geometry and dimensions and architectural details.
2. Details of original construction and materials used.
3. Interior decoration (stucco, paintings, plastering, lining and finishing).
4. Technological problems involved in erecting the building.
5. Effectiveness and advisability of restoration measures.
6. Degree of destruction.

7.9.3 Procedure for Archiving Finds

An inventory of all the finds is to be drawn up to match the pieces wherever possible. A database with electronic pictorial information of all the finds has to be created for future archaeological work. The finds are then mapped on to specially prepared drawings of the latest state of the façades and a number of ground plans, sections and photographs. Many of the objects found, can provide valuable information as models and patterns for the reconstruction.

7.9.4 Criteria for Reconstruction

1. The structure has to be reconstructed with salvaged, reusable materials from the ruins coupled with new materials that are compatible, using modern techniques of construction.
2. The reconstructed structure has to incorporate certain modifications taking into consideration modern day requirements (electrical services, lifts, plumbing services, etc.).
3. Analytical and numerical investigations on the structural behaviour of the original structure will help in incorporating anti-seismic strengthening measures in the new structure.
4. Issue of weathering of materials must be confronted (for example, water-proofing).
5. Durability and precision can be ensured with the use of modern analytical and construction tools and practices.

7.10 SUMMARY

The chapter presents a wide coverage of retrofit techniques for vulnerable heritage structures. The techniques include intervention at a component level and intervention at the structure level. At component level intervention, strengthening of walls, arches, vaults, domes, towers and spires are covered. For structure level intervention, the reduction of forces by base isolation and energy dissipation, the strengthening of soil and the foundation are briefly covered. Besides the retrofit techniques, there are excerpts from the recommendations advocated by the International Council on Monuments and Sites (ICOMOS) and sections on condition assessment and archaeological reconstruction.

7.11 REFERENCES

1. Ballio, G. (1993), “Structural Preservation of the Fraccaro Tower, Pavia”, Structural Engineering International, Vol. 2, No. 1, pp. 9 – 11.
2. Croci, G. (1998) “The Conservation and Structural Restoration of Architectural Heritage”, Computational Mechanics Publications, UK, pp. 147 – 184.
3. ICOMOS (1964), “International Charter for the Conservation and Restoration of Monuments and Sites (Venice Charter)”, International Council on Monuments and Sites.
4. ICOMOS (2003), “ICOMOS Charter - Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage”, International Council on Monuments and Sites.
5. Przewlócki, J., Dardzinska, I. and Swinianski, J. (2005), “Review of Historical Buildings’ Foundations”, Geotechnique, 55, No. 5, pp. 363 – 372.
6. Tomazevic, M. (1999) “Earthquake Resistant Design of Masonry Buildings”, Imperial College Press, UK, pp. 203 – 252.

8

STRUCTURAL ANALYSIS FOR SEISMIC RETROFIT

8.1 OVERVIEW

There are two stages of analysis for a building. First, the structural analysis provides the demands in the members of a building. Second, the member analysis provides the capacities of the members. This chapter covers the structural analysis of buildings. The member analysis can be as per the recommendations of the following codes published by the Bureau of Indian Standards.

1. Masonry: IS 1905: 1987, "Code of Practice for Structural Use of Un-reinforced Masonry"
2. Concrete: IS 456: 2000, "Plain and Reinforced Concrete – Code of Practice"
3. Steel: IS 800 (Draft), "Indian Standard Code of Practice for General Construction in Steel".

Structural analysis is a part of the detailed evaluation of an existing building. The steps involve developing a computational model of the building, applying the external forces, calculating the internal forces in the members of the building, calculating the deformations of the members and building, and finally interpreting the results. This chapter covers the analysis of reinforced concrete or steel framed buildings. The analysis of masonry buildings is covered in Chapter 6, Retrofit of Masonry Buildings.

The analysis can be linear or non-linear, elastic or inelastic, static or dynamic. The equivalent static analysis is a linear elastic static method of analysis. The response spectra method and the linear time history analysis are two types of linear elastic dynamic method of analysis. The pushover analysis is a non-linear inelastic static method of analysis. Finally, the non-linear time history analysis is a non-linear inelastic dynamic method of analysis. This chapter explains the equivalent static analysis. The fundamentals of the response spectra method and the pushover analysis are also elucidated.

The important aspects of developing an appropriate computational model of a building are the modelling of the material properties, structural members, applied loads, and finally the interpretation of the results. The 'knowledge factor' is used to account for the missing information of the material properties of an existing building. Suggestions are provided for assigning appropriate moment of inertia for the beam and column elements in a reinforced concrete building, considering the effect of cracking. Presence of large cut-outs in a rigid slab diaphragm should be considered while modelling the diaphragm action. The shear walls and core walls can be modelled using column elements with suitable rigid links to the connected elements. The infill walls can be modelled by the equivalent strut method to consider their stiffness in the lateral load resistance of a building. Suggestions are provided for modelling the ends of the bottom storey columns considering the rotational restraint offered by the foundation.

For the equivalent static analysis, the essential provisions of IS 1893: 2002, "Criteria for Earthquake Resistant Design of Structures", are discussed. Simple methods of calculating the centre of mass for a floor and the centre of rigidity for a storey are illustrated. For the response spectrum method, the calculation of base shear is explained. The pushover analysis has become popular in the recent years because of its capability of modelling the non-linear response of the elements of a building and the visualisation of the damage states offered by the programs. The essential features of the pushover analysis are discussed.

8.2 INTRODUCTION

> At present, the structural analysis is performed using a suitable computer analysis program. The steps involve developing a computational model of the building, applying the external forces, calculating the internal forces in the members of the building, calculating the deformations of the members and building, and finally interpreting the results.

Structural analysis is a part of the detailed evaluation of an existing building. A detailed evaluation is decided based on the results of preliminary evaluation. Structural analysis can be linear or non-linear, elastic or inelastic, static or dynamic. In a linear elastic analysis, the deformation in a member is considered to be proportional to the internal force and recoverable when the applied force is removed. In a non-linear inelastic analysis, the deformation in a member need not be proportional to the internal force. There is plastic deformation (deformation that cannot be recovered when the applied forces are removed) and energy absorption in a member for higher levels of internal force. This type of non-linear behaviour is referred to as material non-linearity. In addition, geometric non-linearity due to P-Δ effect can be incorporated. The P-Δ effect refers to the increase in moment in the columns due to its lateral deflection or due to the drift of a storey.

In a static analysis, the vibration mode shapes or the time-wise variation of the quantities are not considered. In a dynamic analysis these are considered to a certain extent. The different methods of analysis can be grouped as shown in Table 8.1.

Table 8.1 Methods of structural analysis

	Linear elastic	**Non-linear inelastic**
Static	Equivalent static method	Pushover analysis
Dynamic	Response spectra method, Linear time history analysis	Non-linear time history analysis

Because of the difficulties and uncertainties in a non-linear dynamic analysis, this is not used in regular design practice. This chapter discusses the other types of analysis. The main purpose of these analyses, from the perspective of seismic evaluation, is to check the adequacy of the building components and ascertain code compliance.

8.3 COMPUTATIONAL MODEL

Adequate information is required to develop a reliable computational model of an existing building. The necessary documents include the following.

1. Architectural drawings
2. Structural drawings
3. Geotechnical report
4. Reports from data collection, preliminary evaluation and condition assessment.

The components which are not part of the main structure of the building, can increase the mass and can add to the lateral stiffness (for example infill walls) of the building.

The architectural drawings are necessary to know the location of infill walls, secondary members and other non-structural components. If the drawings are not available then appropriate sketches and measurements are to be made. A suitable grid (commonly an orthogonal X-Y grid) is selected to identify the layout of the columns. The geotechnical report provides the soil profile and information on safe bearing capacity. The reports from data collection and preliminary evaluation (covered in Chapter 3) provide information on the variables to be used in subsequent analysis. The report from condition assessment provides information of the actual condition of the building and properties of the materials. These are more reliable than assuming certain values for the properties of the materials. Chapter 4 describes the tests for condition assessment.

For a building to be analysed, the vertical and lateral load resisting systems are first identified. The vertical load resisting system consists of slabs, beams, columns and footings. The lateral load resisting system can be moment resisting frames, braced frames, shear walls or a dual system comprising of both frames and shear walls. In general, a building should be modelled and analysed as a three-dimensional assembly of the members that completely represents the characteristics of the building such as distribution of the mass, strength, stiffness and deformability. Such a model is referred to as a 3D model. A 3D model is required for buildings with plan and vertical irregularities.

To reduce the time to build a model, the following simplifications can be done. The secondary beams that are supported on primary beams (beams that are supported by columns) need not be explicitly modelled. Only the loads from these beams are assigned on the primary beams. The secondary beams can be analysed separately for gravity loads. When only a slice of a building is modelled and analysed as a two-dimensional assembly of the members, it is referred to as a 2D model. It is quicker and easier to develop a 2D model. But the suitability of a 2D model is to be judged based on the type of building. A 2D model is adequate for a building where the torsional effects are either sufficiently small to be ignored, or are indirectly accounted for. The different aspects of developing a model are discussed next.

8.3.1 Material Properties

The basic material properties for the members required in a linear elastic analysis are the unit weight, modulus of elasticity, Poisson's ratio and shear modulus. Depending on the type of analysis and the material, other properties such as, yield stress and strain, ultimate (tensile or compressive) strength and corresponding strain, shear strength, coefficient of thermal expansion, creep coefficient, shrinkage strain, modulus of sub-grade reaction and modulus of compressibility of soil may be required.

For evaluation of member capacities, precise values of the material strength and the member dimensions are desirable. Chapter 4 on Condition Assessment of Buildings covers the non-destructive and intrusive techniques for determining the strength of the materials. If the values obtained from the design documents are used in absence of any testing, the values need to be modified to account for the uncertainty regarding the present condition of the material. A "knowledge factor" (m_k) is used to account for this uncertainty. The proposed values of the knowledge factor are shown in Table 8.2[1].

Table 8.2 Knowledge factors

No.	Description of available information	m_k
1	Original construction documents, including material testing report	1.0
2	Documentation as in (1) but no material testing undertaken	0.9
3	Documentation as in (2) and minor deteriorations of original condition	0.8
4	Incomplete but usable original construction documents	0.7
5	Documentation as in (4) and limited inspection and material test results with large variation.	0.6
6	Little knowledge about the details of components	0.5

[1] The table is adopted from "IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings" (2005).

8.3.2 Modelling of Members

1. Beams and columns

In a computer analysis program, the beams and columns can be modelled by frame elements. A frame element is a one-dimensional element that can carry bending moment, shear force and axial force, with degrees-of-freedom at the ends. While modelling the beams and columns, the properties to be assigned are the cross-sectional dimensions, the section properties such as area, moment of inertia, reinforcement details (for reinforced concrete members) and the types of material used. The plinth beams should also be modelled as frame elements.

The moment of inertia of a reinforced concrete (RC) section should be modelled properly to account for the effect of cracking and the contribution of the slab acting as a flange (for monolithic T- beam or inverted L- beam). The suggested effective moment of inertia (I_{eff}) for the beams including the effect of cracking and flanges are listed in Table 8.3.

Table 8.3 Effective moment of inertia for beam sections

Beam Sections	I_{eff}
Rectangular	$0.5\ I_g$
T - section	$0.7\ I_g$
Inverted L - section	$0.6\ I_g$

Here, the moment of inertia of the gross section (I_g) should be calculated considering the rectangular area only, as shown in Figure 8.1. In the case of columns, the reduction in stiffness due to cracking is reduced by the presence of axial compression. The suggested moment of inertia for column is $I_{eff} \equiv 0.7\ I_g$.

Figure 8.1 Rectangular area for the calculation of I_g (area shown shaded)

<u>2. Beam-column Joints</u>

The beam-column joints in a frame need not be modelled explicitly for most analysis. The effect of the joints is considered by connecting the members appropriately. For a braced frame, the connections are modelled as pinned. For a moment resisting frame, the connections are modelled as rigid. End-offsets can be specified for the elements to obtain the moments and shear forces at the beam and column faces (ends of clear length), rather than at the centre of the joints (ends of total length) (Figure 8.2).

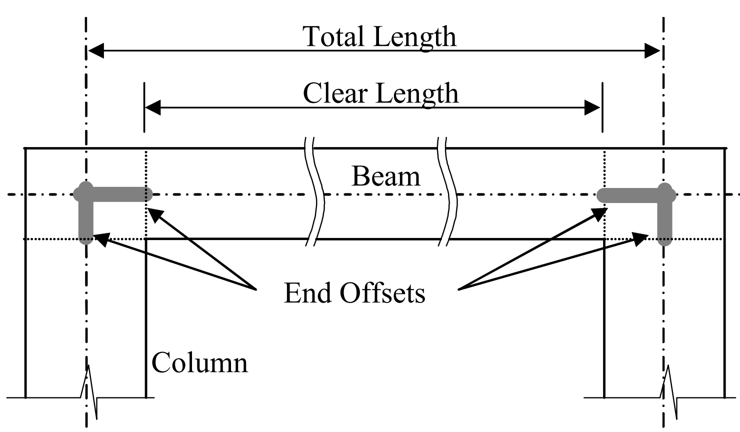


Figure 8.2 Use of end offsets at beam-column joints

For eccentric beam-column joints (centre-lines of beams do not pass through the centre-line of columns), the eccentricity of the centre lines should be modelled to consider the torsion in the columns.

Explicit modelling of a joint is required if the joint is weaker than the connected members, and/or the flexibility of the joint results in a significant increase in the drift of the building.

3. Slabs

The structural actions of the slab are two-fold.

1. The slab transfers the gravity loads to the supporting beams.
2. The slab mobilises the lateral load resisting systems, such as the frames and shear walls. This is referred to as the diaphragm action.

To simplify the model, the slabs need not be modelled explicitly by plate elements. A plate element refers to a two-dimensional element that can carry bending moment, shear force and twisting moment, with degrees-of-freedom at the corners. The transfer of gravity loads can be modelled by assigning the loads from the tributary areas of the slabs on to the beams. For a two-way slab, the triangular and trapezoidal tributary areas are considered for the beams along the shorter and longer sides of the slab, respectively (Clause 24.5, IS 456: 2000).

Based on the diaphragm action, the slabs can be divided into two types: rigid diaphragm and flexible diaphragm. A concrete slab or a metal deck with thick concrete topping has very large stiffness for in-plane deformation. This type of diaphragm is termed as a rigid diaphragm and it simultaneously mobilises all the lateral load resisting systems that are adequately attached to it. The structural effect of a rigid diaphragm can be modelled by assigning 'diaphragm action' at the floor level. The provision of diaphragm action is a constraint that does not allow any relative movement of the points in the diaphragm. The centre of mass (CM) is located on the diaphragm for assigning the lateral force for the level. The calculations for locating the CM are explained later. A wooden floor or a metal deck roof does not have sufficient stiffness to resist in-plane deformation. This type of diaphragm is termed as a flexible diaphragm. Each lateral load resisting system is considered to be mobilised only by the tributary area of the diaphragm independently. In the model, the diaphragm action is not assigned. A centre of mass is located for each lateral load resisting system. A fraction of the lateral force for the level, calculated based on the tributary area, is assigned at each CM.

In case of large openings in a slab, the different portions of the slab may have differential translations due to the reduced in-plane stiffness. In such a case, diaphragm action and lumped mass should be assigned separately to the different portions of the slab. The connecting portions of the slab (if any) can be modelled by two diagonally placed strut elements. A strut element is a one-dimensional element that can carry only axial force. The properties of each strut element can be taken as follows.

a. Thickness = thickness of the slab
b. Width = 3 × thickness of the slab
c. Elastic modulus = elastic modulus of the slab
d. Length = length of the diagonal of the slab.

The strut elements are connected by pin joints at the corners of the slab.

Figure 8.3 shows the plan of a residential apartment building. The four dwelling units are split into two portions with the lift well and stair case in between. The RC slab at a floor level should be modelled as two diaphragms.

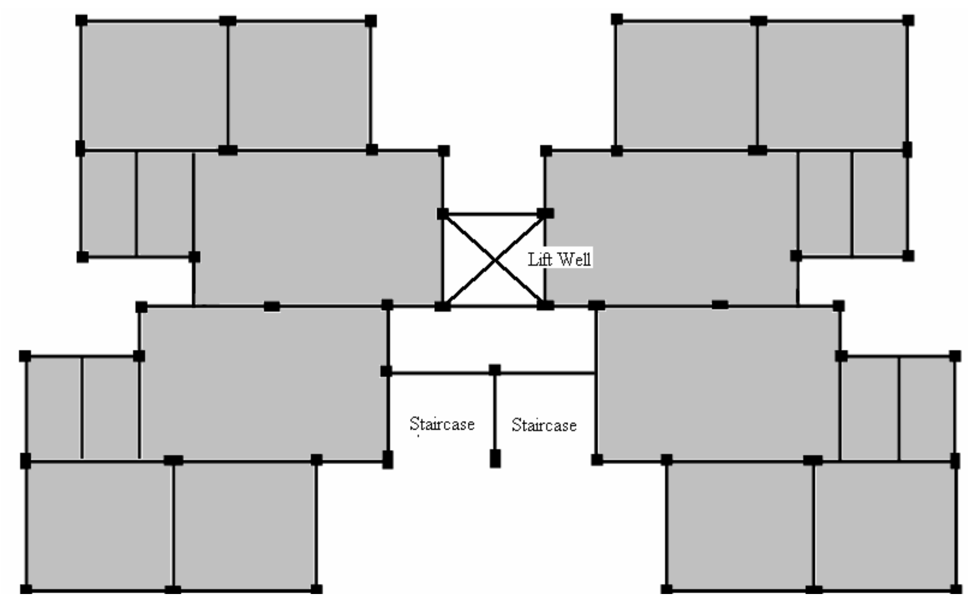


Figure 8.3 Floor modelled as two rigid diaphragms

4. Shear Walls and Core Walls

Shear walls and core walls (walls of a building core) can be the primary lateral load resisting system, if designed properly. They should be integrally connected to the floor slabs to consider their structural action. They can be modelled using equivalent column elements. The 'master' node of the column element can be at the centre of gravity of the shear wall or core and it should be connected to the 'slave' nodes of the adjacent beams by rigid links (Figure 8.4).

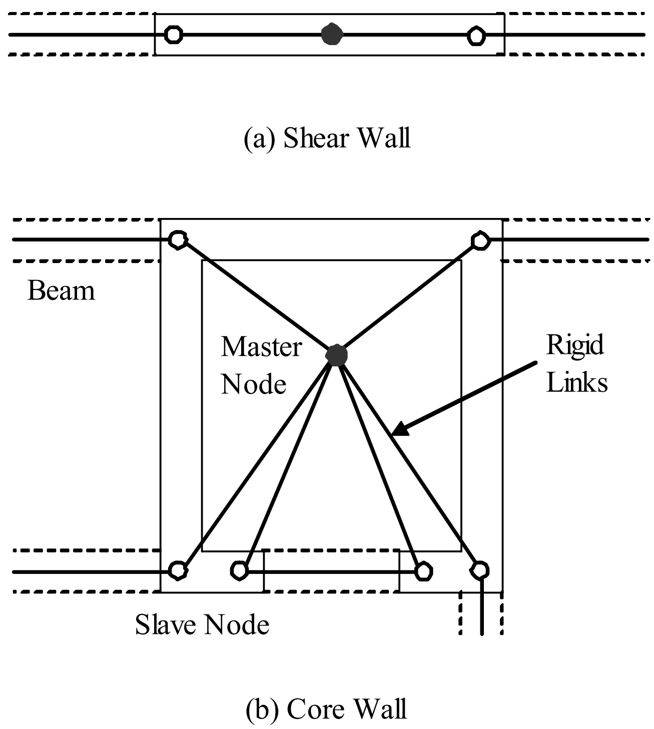


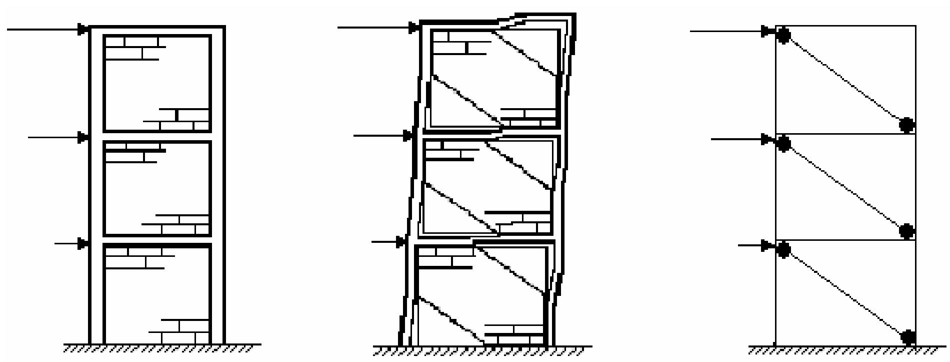
Figure 8.4 Modelling of a shear wall and a core wall

5. Appendages

The effects of all significant appendages (for example, stairways, cantilever slabs, water tanks) should be included in the model. The spandrel beams supporting the waist slab of a stairway can be modelled as inclined frame elements. The load from the waist slab should be assigned on the frame elements. For water tanks and cantilever slabs, the loads are to be assigned on the respective supporting elements.

6. Infill Walls

The infill walls made of masonry and placed within a frame are not designed as load bearing walls. But they add to the weight and lateral stiffness of the building. The weight of an infill wall should be assigned as a uniform load on the supporting beam. The stiffness contribution of an infill wall can be modelled using a simplified 'equivalent strut' approach. The equivalent strut is a one dimensional member that can carry only compression. The width of the strut can be 3 times the thickness of the wall or can be obtained from advanced analysis (FEMA 273, Smith and Carter, 1969, Saneinejad and Hobbs, 1995). The thickness and elastic modulus of the strut are same as those of the wall. The bounding beams and columns are connected by rigid joints, but the equivalent struts are connected by pin joints at the beam-column joints (Figure 8.5).



a) Infilled frame　　b) Deformed shape　　c) Equivalent compression strut model

Figure 8.5 Modelling of infill walls

When the stiffness contribution of the infill walls is included by the equivalent strut model, the natural period of the building is reduced and the base shear may increase. But, the moments in the beams and columns may reduce due to the 'truss' action of the equivalent struts. During an earthquake, the infill walls may fail due to out-of-plane bending. This will increase the moments in the beams and columns. To calculate the demands in the beams and columns, two extreme cases can be modelled. In the first model, the lateral stiffness due to the infill walls is modelled by the equivalent struts. In the second model, the stiffness is ignored. However, the weight of the infill walls on the supporting beams should be considered in both the models.

7. <u>Column Ends at Foundation</u>

The modelling of the ends of the columns at the foundation is important, and appropriate boundary conditions need to be imposed. The ends can be modelled by considering the degree of fixity provided by the foundation. Depending on the type of footing, the end condition may be modelled as follows.

a. *Isolated footing*: A hinge is to be provided at the column end at the bottom of the foundation. However, when it is founded on hard rock strata, the column end may be modelled as fixed, with the level of fixity at the top of the footing. The definition of hard rock is covered in Chapter 11, Geotechnical Seismic Hazards. If the columns are connected by plinth beams, the later has to be modelled by frame elements.

b. *Raft foundation*: The column ends are to be modelled as fixed at the top of the raft.

c. *Combined footing*: Engineering judgement must be exercised in modelling the fixity provided by the combined footings. If the footings are adequately restrained by tie beams, the column ends can be modelled as fixed.

d. *Single pile*: Depending upon the type of soil, fixity of column is recommended at a depth of five to ten times the diameter of pile, from the top of pile cap.

e. *Multiple piles*: Assume fixity of column at top of the pile cap.

8.3.3 Modelling of Loads

The minimum load cases to be considered are dead load, live load and earthquake load. The values of dead load can be calculated from the unit weights as specified in IS 875: 1987, Part 1. The dead loads are explicitly assigned for the slabs, infill walls and non-structural components. For the members modelled as frame elements, the in-built option of the program to calculate the self weights may be utilised. The live load intensities for the various areas of the building can be obtained from IS 875: 1987, Part 2. The live loads are assigned on the beams supporting the slabs. The earthquake loads are obtained based on the method of analysis. These are explained in the subsequent sections. The earthquake loads are assigned along one of the X- or Y- directions at a time.

8.3.4 Load Combinations

After the computational model is developed and the loads are assigned, the model is analysed for the individual load cases. The internal forces in the members (such as bending moment, shear force and axial force) for the individual load cases are combined as per the following load combinations (IS 1893: 2002, Section 6.3).

$$\begin{aligned}
COMB1 &= 1.5\,(DL + IL) \\
COMB2 &= 1.2\,(DL + IL + EL) \\
COMB3 &= 1.2\,(DL + IL - EL) \\
COMB4 &= 1.5\,(DL + EL) \qquad \mathbf{(8.1)} \\
COMB5 &= 1.5\,(DL - EL)
\end{aligned}$$

$COMB6 = 0.9DL + 1.5EL$

$COMB7 = 0.9DL - 1.5EL$

Here, DL denotes the internal forces due to dead load, IL denotes the internal forces due to live load, and EL denotes the internal forces due to earthquake load. The maximum value from the above load combinations gives the demand for a particular internal force.

When the lateral load resisting systems are oriented along the X- and Y- directions, the value of EL is due to earthquake along X- or Y- directions, one at a time. But when the lateral load resisting systems are not oriented along the X- and Y- directions, the internal forces due to earthquakes along the X- and Y- directions are combined. One method to combine them is with the following weighting factors for the individual forces.

(a) 100% of the forces due to earthquake in X-direction and 30% of the forces due to earthquake in Y-direction.
(b) 100% of the forces due to earthquake in Y-direction and 30% of the forces due to earthquake in X-direction.

An alternative method to combine the forces due to earthquakes in the two directions is the square root of the sum of the squares (SRSS) rule.

$$EL = \sqrt{EL_x^{\,2} + EL_y^{\,2}} \tag{8.2}$$

Here, EL_x denotes the internal forces due to earthquake along X-direction, El_y denotes the internal forces due to earthquake along Y-direction. The vertical component of the ground motion is considered only for special elements like cantilevers in seismic Zones IV and V.

8.3.5 Interpretation of Results

From the output, the following results are of primary interest.

1. The values and diagrams of the internal forces (axial force, bending moment and shear force) in the columns and shear walls.
2. The values and diagrams of the internal forces (bending moment and shear force) in the beams.
3. The values of the internal force (axial force) in the braces, equivalent struts for infill walls and slab connecting two diaphragms.
4. The reactions at the columns at foundation.
5. The vertical deflections of beams and lateral drifts of storeys.

6. The natural periods and mode shapes of the model for response spectrum method of analysis.
7. The time-wise variations of the internal forces in the beams and columns for time history analysis.

The different types of analysis are explained in the following section.

The demand for an internal force is compared with the corresponding capacity. A convenient way to study this is to check the demand-to-capacity ratio (DCR). The DCR should be less than 1.0 for the members in the lateral load resisting systems. The lateral drift of a storey is compared with the specified limit (IS 1893: 2002, Clause 7.11.1).

8.4 EQUIVALENT STATIC ANALYSIS

The equivalent static analysis is widely employed for single storey buildings, and can be used for all regular buildings up to six storeys. The calculations are based on applying horizontal forces statically. The results of this analysis can be very inaccurate when applied to highly irregular buildings, unless the building is capable of responding to the design earthquake in a nearly elastic manner. Therefore, this analysis should not be used for highly irregular buildings.

The equivalent static analysis is also known as the “equivalent lateral force procedure” or “seismic coefficient method”. The calculations are relatively simple and can be performed by hand, although a number of computer programs are available to facilitate the analysis. In this analysis, first the base shear along X- or Y- direction is calculated in terms of a spectral acceleration coefficient, the weight of the building and a few other variables. Next, the base shear is distributed at the various levels along the height of the building approximately based on the first mode of vibration. The lateral force at a level is applied at the design centre of mass location. Finally, the model is analysed for the applied forces. The essential features of the analysis are elaborated here.

8.4.1 Calculation of Base Shear

The base shear (V_B) is calculated as per Clause 7.5.3 of IS 1893: 2002.

$$V_B = A_h W \tag{8.3}$$

W is the total seismic weight of the building. The seismic weight of each floor of the building includes the dead load and fraction of the live load (as per Table 8 of IS 1893: 2002) acting on the floor. The weight of the columns and walls (up to the tributary height) are to be included. The tributary height is between the centre-line of the storey above and centre-line of the storey below.

The horizontal seismic coefficient (A_h) is given as follows.

$$A_h = \frac{Z}{2} \frac{I}{R} \frac{S_a}{g} \tag{8.4}$$

The variables are as follows.

Z = zone factor

I = importance factor

R = response reduction factor

S_a/g = spectral acceleration coefficient.

The value of S_a/g is determined from the response spectrum corresponding to an approximate time period (T_a) which is given in seconds by the following equations.

For RC buildings,

$$T_a = 0.075\, h^{0.75} \tag{8.5a}$$

For steel buildings,

$$T = 0.085\, h^{0.75} \tag{8.5b}$$

For frame buildings with masonry infill walls and other buildings,

$$T_a = \frac{0.09h}{\sqrt{d}} \tag{8.5c}$$

The height of the building measured from the base is represented as h (in metres). The horizontal dimension of the building at the base along the direction of lateral forces is represented as d (in metres).

The response spectrum is a plot of the maximum response (displacement, velocity, acceleration or any other quantity of interest) to a specified load function for single degree-of-freedom systems of different time-periods. The abscissa of the spectrum is the natural period (or frequency) of the system and the ordinate is the maximum response. The maximum response depends on the damping and type of soil. Figure 8.6 shows the response spectrum for acceleration for a 5 percent damped system. For other values of damping, factors recommended in Table 3, IS 1893: 2002, can be used to modify the values of the ordinate.

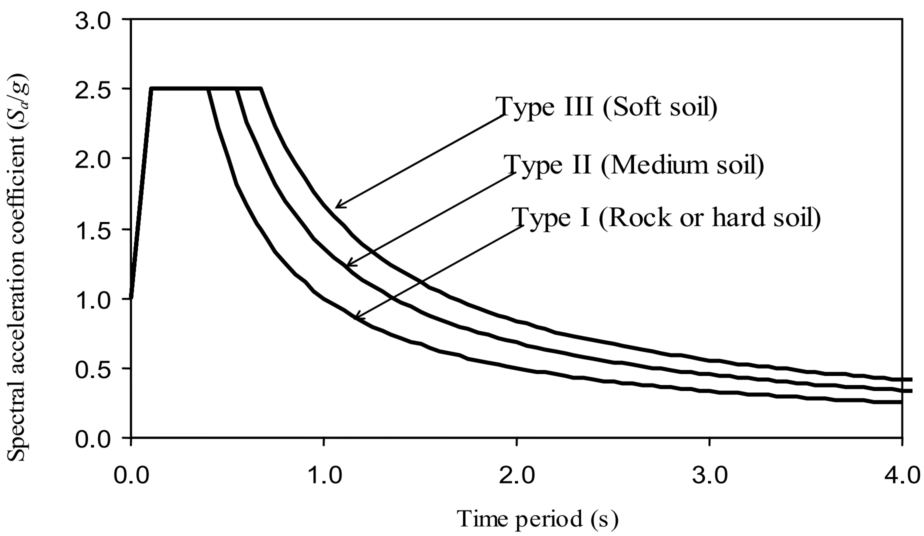


Figure 8.6 Response spectrum for acceleration for 5 percent damping

If the retrofitting of an existing building for the design base shear as per Equation 8.3 is not warranted due to limited resources, the value can be reduced. A proposed model for the reduction of the design base shear is based on the remaining useful life of the building (Draft Indian Standard on Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines). The modification factor for the reduction of the base shear is given as follows.

$$U = \left(\frac{T_{rem}}{T_{des}}\right)^{0.5} \qquad \textbf{(8.6)}$$

$$\geq 0.7$$

Here,

T_{rem} = remaining useful life of the building

T_{des} = design useful life of the building.

8.4.2 Distribution of Base Shear

The base shear (V_B) is distributed to the floor levels as per the expression of Clause 7.7.1 of IS 1893: 2002.

$$Q_i = V_B \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \tag{8.7}$$

Here,

Q_i = design lateral force at floor i
W_i = seismic weight of floor i
h_i = height of floor i measured from the base (Figure 8.7)
n = number of floors, including roof.

The shear at a storey is the sum of the lateral forces acting at all the floors above the storey.

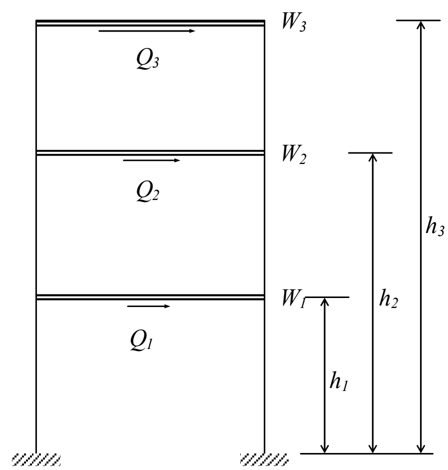


Figure 8.7 Variables used in distribution of base shear

8.4.3 Effect of Torsion

The effect of horizontal torsion must be considered for buildings with rigid diaphragms. The individual torsional moment at a given floor level is the product of the lateral force Q_i and the design eccentricity (e_{di}) between the centre of mass (CM) of the floor and the centre of rigidity (CR) of the vertical lateral load resisting elements in the storey below the floor.

The design eccentricity is considered to be a sum of two components. The first component is based on the distance between the calculated CM and calculated CR. This eccentricity is referred to as the static eccentricity. The second component considers an accidental torsional moment, which is produced by an offset of the CM from the CR. To simplify the modelling, the calculated CM is shifted to a 'design' CM based on the design eccentricity from the CR. The total moment acting at a storey is sum of all the individual torsional moments at the floors above the storey.

The calculations for locating the CM and CR and the design eccentricity are explained next.

<u>1. Centre of mass of a floor</u>

The CM is the point where the total mass of the floor level is assumed to be lumped. The CM can be calculated by taking moments of the masses / weights of the different segments of the floor and tributary portions of the columns and walls, about assumed reference axes. Else, it can be calculated by taking moments of the axial forces (from gravity load analysis of that floor only) in the columns.

$$CM_x = \frac{\sum W_i x_i}{\sum W_i}, CM_y = \frac{\sum W_i y_i}{\sum W_i} \quad \textbf{(8.8)}$$

Here,

CM_x = coordinate of the centre of mass along X-direction
CM_y = coordinate of the centre of mass along Y-direction
$\sum W_i$ = sum of the weights of all segments of the floor, columns and walls
$\sum W_i x_i$ = sum of the moments of weights about Y-axis
$\sum W_i y_i$ = sum of the moments of weights about X-axis.

2. Centre of rigidity of a storey

The CR is the point through which the resultant of the restoring forces in a storey acts when the storey undergoes translation. The CR for each storey should be found out separately. There are different procedures to calculate the CR. One of the procedures is explained below. The columns of the storey are assumed to be fixed at the bottom. A unit force along the X-direction and a unit moment about the Z- axis (vertical axis) are applied separately at a certain test point in the top of the storey and the corresponding rotations are noted down. The distance of the CR from the test point, along Y- direction, is calculated from the ratio of the two rotations. Similarly the distance along X- direction is found out by applying a unit force along the Y-direction and a unit moment.

Let the co-ordinates of the test point be (x, y). Let $(\theta_z)_x$, $(\theta_z)_y$ and $(\theta_z)_z$ be the rotations about the Z-axis for the unit loads along X- and Y- directions and unit moment about Z-axis, respectively. The co-ordinates of the CR are given as $CR_x = x+x^1$, $CR_y = y+y^1$, where,

$$x^1 = -\frac{(\theta_z)_x}{(\theta_z)_z} \quad \textbf{(8.9a)}$$

$$y^1 = \frac{(\theta_z)_x}{(\theta_z)_z} \quad \textbf{(8.9b)}$$

The static eccentricity of the CM with respect to the CR is given as follows.

$$e_{six} = CM_x - CR_x \quad \textbf{(8.10a)}$$

$$e_{siy} = CM_y - CR_y \quad \textbf{(8.10b)}$$

The design eccentricity of the CM (e_{dix}, e_{diy}) is calculated considering a dynamic amplification factor and an additional eccentricity of 5 percent of the dimension of the building perpendicular to the direction of the seismic force. For either of X- or Y- directions,

$$e_{di} = 1.5e_{si} - 0.05b_i \quad \textbf{(8.11a)}$$

or,

$$e_{di} = e_{si} - 0.05b_i \quad \textbf{(8.11b)}$$

There can be four possible locations of the design CM. To reduce computation, only two appropriate diagonal locations can be considered.

3. Lumped mass of a floor

The lumped mass of each floor is the total mass that is lumped at the design CM of the respective floor. The total mass of a floor is obtained from the seismic weight of that floor divided by the acceleration due to gravity. The magnitudes and locations of the lumped masses of the floors are used to calculate the natural periods and mode shapes.

The equivalent static analysis can be used unless one or more of the following conditions apply. In these cases the response spectrum method, described in the next section, should be used.

- For a regular building, if the building height exceeds 40 m in Zones IV and V or exceeds 90 m in Zones II and III.
- For an irregular building, if the building height exceeds 12 m in Zones IV and V or exceeds 40 m in Zones II and III.
- The ratio of the building's horizontal dimensions at any storey to the corresponding dimensions at an adjacent storey exceeds 1.4 (excluding penthouse).
- The building is found to have a severe torsional stiffness irregularity in any storey. A severe torsional stiffness irregularity may be deemed to exist in a storey if the diaphragm above the storey is rigid, and the results of the analysis indicate that the drift along any side of the structure is more than 150 percent of the average storey drift.
- The building is found to have a severe vertical mass or stiffness irregularity. A severe vertical mass or stiffness irregularity may be deemed to exist when the average drift in any storey (except penthouses) exceeds that of the adjacent storeys by more than 150 percent.
- The building has a non-orthogonal lateral force-resisting system.

According to IS 1893: 2002, high rise and irregular buildings must be analysed by the response spectrum method. However, this method can also be used for regular buildings.

8.5 RESPONSE SPECTRUM METHOD

The response spectrum method is suitable for irregular buildings. The calculations are based on the likely maximum values of the response quantities from the equations of motion. The method is applicable for all buildings, except those incorporating supplemental energy dissipation devices and some types of base isolation systems.

The response spectrum method is also known as the "modal analysis procedure" and can be performed in accordance with the requirements of Clause 7.8.4, IS 1893: 2000. The method is based on superposition of modes. Hence, free vibration modes are computed using eigenvalue analysis. The maximum value of a quantity (say λ_k) termed as the modal response, is obtained for each mode (say k^{th} mode). The number of modes considered is based on a quantity termed as the mass participation factor for each mode. Sufficient number of modes (r) to capture at least 90 percent of the total participating mass of the building (in each of the horizontal directions), should be considered in the analysis. The modal responses from all the considered modes are then combined together using either the square root of the sum of the squares (SRSS) method or the complete quadratic combination (CQC) method. The SRSS method is expressed as follows.

$$\lambda = \sqrt{\sum_{k=1}^{r} (\lambda_k)^2} \qquad \textbf{(8.12)}$$

If the building has very closely spaced time periods, then the CQC method is preferable.

The base shear for response spectrum method is calculated in the following manner. The S_a/g value corresponding to each period is first calculated from the response spectrum (Figure 8.6). The base shear corresponding to each period is then calculated using Equations 8.3 and 8.4. Each base shear is multiplied with the corresponding mass participation factor and then combined as per the selected mode combination method, to get the total base shear for the building. The subsequent calculations for the distribution of base shear and the analysis are similar to the equivalent static method.

If the base shear calculated from the response spectrum analysis $(\bar{V}_B)$ is less than the design base shear (V_B) calculated from Equation 8.3, then as per Clause 7.8.2, IS 1893: 2002, all the response quantities (member forces, displacements, storey shears and base reactions) from the response spectrum analysis have to be scaled up by the factor $V_B / \bar{V}_B$.

8.6 TIME HISTORY ANALYSIS

In time history analysis, the equations of motion representing the response of a building to ground motion are solved. From the solution, the variations of axial force, bending moment and shear force in a member can be noted. The maximum value of an internal force is selected for subsequent calculation of the demand under load combinations.

The equations of motion can be compactly written as follows.

$$\mathbf{M\ddot{u}}(t)+\mathbf{C\dot{u}}(t)+\mathbf{Ku}(t)=\mathbf{I_x}\ddot{u}_{gx}(t)+\mathbf{I_y}\ddot{u}_{gy}(t)+\mathbf{I_z}\ddot{u}_{gz}(t) \quad \textbf{(8.13)}$$

Here, **M** is the diagonal mass matrix, **C** is the proportional damping matrix, **K** is the stiffness matrix, $\mathbf{\ddot{u}}$, $\mathbf{\dot{u}}$ and $\mathbf{u}$ are the relative (with respect to the ground) acceleration, velocity and displacement vectors, respectively, $\mathbf{I_x}$, $\mathbf{I_y}$, and $\mathbf{I_z}$ are the unit acceleration loads and $\ddot{u}_{gx}$, $\ddot{u}_{gy}$ and $\ddot{u}_{gz}$ are the three components of ground acceleration. The time-wise variation can be the same as a recorded earthquake or can be generated based on the geologic conditions of the building site. The equations of motion can be solved by numerical techniques (Clough and Penzien, 1993).

8.7 NEED FOR NON-LINEAR ANALYSIS

> A building subjected to earthquake is expected to show inelastic behaviour, that is, the deformation in a member does not remain proportional to the internal force. A non-linear analysis accounts for the inelastic response. The calculated internal forces are better estimates than the values obtained from a linear analysis. A non-linear analysis also accounts for the redistribution of forces that occur in a structure as parts of it undergo inelastic response.

In order to determine whether a building may be analyzed with sufficient accuracy by the linear procedures, it is necessary to examine the results to determine the magnitude and distribution of the inelastic demand on the various members of the lateral load resisting systems. The magnitude of the inelastic demand is indicated by the demand-to-capacity ratio (DCR) for each internal force (such as axial force, bending moment and shear force) of each member. If all the DCRs for a member are less than or equal to 1.0, then the member is expected to respond elastically to the earthquake ground shaking of design intensity. If one or more of the computed DCRs for a member is greater than 1.0, then the member is expected to respond inelastically to the ground shaking. The largest DCR calculated for a given member identifies the critical action for the member, that is, the action in which the member will first yield or fail. This DCR is termed as the critical component DCR. If the DCRs computed for all the actions of all the members (such as beams, columns, walls and braces) are less than 2.0, then linear analysis procedures are applicable, regardless of regularity.

If some computed DCRs exceed 2.0, then linear methods should not be used if any one of the following applies.

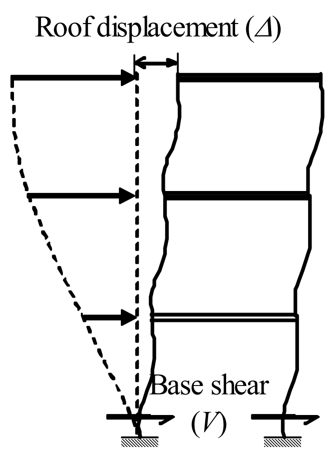
- There is an in-plane discontinuity in any member of the lateral load resisting system
- There is an out-of-plane discontinuity in any member
- There is a weak storey in any direction of the building.
- There is a torsional irregularity in any storey.

The irregularities are explained in Chapter 2.

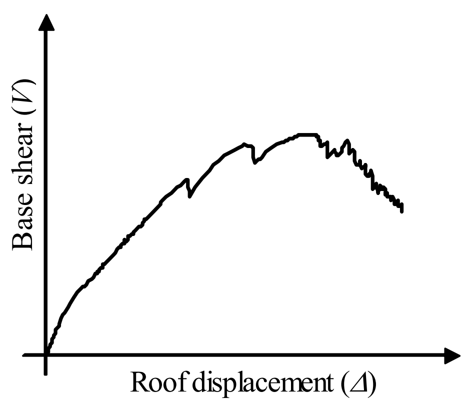
8.8 PUSHOVER ANALYSIS

Pushover analysis is a form of non-linear analysis, where the magnitudes of the lateral loads are incrementally increased, maintaining a pre-defined distribution pattern along the height of the building, until a collapse mechanism develops in the building. With the increase in the loads, non-linear responses of the members are modelled.

The pushover analysis can determine the lateral load versus deformation behaviour of a building corresponding to the incremental load. Programs supporting pushover analysis provide elegant visualisation of the damage state for each load step and the redistribution of the internal forces in the members. At each step, the base shear (total lateral force) and the roof displacement can be plotted to generate the pushover curve (Figure 8.8). It gives an idea of the lateral strength and the maximum inelastic drift the building can sustain. For regular buildings, it can also give a rough estimate of the lateral stiffness of the building.



a) Building model



b) Pushover curve

Figure 8.8 Pushover analysis

8.8.1 Capacity Spectrum, Demand Spectrum and Performance Point

Instead of plotting the base shear versus roof displacement, the base acceleration can be plotted with respect to the roof displacement (Figure 8.9). This curve is called the *capacity spectrum*. Simultaneously, the acceleration and displacement spectral values as calculated from the corresponding response spectrum for a certain damping (say 5 percent initially), are plotted as the ordinate and abscissa, respectively. The representation of the two curves in one graph is termed as the Acceleration versus Displacement Response Spectrum (ADRS) format. With increasing non-linear deformation of the components, the equivalent damping and the natural period increase. The spectral values of the acceleration and displacement can be modified from the 5 percent damping curve by multiplying a factor corresponding to the effective damping (Table 3, IS 1893: 2002). Thus, the instantaneous spectral acceleration and displacement point (demand point) shifts to a different response spectrum for higher damping. The locus of the demand points in the ADRS plot is referred to as the *demand spectrum*. The demand spectrum considers the inelastic deformation of the building.

The *performance point* is the point where the capacity spectrum crosses the demand spectrum. If the performance point exists and the damage state at this point is acceptable, then the building is considered to be adequate for the design earthquake.

It must be emphasised that the pushover analysis is approximate in nature and is based on a statically applied load. It estimates an envelope curve for the behaviour under the actual dynamic load. Moreover, the analysis cannot predict accurately the higher mode responses of a flexible building. Therefore, it must be used with caution while interpreting the actual behaviour under seismic load.

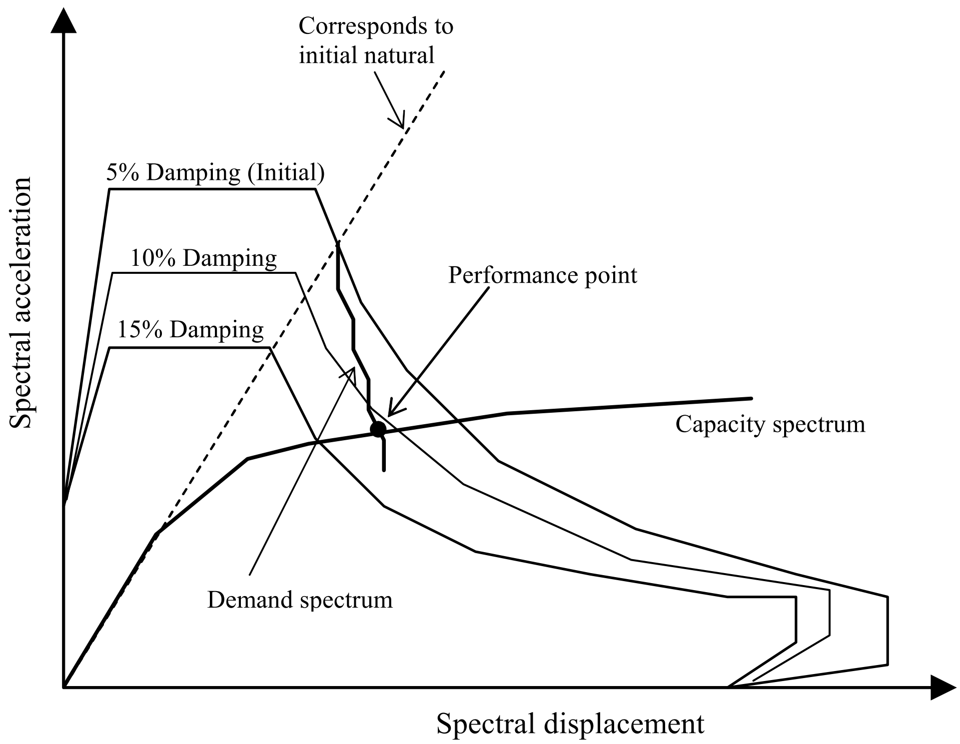


Figure 8.9 Demand and capacity spectra

8.8.2 Procedure for Pushover Analysis

A pushover analysis involves the application of increasing lateral forces or displacements to a computational model of a building. The analysis can be executed in two ways, force-controlled or displacement-controlled. In a *force-controlled* push, the forces are increased monotonically until either the total force reaches a target value or, the building has a collapse mechanism. In a *displacement-controlled* push, the displacements are increased monotonically until either the displacement of a pre-defined control node in the building model reaches a target value or, the building has a collapse mechanism. For convenience, the control node can be taken at the design centre of mass of the roof of the building. The target displacement is intended to represent the maximum displacement likely to be experienced during the earthquake. The essential requirements of pushover analysis are explained next.

i) Distribution of Lateral Loads

The pushover analysis requires the distribution of lateral loads, which are applied incrementally. Frequently, an inverted triangular distribution or the distribution same as the first mode shape is used. The load distribution pattern given in IS 1893: 2002 can be used for low- to mid-rise buildings (Equation 8.7 with the value of V_B varying). The importance of the load distribution increases for tall buildings, whose earthquake response is not dominated by a single mode shape. For such buildings, the load distribution based on the first mode shape may seriously under-estimate the loads on the intermediate floor levels.

The pushover analysis should be first carried out along the vertical direction followed by the analysis along two orthogonal horizontal directions separately. Therefore, there are three pushover cases for evaluating a building.

1. The gravity push, which is carried out to apply the gravity loads.
2. The lateral push in X-direction, starting at the end of gravity push.
3. The lateral push in Y-direction, starting at the end of gravity push.

Initially, the gravity loads are increased in a force-controlled manner till the total load reaches the target value. The target value can be the same as the design gravity load for a linear analysis. Next, the lateral loads are applied in the X- or Y- direction, in a displacement-controlled manner. The direction of monitoring of the behaviour is same as the direction of lateral load.

ii) Load versus Deformation Behaviour of Elements

As the forces or displacements are increased, some elements may undergo inelastic deformation. Thus, it is necessary to model the non-linear load versus deformation behaviour of an element under each of the internal force (bending moment, shear force or axial force). The non-linear behaviour is modelled by assigning appropriate load versus deformation property at a location of the element. These properties are referred to as *hinge properties*. The beam, column and shear wall elements should have moment versus rotation and shear force versus shear deformation hinge properties. For column and shear wall elements, the moment versus rotation property should be calculated for the level of axial load available from the conventional gravity load analysis. The brace and equivalent strut elements have to be modelled with axial load versus axial deformation hinge properties.

A typical moment versus rotation curve for a beam element and the corresponding idealised curve are shown in Figure 8.10. The second curve is a piece-wise linear curve defined by the following five points[2].

1. Point 'A' corresponds to the unloaded condition.
2. Point 'B' corresponds to the onset of yielding.
3. Point 'C' corresponds to the ultimate moment of resistance (M_{uR}).
4. Point 'D' corresponds to the drop in strength beyond C. In the absence of the modelling of the descending branch of a load versus deformation curve, the drop is considered to be vertical. For the computational stability, it is recommended to specify a non-zero residual strength, say 20 percent of the ultimate strength.
5. Point 'E' corresponds to the maximum deformation capacity with the residual strength. To maintain computational stability, a high value of deformation capacity can be assumed.

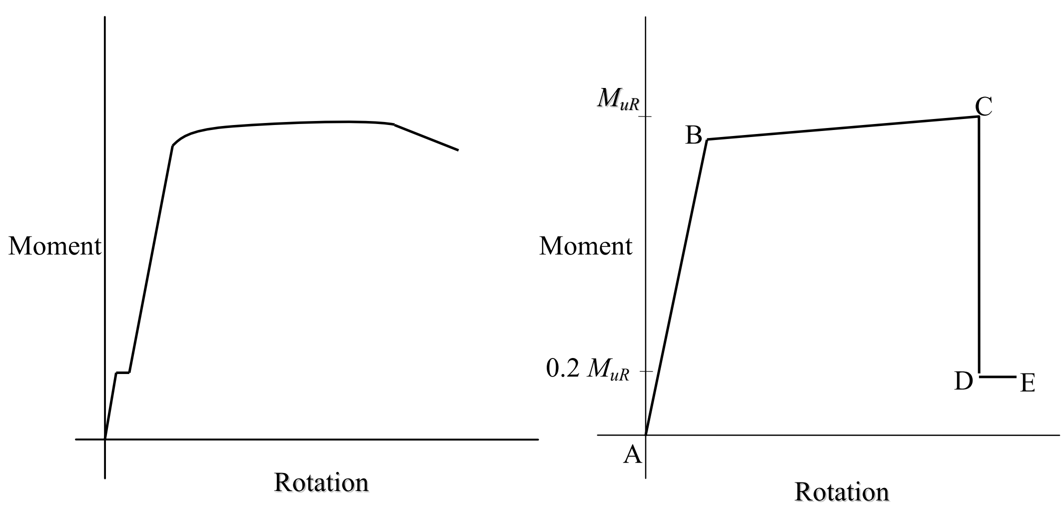


Figure 8.10 (a) Typical moment versus rotation curve for a beam element, (b) Idealised curve

[2] The load versus deformation hinge property is assigned with performance levels in a performance based analysis. Recommendations for the performance levels are given in ATC 40 and FEMA 356.

8.8.3 Target Displacement

In the displacement controlled lateral push, the target displacement δ_t for a building with rigid diaphragms at each floor level can be estimated using a procedure that accounts for the likely non-linear response of the building. The target displacement is given by the following equation (FEMA 356).

$$\delta_t = C_0 C_1 C_2 C_3 S_a \left(\frac{T_e}{2\pi}\right)^2 \quad \textbf{(8.16)}$$

Here,

T_e = effective fundamental period of the building (in seconds) in the direction under consideration
= $T_i \sqrt{(K_i/K_0)}$

T_i = elastic fundamental period in the direction under consideration

K_i = elastic lateral stiffness of the building in the direction under consideration

K_e = effective lateral stiffness of the building determined from the base shear versus roof displacement curve (Figure 8.11)

C_0 = modification factor to relate the spectral displacement of an equivalent single degree of freedom system to the roof displacement. Suggested values are given in Table 8.4.

C_1 = modification factor to relate the expected maximum inelastic displacement to the displacement calculated for linear elastic response.
= 1.0 for $T_e \geq T_s$
= $[1.0 + (R - 1)\, T_s/T_e]/R$ for $T_e < T_s$
but need not exceed 1.5 for $T_e < 0.10$ second.

T_s = the period of the response spectrum associated with the transition from the acceleration governed segment of the spectrum (where the acceleration response is constant) to the velocity governed segment of the spectrum (where the acceleration response starts dropping)

R = ratio of elastic strength demand to calculated yield strength
= S_a . (W/V_y). $(1/\ C_0)$

S_a = spectral acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration.

W = seismic weight of the building

V_y = yield strength calculated using pushover analysis, where the non-linear base shear versus roof displacement curve of the building is replaced by a bilinear curve (Figure 8.11).

C_2 = modification factor to represent the effect of the shape of the hysteresis loop on the maximum displacement response. Suggested values for C_2 are given in Table 8.5.

C_3 = modification factor to represent increased displacement due to the P-Δ effects.
For buildings with positive post-yield stiffness, $C_3 = 1.0$.
For buildings with negative post-yield stiffness, $C_3 = 1.0 + [|\alpha|(R - 1)^{1.5}/T_e]$.

α = ratio of post-yield stiffness to elastic stiffness (Figure 8.11).

Table 8.4 Values for modification factor C_o

Number of Stories	**Modification Factor C_0**
1	1
2	1.2
3	1.3
5	1.4
10+	1.5

Table 8.5 Values for modification factor C_2

Performance Level	$T = 0.1$ second	$T \geq T_s$
Immediate Occupancy	1.0	1.0
Life Safety	1.3	1.1
Collapse Prevention	1.5	1.2

The performance levels are explained in FEMA 356.

To estimate K_e, the capacity curve of the base shear versus the roof displacement is replaced with a bilinear curve as indicated in Figure 8.11. K_e is taken as the secant stiffness calculated at a base shear force equal to 60 percent of the yield strength (V_y).

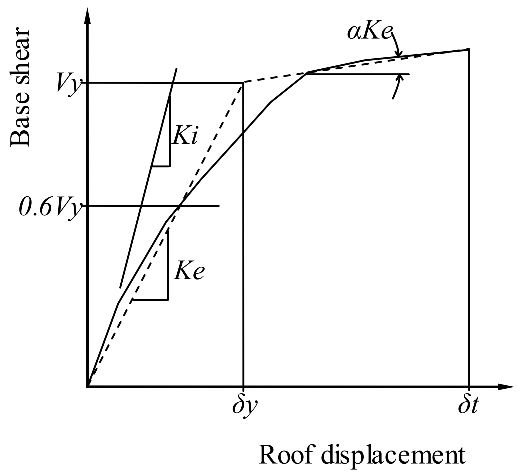


Figure 8.11 Calculation of effective stiffness K_e

For a building with flexible diaphragms at each floor level, a target displacement shall be estimated for each line of lateral load resisting system. The target displacement for an individual line of lateral load resisting system is given by Equation 8.16. The fundamental period of each lateral load resisting system is calculated based on the mass assigned at each level as per the tributary area.

8.9 SUMMARY

The different methods of analysing a building for seismic forces are first explained. Next, the section on computational model covers the important aspects related with material properties, structural elements, applied loads and interpretation of the results. The equivalent static analysis is explained along with the calculation of base shear, distribution of the base shear and the effect of torsion. The fundamentals of the response spectrum method and the pushover analysis are elucidated.

8.10 REFERENCES

1. ATC 40 (1996), "Seismic Evaluation and Retrofit of Concrete Buildings: Vol. 1", Applied Technology Council, USA.

2. FEMA 273 (1997), "NEHRP Guidelines for the Seismic Rehabilitation of Buildings", Applied Technology Council, USA.
3. FEMA 356 (2000), "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", American Society of Civil Engineers, USA.
4. IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of Buildings (2005), Indian Institute of Technology Kanpur. Project sponsored by the Gujrat State Disaster Mitigation Authority.
5. IS 456: 2000, "Plain and Reinforced Concrete – Code of Practice", Bureau of Indian Standards.
6. IS 800 (Draft), "Indian Standard Code of Practice for General Construction in Steel", Bureau of Indian Standards.
7. IS 875: 1987, "Code of Practice for Design Loads (other than earthquake) for Buildings and Structures", Part 1: Dead Loads – Unit Weights of Building Materials and Stored Materials, Part 2: Imposed Loads, Bureau of Indian Standards.
8. IS 1893: 2002, "Criteria for Earthquake Resistant Design of Structures", Part 1: General Provisions and Buildings, Bureau of Indian Standards.
9. IS 1905: 1987, "Code of Practice for Structural Use of Un-reinforced Masonry", Bureau of Indian Standards.
10. Draft Indian Standard on Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines, Bureau of Indian Standards.
11. Agarwal, P. and Shrikhande, M. (2005), "Earthquake Resistant Design of Structures", Prentice-Hall of India Pvt. Ltd.
12. Chopra, A. K. (2004), "Dynamics of Structures", Second Edition, Prentice-Hall of India Pvt. Ltd.
13. Clough, R. and Penzien J. (1993), "Dynamics of Structures", Second Edition, McGraw-Hill, Inc.
14. Saneinejad, A. and Hobbs, B. (1995), "Inelastic Design of Infilled Frames", Journal of Structural Engineering, American Society of Civil Engineers, Vol. 121, No. 4, pp. 634 – 650.
15. Smith, S. B. and Carter, C. (1969), "A Method of Analysis for Infilled Frames", Proceedings of Institution of Civil Engineers, Vol. 44, pp. 31 – 48.

9

RETROFIT OF REINFORCED CONCRETE BUILDINGS

9.1 OVERVIEW

Reinforced concrete (RC) buildings include residential, dormitory, institutional, office, commercial and industrial buildings. The multi-storeyed buildings have moment resisting frames consisting of a framework of RC beams and columns. The frames are intended to carry vertical gravity loads as well as resist the lateral earthquake forces. Some buildings have shear walls or braces in addition to frames. Recent buildings have flat slabs / flat plates on column system to carry the vertical loads. In the seismic regions, these buildings should be augmented with shear walls or braces to carry the lateral loads.

The performance of a frame under lateral loads generates from the flexural action of beams and columns and the flexural rigidity of the beam-column joints. There should be adequate number of well laid out frames in the two orthogonal directions in order to generate the lateral stiffness and strength of a building. Some existing building configurations that are adequate for resisting gravity loads, are not suitable for resisting earthquake forces. In this chapter, first the common deficiencies observed in existing RC buildings for resisting earthquake forces are identified. It is essential to identify the deficiencies in a building before undertaking retrofit. Identification of the deficiencies is also expected to create awareness for future construction.

For convenience, the deficiencies can be broadly classified as global deficiencies and local deficiencies. Global deficiencies refer to the deficiencies which are observed in the building as a whole. These deficiencies are subjective in nature. But for the purpose of evaluation, IS 1893: 2002 (Part 1) provides quantitative or qualitative definitions for some of the deficiencies.

Local deficiencies refer to the deficiencies in individual members. Each beam and column in a frame should have not only the required strength, but also sufficient deformation capacity. A slab should be properly connected to the frames. The deficiencies can be detected by checking the capacities and studying the detailing of reinforcement in the slabs, beams, columns and joints as shown in the as-built drawings. The capacities can be compared with the force demands calculated from a building analysis. The detailing of reinforcement in the drawings can be compared with the requirements of the code IS 13920: 1993.

When a building has deficiencies that have not been accounted for in the structural design, it needs retrofit. The different retrofit strategies can be grouped under global retrofit strategies or local retrofit strategies. A global retrofit strategy targets the performance of the building as a whole under lateral loads. Addition of new walls, frames or braces, reduction of any irregularity or mass of the building are grouped under global retrofit strategies. These improve the lateral strength and stiffness of the building. If a building is significantly deficient in resisting seismic forces, a global retrofit strategy should first be investigated. A local retrofit strategy targets the strength and ductility of a member, without significantly affecting the lateral strength or stiffness of the building. Repeated use of a local retrofit strategy can improve the ductility in the base shear versus roof displacement behaviour of the building. The local retrofit strategies include concrete jacketing or attaching steel plates or wrapping polymer sheets to a column or a beam. Each type of these strategies is described in this chapter. Finally some general remarks on retrofit of RC buildings are provided.

9.2 BUILDING DEFICIENCIES

This section lists some common deficiencies observed in multi-storeyed RC buildings in India. The observations were made from the buildings which were damaged or collapsed due to the earthquake at Bhuj, Gujarat, in 2001 (Murty et al., 2002). A few other observations were made in a project on evaluation and retrofit of RC buildings.

9.2.1 Global Deficiencies

Global deficiencies are the attributes that degrade the lateral load resisting mechanism of a building subjected to an earthquake. Some of the deficiencies are caused by 'irregularities' in the structural configuration (IS 1893: 2002). The irregularities are broadly classified as plan irregularities and vertical irregularities. The plan irregularities can be detected by observation and simple calculations based on the plan of a building. Similarly, the vertical irregularities can be detected from the elevation of a building.

The irregularities result in an irregular load path, leading to structural damage and failure. The effects of irregularities may not be detected by the conventional equivalent static analysis. The irregularities are discussed in Chapter 2 in detail. In the present chapter, the instances of the irregularities in existing buildings are highlighted.

<u>Plan Irregularities</u>

1. Torsional irregularity: This is due to a plan configuration that leads to twisting of the building and increased forces especially in the columns located at the corners of the building. Torsional irregularity is caused by plan asymmetry and/or eccentricity between the centre of mass of the floors and the centre of rigidity of the frames. It is commonly observed in buildings with overhead water tanks, roof-top swimming pools and heavy auxiliary equipments.
2. Re-entrant corners: A re-entrant corner in the floor plan refers to the corner which points inwards. Re-entrant corners cause stress concentration in the slabs. Figure 8.3 shows the plan of an existing residential building with several re-entrant corners.
3. Diaphragm discontinuity: A rigid floor slab acts as a horizontal diaphragm that mobilises the frames and shear walls to resist the lateral load, with each undergoing the same displacement at the floor level. A diaphragm discontinuity refers to a large cut-out in a floor slab which generates stress concentration in the corners of the cut-out. In many residential buildings there are multiple dwelling units on a given level. To provide windows on different faces, to maintain privacy of each unit and to provide lift- and stair-wells, often large cut-outs are provided in the floor slabs. This leads to diaphragm discontinuity (Figure 8.3). When the dwelling units on two sides of a building are staggered in elevation, the diaphragm action is reduced.

4. Out-of-plane offsets: When the lateral load resisting elements are discontinued or interrupted above the foundation within their own plane, the resulting irregularity is referred to as out-of-plane offset. An example is when the columns along the perimeter of a building are discontinued at the ground storey (Figure 9.1). These columns are supported on cantilever overhang beams and are termed as floating columns. The offset in load path is from the perimeter frame in the upper storeys to the outer columns in the ground storey. This type of frame may be adequate for gravity loads, but perform poorly when subjected to earthquakes. Instances of such building frames occur where there is a limitation for moving space along the periphery of the building at the ground level.
5. Non-parallel systems: If the frames or shear walls are not laid out in mutually perpendicular directions or if the columns axes are inclined (in plan) to the orthogonal directions of the building, the lateral load resistance of the building is diminished. This deficiency may occur in buildings with non-rectangular grid plans, such as curved buildings.

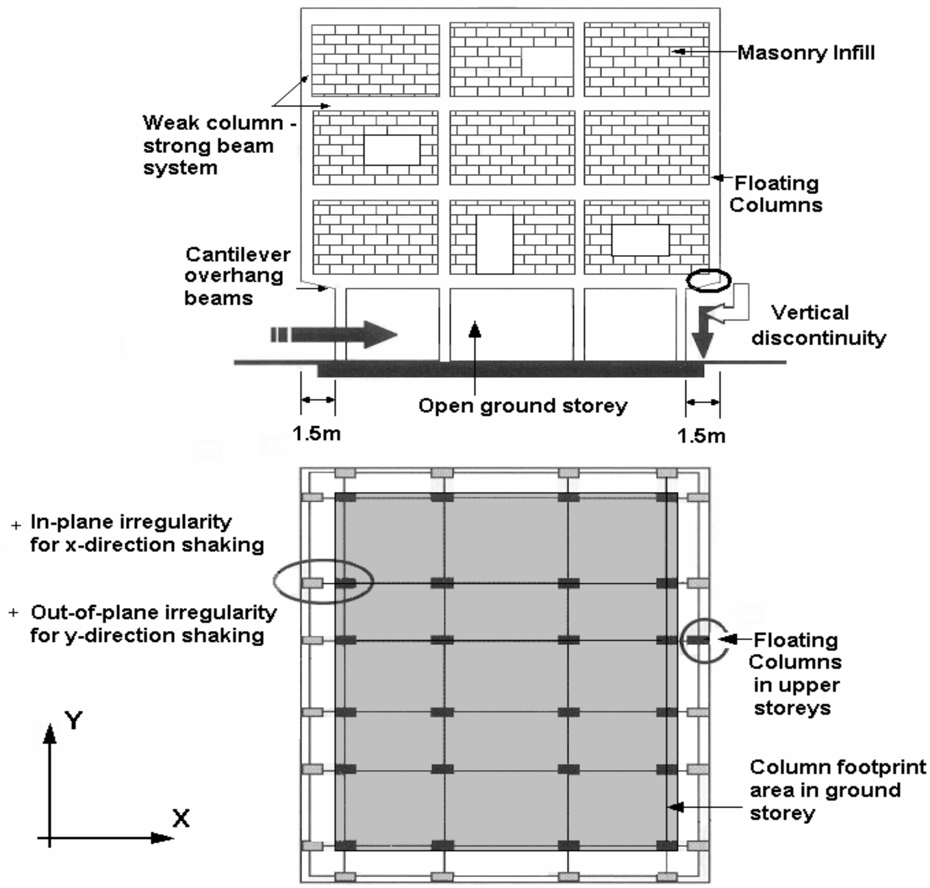


Figure 9.1 Typical building elevation and plan showing plan and vertical irregularities (Murty et al., 2002)

<u>Vertical Irregularities</u>

1. Stiffness irregularity: This arises when there is substantial reduction in lateral stiffness in any storey with respect to that in the upper storey. The storey with reduced stiffness is referred to as soft storey. The wall panels within a frame (infill walls) affect the stiffness of a storey. In the recent years, to facilitate parking of vehicles, infill walls are omitted in the ground storey (Figure 9.1). This type of *open ground storey* may lead to a soft storey. Irrespective of the number of storeys, one side of the columns is 230 mm (if not both) to flush them with the walls in the upper storeys. Such buildings are commonly referred to be on *stilts*. If the ground storey is used as shops, infill walls are not placed in the front

side to have open front or glazing. Absence of adequate plinth beams or tie beams leads to long columns and differential lateral movement of the isolated footings.

2. Mass irregularity: When there is substantial difference in mass between two storeys, it is designated as mass irregularity. Although mass irregularity is not commonly observed, it may exist in a particular floor due to heavy equipment, for example.
3. Vertical geometric irregularity: If a part of a building continues above the rest, such as a set-back tower, the forces in the members near the base of the tower tend to be high. Instances of this irregularity are observed in institutional buildings with a plaza type elevation.
4. In-plane discontinuity: When the lateral load resisting system is shifted within its plane in a certain storey, the irregularity is referred to as in-plane discontinuity. A floating column is an example of in-plane discontinuity for the frame in the elevation (Figure 9.1). A column in the upper storeys that is interrupted at the first floor and supported on a transfer beam, is another example. For a strong transfer beam, the supporting columns may be weak. Such type of beam-column joints is undesirable for seismic resistance. The failure of a column before the formation of hinges in the supported beams is disastrous. Hence, the strong-column–weak-beam concept is strongly advocated in seismic design.
5. Strength irregularity: This arises when there is substantial reduction in lateral strength in any storey with respect to that in the upper storey. The storey with reduced strength is referred to as weak storey. An open ground storey with 230 mm columns may lead to a weak storey, resulting in an undesirable sway mechanism under seismic load. The sway mechanism refers to the movement of the top storeys like a single block with large deformation of the ground storey columns. The building swings back and forth like an inverted pendulum during an earthquake. In the process the columns in the ground storey get severely damaged.

9.2.2 Local Deficiencies

Local deficiencies arise due to improper design, faulty detailing, poor construction and poor quality of materials. These lead to the failure of individual members of the building such as flexural and shear failures of beams, columns and shear walls, crushing or diagonal cracking of masonry walls and failure of beam-column joints or slab-beam or slab-column connections.

Local deficiencies can be minimised by following the ductile detailing requirements specified by IS 13920: 1993. The basic principles and guidelines are explained with the aid of Figure 9.2.

For Columns

1. The splicing of longitudinal bars should be at the central half of the column.
2. There should be sufficient transverse reinforcement in the form of closed hoops near the joints and at the location of splices of longitudinal bars. The amount of hoops near a joint should be such that the supported beams generate their flexural capacities before the column fails in shear.
3. The hoops should be continued throughout the joint and in to the footing.

For Beams

1. There should be minimum flexural capacities both for sagging and hogging throughout the length of the beam across a frame. There should be at least two longitudinal bars at the top and two bars at the bottom which are continuous throughout the length and interior joints.
2. There should be proper anchorage of the longitudinal bars at the exterior joints. A joint in which there is a beam only on one side in the plane of the frame is termed as an exterior joint.
3. There should be sufficient transverse reinforcement in the form of closed hoops near the joints and at the location of splices of longitudinal bars. The amount of hoops near a joint should be such that the beam generates its flexural capacity with yielding of the reinforcing bars (rebar) before it fails in shear. The yielding of the rebar under reversed cyclic loading increases the rotation near a joint, resulting in the formation of a *plastic hinge*. The ductile behaviour leads to the absorption and dissipation of internal energy.

The commonly observed local deficiencies in the members are summarised next.

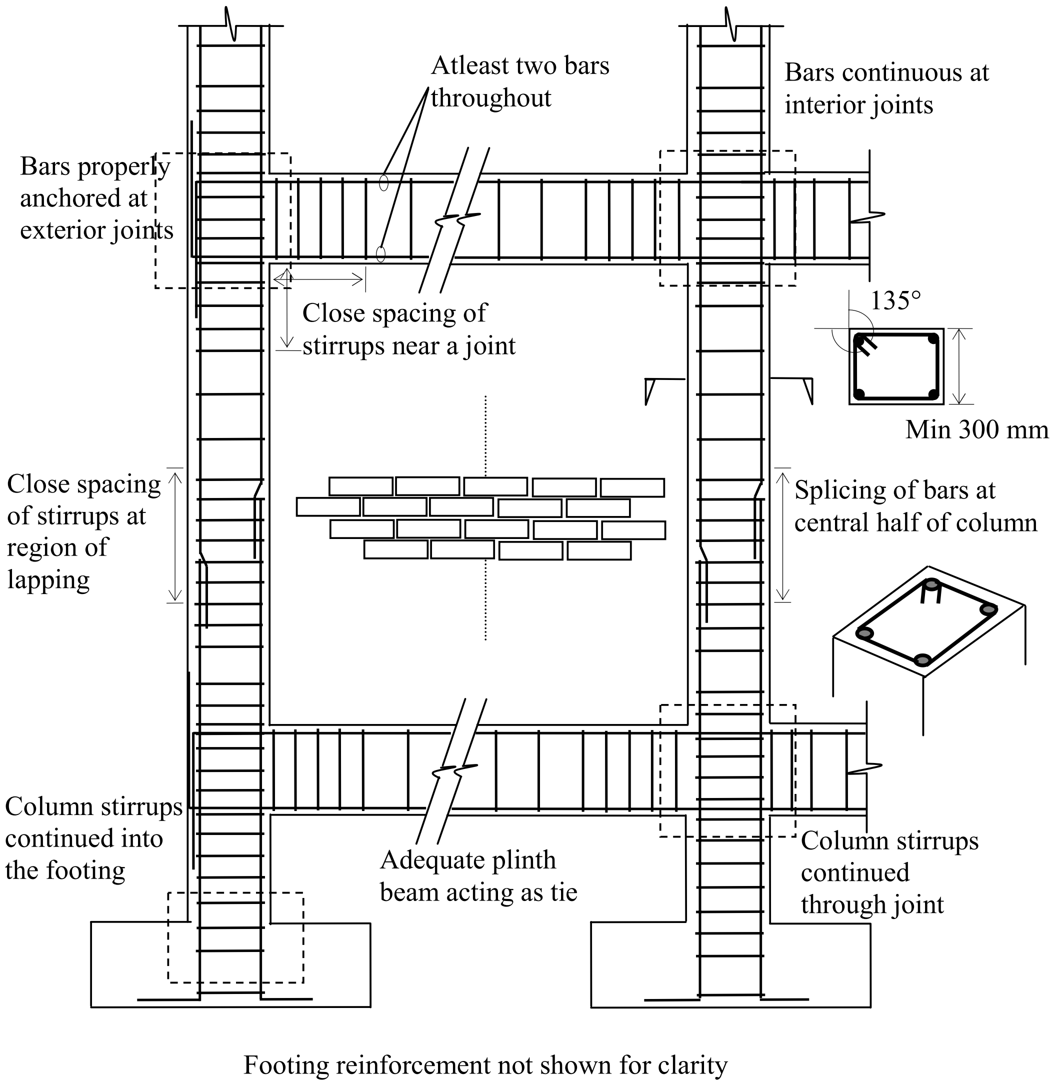


Figure 9.2 Elevation of a frame showing detailing for seismic forces

Columns

1. As mentioned before that one side of the columns in the primary lateral load resisting frames is 230 mm (if not both) to flush them with the walls in the upper storeys. The flexural and shear strengths of such a column in the lower storeys of multi-storeyed buildings may not be adequate. Columns with large aspect ratio (length to width ratio) can be inadequate under biaxial moments. Since earthquake ground motion can occur in any direction, the columns which are part of orthogonal frames should be designed for biaxial moments.
2. The ties are widely spaced. A tie gets warped (the sides are not in a plane) and the ends are not bent by 135º with adequate length inserted within the core of the section. As a result the longitudinal bars tend to buckle and the confinement of concrete is poor. This leads to failure of the column before the formation of hinges in the supported beams.
3. Faulty splicing of rebar is also detrimental to the formation of hinges. The location of splice in a column just above the floor level with inadequate splice length is inappropriate.
4. Short and stiff (*captive*) columns due to infill walls of partial height or columns next to openings attract larger shear which leads to their failure.

Beams and Beam-column Joints

1. The positive flexural strength and shear strength of beams in the primary lateral load resisting frames at the plinth level or first floor level tend to be inadequate. This is aggravated by the inadequate anchorage of the longitudinal rebar at the joints. In an exterior joint, the bars (especially the bottom bars) may not have adequate hooks. In an interior joint, the bottom bars may be discontinuous. Under moment reversal, the discontinuous bottom bars may pull-out leading to loss of flexural strength of the beam.
2. The rotation capacity of a beam near the joint (potential hinge location) may be inadequate due to lack of confining reinforcement. This may lead to a sudden shear failure before a hinge is formed.
3. Inadequate confinement of reinforcement in a joint leads to undesirable shear deformation of the joint.

The observed deficiencies are schematically shown in Figure 9.3.

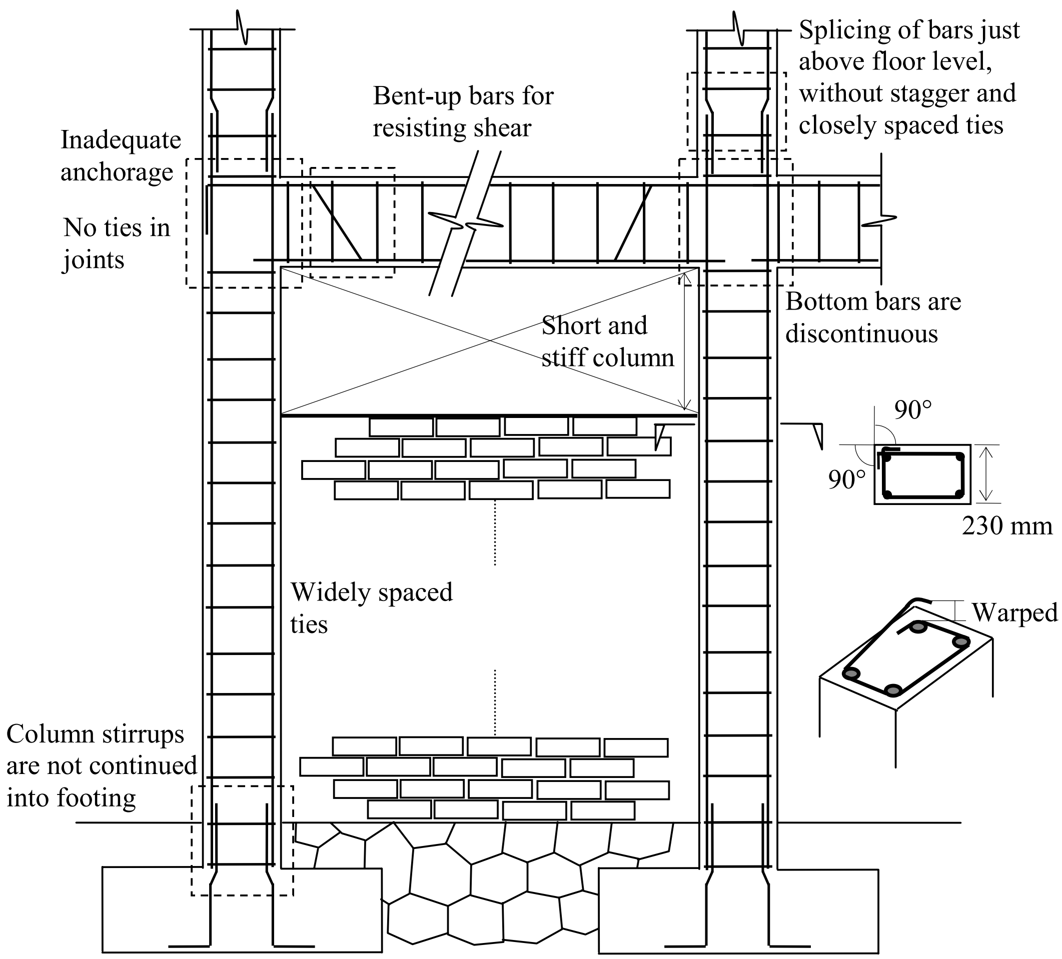


Figure 9.3 Elevation of a typical frame showing local deficiencies

Slabs and Slab-column Connections

The following deficiencies affect the diaphragm action of the floor slabs.

1. When a floor slab acts as a diaphragm, additional reinforcement is required at the edges. The reinforcement at the edges perpendicular to the direction of lateral force is termed as chord reinforcement. If the slab is not supported on edge beams, for example in a flat slab, the requirement of chord reinforcement cannot be neglected.
2. The lack of adequate shear reinforcement at the slab-column connections in a flat slab lead to failure of the slab.

Structural Walls

If a wall is not designed adequately, the contribution of the wall in the lateral load resistance of a building is not utilised. The deficiencies that affect the performance of the wall are mentioned.

1. Lack of adequate boundary members to resist the axial forces due to in-plane bending of a wall.
2. Inadequate reinforcement at the slab-wall or beam-wall connections. This reduces the integrity of the wall with the building frame.

Foundation

The isolated footings are not properly designed for the seismic forces. The plinth beams or tie beams may be absent which makes the footings vulnerable to lateral spread.

Unreinforced Masonry Walls

1. Due to lack of out-of-plane bending capacity, the falling of masonry blocks from infill walls and parapets at large height can be hazardous. The potential of the walls to resist in-plane lateral loads is not materialised if there is failure due to out-of-plane bending.

Precast Members

1. Lack of tie reinforcement in precast members can lead to dislocation of the members and collapse of the building.

9.2.3 Miscellaneous Deficiencies

Deficiencies in Analysis and Design

Inadequate tools for proper analysis lead to deficiencies in analysis and design. Some of the common deficiencies are listed.

1. A building is designed only for gravity loads. There is no analysis for seismic loads.

2. Neglecting the effect of infill walls.

Frames with brittle infills, such as unreinforced masonry, behave similar to braced frames with the infills acting as diagonal compression 'struts'. When the infill walls are neglected in the analysis of a building, a longer time period is estimated by the analysis, thus resulting in lower calculated seismic forces. This concern is addressed in Clause 7.10.3, IS 1893: 2002, which recommends a more stringent formulation for calculating the time period of masonry infilled frames. The stiffness of an infill wall influences the location of centre of rigidity of the lateral load resisting system in a storey. If infill walls are present only on one side, neglecting the infill walls in the analysis may lead to overlooking a torsional irregularity.

3. Inadequate geotechnical data

The foundations are designed without adequate information of the type of soil, bearing capacity and locations of fill and fault. In the absence of data on faults, amplification effects due to interference of the earthquake waves are neglected.

4. Neglecting the P-Δ effect

For a building on soft soil especially, the loss of stiffness during an earthquake leads to an increase in the displacement response. The increased displacement leads to higher eccentricity of the vertical loads, causing additional moment in the columns (P-Δ effect). If the P-Δ effect is not accounted for in the analysis, the member design forces are likely to be under-estimated.

Deficient Construction Practices

1. Volume batching of concrete, that may lead to increased water content.
2. Additional dosage of water to increase slump, resulting in higher water-to-cement ratio.
3. Inadequate compaction and curing of concrete, leading to honey-combed and weaker concrete.
4. Inadequate cover leading to rebar corrosion.

5. Poor quality control in the constituents of concrete.
6. Use of re-rolled steel, having inadequate strength and ductility.

The deficiencies in construction practices lead to deviations from design assumptions and calculations.

Lack of Integral Action due to Poor Design

The building performance is degraded due to the absence of tying of the lateral load resisting members. For example, the beams are not framed into the elevator core walls (Fig. 9.4) and spandrel beams between perimeter columns are missing. If the beams are eccentric to column lines with large offsets, then unaccounted torsion is introduced in the columns and beam-column joints.

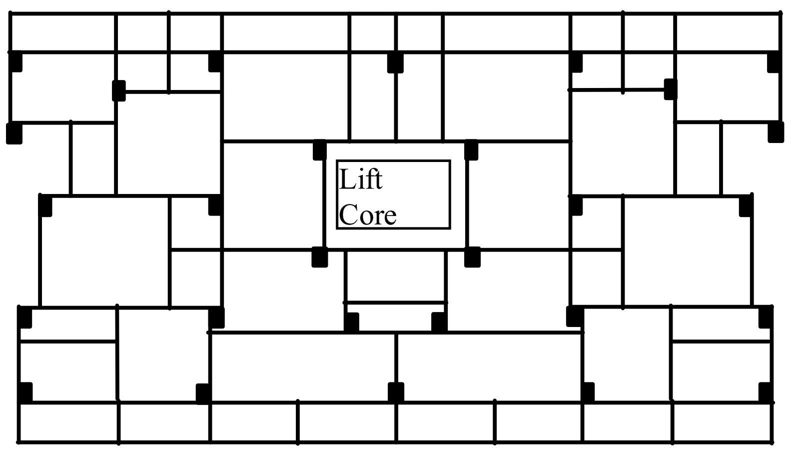


Figure 9.4 A building plan showing lack of integral action

Deficient Staircase

If the stair-case slab is simply supported longitudinally, a collapse of the slab closes the escape route for the residents.

Pounding of Buildings

Another poor design concept arises from inadequate gap between adjacent buildings or seismic joints between the segments of a building. When the provided gap is not adequate, the buildings or segments of a building will collide or "pound" against each other as they respond to the earthquake excitation in out-of-phase motion. This phenomenon transfers forces to the buildings which are unaccounted for in the design.

<u>Lack of Maintenance and Unaccounted Addition or Alteration of the Buildings</u>

Lack of maintenance and unscrupulous addition or alteration without any seismic analysis and design check, can lead to high seismic vulnerability or even collapse, possibly under gravity loads.

9.3 RETROFIT STRATEGIES

> A retrofit strategy is a technical option for improving the strength and other attributes of resistance of a building or a member to seismic forces. The retrofit strategies can be classified under global and local strategies. A global retrofit strategy targets the performance of the entire building under lateral loads. A local retrofit strategy targets the seismic resistance of a member, without significantly affecting the overall resistance of the building.

The grouping of the retrofit strategies into local and global are generally not be mutually exclusive. For example, when a local retrofit strategy is used repeatedly it affects the global seismic resistance of the building. It may be necessary to combine both local and global retrofit strategies under a feasible and economical retrofit scheme.

9.3.1 Global Retrofit Strategies

When a building is found to be severely deficient for the design seismic forces, the first step in seismic retrofit is to strengthen and stiffen the structure by providing additional lateral load resisting elements. Additions of infill walls, shear walls or braces are grouped under global retrofit strategies. A reduction of an irregularity or of the mass of a building can also be considered to be global retrofit strategies. The analysis of a building with a trial retrofit strategy should incorporate the modelling of the additional stiffening members.

<u>Addition of Infill Walls</u>

The lateral stiffness of a storey increases with infill walls. Addition of infill walls in the ground storey is a viable option to retrofit buildings with open ground storeys (Fig. 9.5). Due to the 'strut action' of the infilled walls, the flexural and shear forces and the ductility demand on the ground storey columns are substantially reduced. Of course, infill walls do not increase the ductility of the overall response of the building.

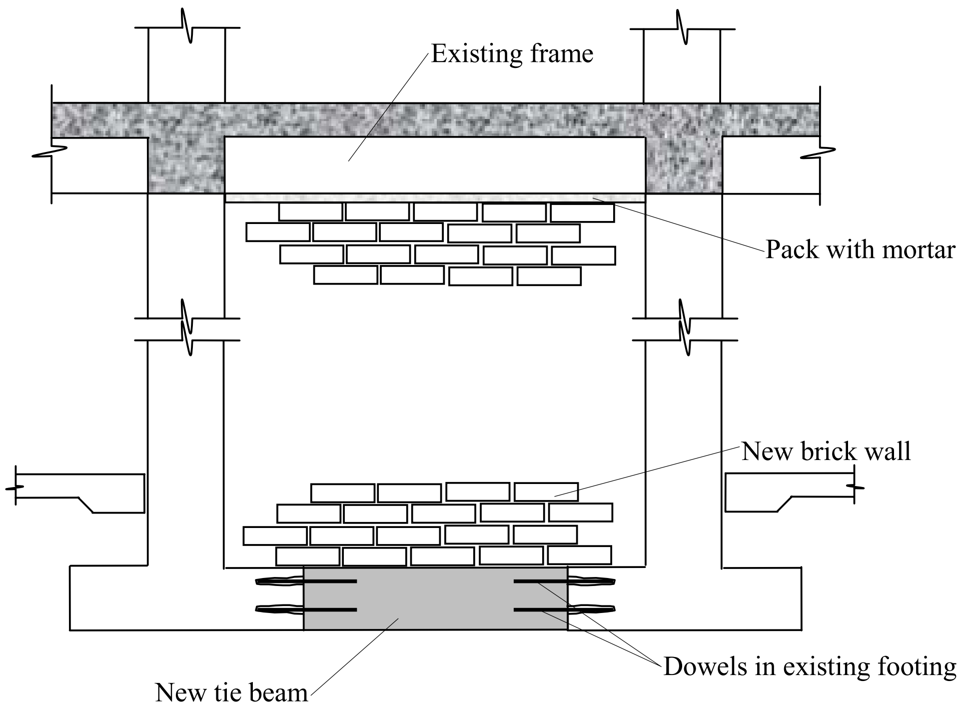


Figure 9.5 Addition of a masonry infill wall

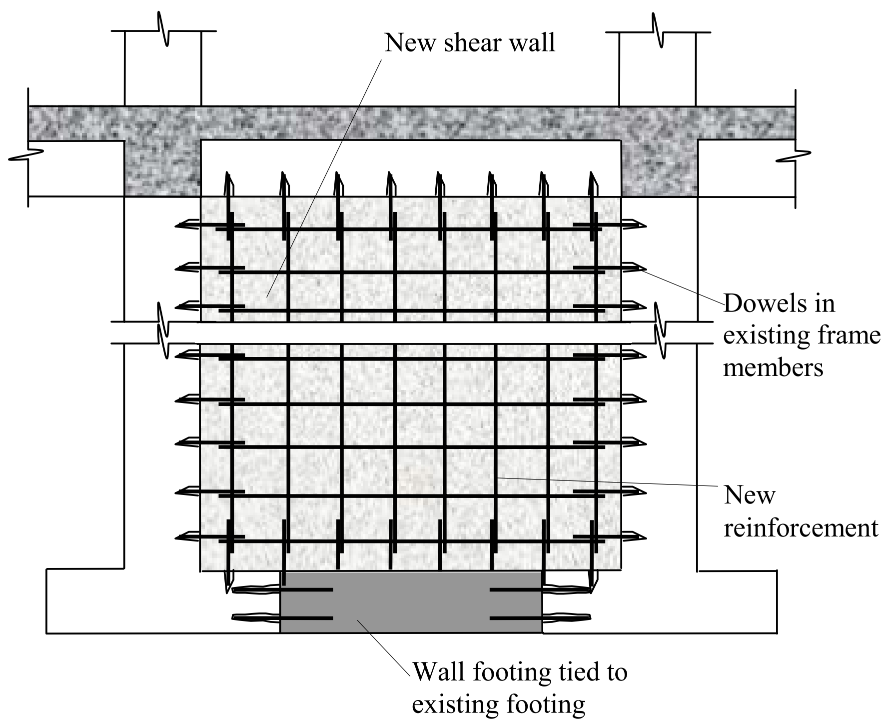
<u>Addition of Shear Walls or Wing Walls or Buttress Walls</u>

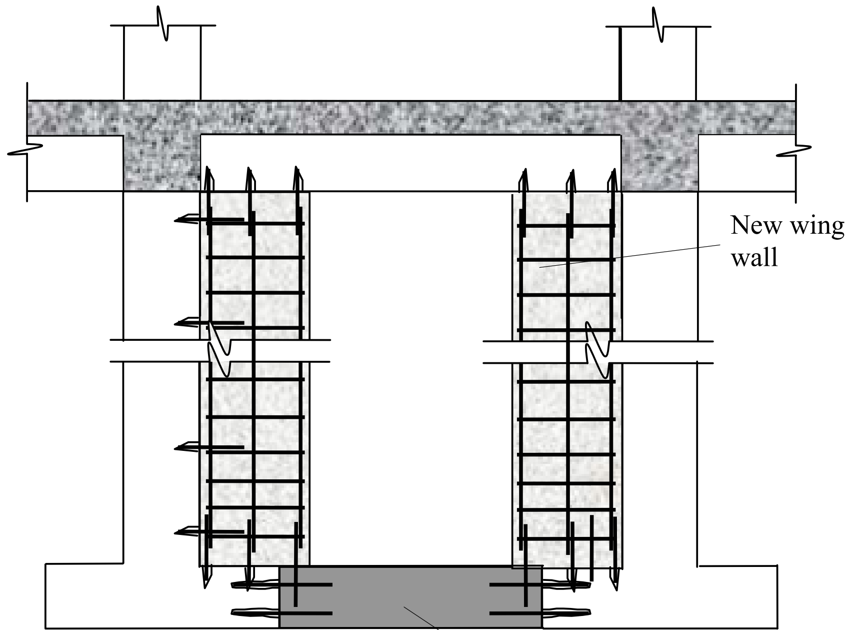
Shear walls, wing walls or buttress are added to increase lateral strength and stiffness of a building. The shear walls are effective in buildings with flat slabs or flat plates. Usually the shear walls are placed within bounding columns (Fig. 9.6a), whereas wing walls are placed adjacent to columns (Fig. 9.6b). The buttress walls are placed on the exterior sides of an existing frame (Fig. 9.6c). The critical design issues involved in the addition of such a wall are as follows.

a) To integrate the wall to the building for transferring of lateral forces.

b) To design the foundations for the new wall.

The disadvantage is that if only one or two walls are introduced, the increase in lateral resistance is concentrated near the new walls. Hence, it is preferred to have distributed and symmetrically placed walls. The shift of the centre of rigidity should not be detrimental. For a buttress wall, the new foundation should be adequate to resist the overturning moment due to the lateral seismic forces without rocking or uplift. The stabilising moment is only due to the self-weight of the wall. This can be low as compared to the overturning moment.




New wing wall
New tie beam

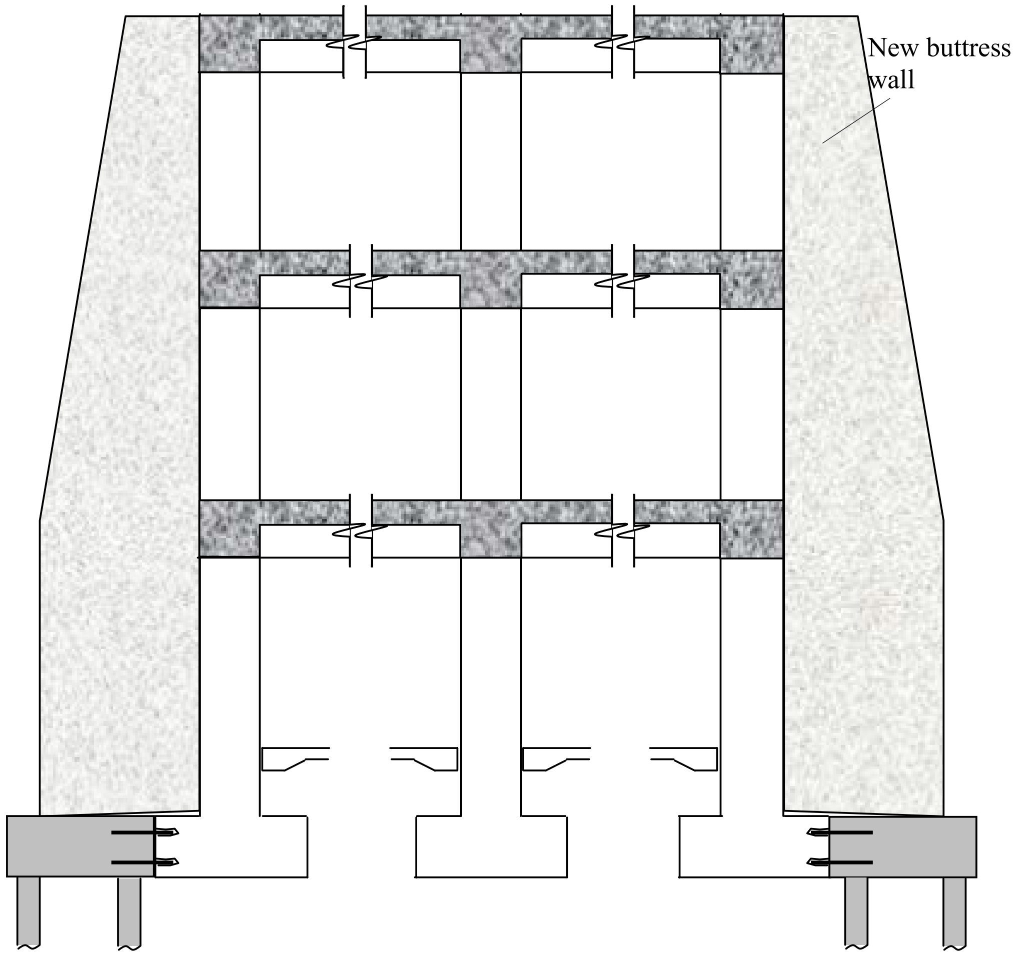


Figure 9.6 a) Addition of a shear wall (Courtesy: FEMA 172), b) Addition of wing walls, c) Addition of buttress walls

Addition of Steel Braces

A steel bracing system can be inserted in a frame to provide lateral stiffness, strength, ductility, hysteretic energy dissipation, or any combination of these (Fig. 9.7). The braces are effective for relatively more flexible frames, such as those without infill walls. The braces can be added at the exterior frames with least disruption of the building use. For an open ground storey, the braces can be placed in appropriate bays while maintaining the functional use. Passive energy dissipation devices may be incorporated in the braces to enhance the seismic absorption. The types of bracing, analysis and design of braces are covered in Chapter 10. The connection between the braces and the existing frames is an important consideration of this strategy. One technique of installing braces is to provide a steel frame within the designated RC frame. The steel frame is attached to the RC frame by installing headed anchors in the later (FIB Bulletin 24, 2003). Else, the braces can be connected directly to the RC frame. Here, since the braces are connected to the frames at the beam-column joints, the forces resisted by the braces are transferred to the joints in the form of axial forces, both in compression and tension. While the addition of compressive forces may be tolerated, the resulting tensile forces are of concern.

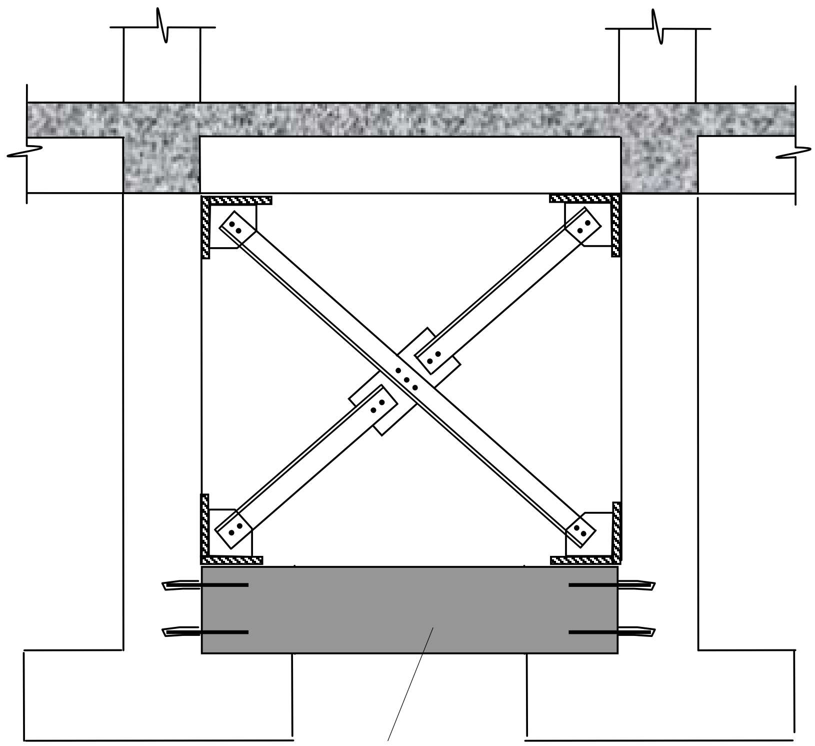


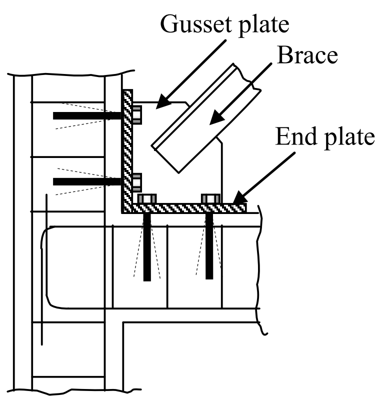
Figure 9.7 Addition of steel braces

A few types of connections are shown in Figure 9.8.

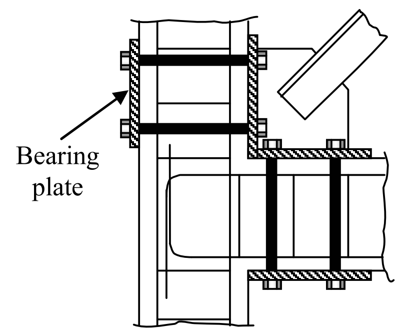
Type I (Figure 9.8a): The force in brace is transferred to the frame through the gusset plate, end plates and anchor inserts.

Type II (Figure 9.8b): This type of connection is similar to Type I, except for the method of anchoring the end plates at the joint. An end plate is connected using through bolts which are anchored at the opposite face to a bearing plate. In this type, the widths of end plate and bearing plate are equal to or less than the width of beam or column.

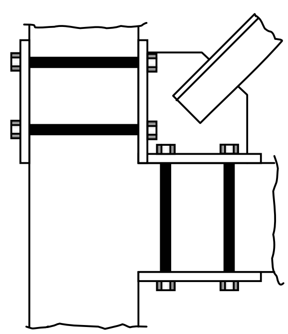
Type III (Figure 9.8c): This type of connection is similar to Type II, except for the location of bolts. In this type, the end plate and bearing plate project beyond the width of the beam and column. Since the bolts are out side the members, drilling of holes through the members is avoided.



(a) Connection by anchor bolts



(b) Connection by through bolts



(c) Connection by bolts placed outside the members

Figure 9.8 Types of connection of braces to an RC frame

Addition of Frames

A new frame can be introduced to increase the lateral strength and stiffness of a building. Similar to a new wall, integrating a new frame to the building and providing foundations are critical design issues.

Reduction of Irregularities

The plan and vertical irregularities are common causes of undesirable performance of a building under an earthquake. Reduction of the irregularities may be sufficient to reduce force and deformation demands in the members to acceptable levels. Addition of infill walls, shear walls or braces can alleviate the deficiency of soft and/or weak storeys. Discontinuous components of the lateral load resisting system such as floating columns can be extended up to the foundation. In this case, the supporting cantilever beams have to be checked for the sagging moment. The infill walls of partial height can be extended to reduce the vulnerability of short and stiff (*captive*) columns.

Torsional irregularities can be corrected by the addition of frames or shear walls to balance the distribution of stiffness and mass. An eccentric mass due to an overhead water tank can be relocated. Seismic joints can be introduced to transform a single irregular building into multiple regular structures. Although partial demolition can have impact on the appearance and utility of the building, it can be an effective measure to reduce irregularity.

Reduction of Mass

A reduction in mass of the building results in reduction of the lateral forces. Hence, this option can be considered instead of structural strengthening. The mass can be reduced through demolition of unaccounted additional storeys, replacement of heavy cladding or removal of heavy storage and equipment loads, or change in the use of the building.

Energy Dissipation Devices and Base Isolation

Most energy dissipation devices not only supplement damping, but also provide additional lateral stiffness to a building. Base isolation devices reduce the structural force entering a building by elongating the time period of the structure and thus decreasing the base shear force. These devices are described in Chapter 14.

A comparative evaluation of the different global retrofit strategies is provided in Table 9.1

Table 9.1 Comparative evaluation of the global retrofit strategies

Retrofit strategy	Merits	Demerits	Comments
Addition of infill walls	• Increases lateral stiffness of a storey • Can support vertical load if adjacent column fails	• May have premature failure due to crushing of corners or dislodging • Does not increase ductility • Increases weight	• Low cost • Low disruption • Easy to implement
Addition of shear walls, wing walls and buttress walls	• Increases lateral strength and stiffness of the building substantially • May increase ductility	• May increase design base shear • Increase in lateral resistance is concentrated near the walls • Needs adequate foundation	• Needs integration of the walls to the building • High disruption based on location, involves drilling of holes in the existing members
Addition of braces	• Increases lateral strength and stiffness of a storey substantially • Increases ductility	• Connection of braces to an existing frame can be difficult	• Passive energy dissipation devices can be incorporated to increase damping / stiffness or both
Addition of frames	• Increases lateral strength and stiffness of the building • May increase ductility	• Needs adequate foundation	• Needs integration of the frames to the building

9.3.2 Local Retrofit Strategies

Local retrofit strategies pertain to retrofitting of columns, beams, joints, slabs, walls and foundations. The local retrofit strategies are categorised according to the retrofitted elements. The retrofitting of foundations is separately covered in Chapter 12. The analysis of a building with a trial local retrofit strategy should incorporate the modelling of the retrofitted elements.

The local retrofit strategies fall under three different types: concrete jacketing, steel jacketing (or use of steel plates) and fibre-reinforced polymer (FRP) sheet wrapping. In this chapter the first two types are discussed. Chapter 13 is exclusively devoted for the retrofit using FRP. Out of the first two types, each one has its merits and demerits. Table 9.2 at the end of the section provides a comparative evaluation of the strategies with general statements. It may be noted that deviations are expected in individual retrofit projects.

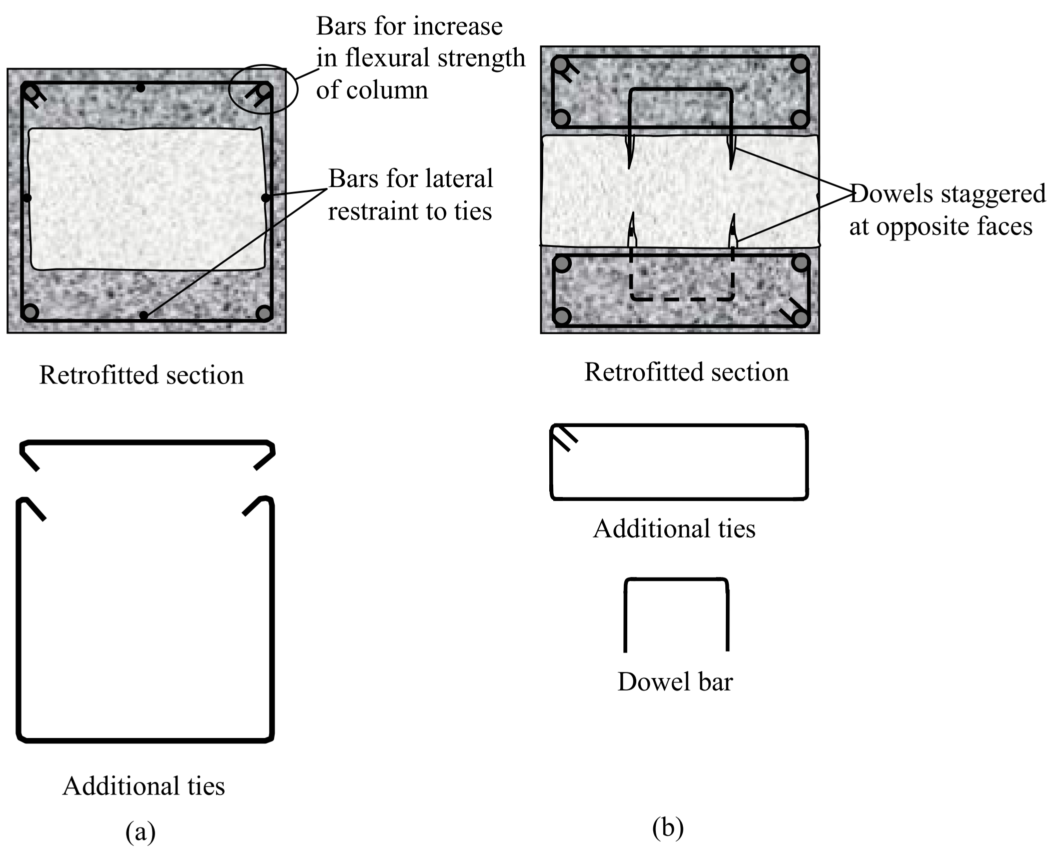
Column Retrofitting

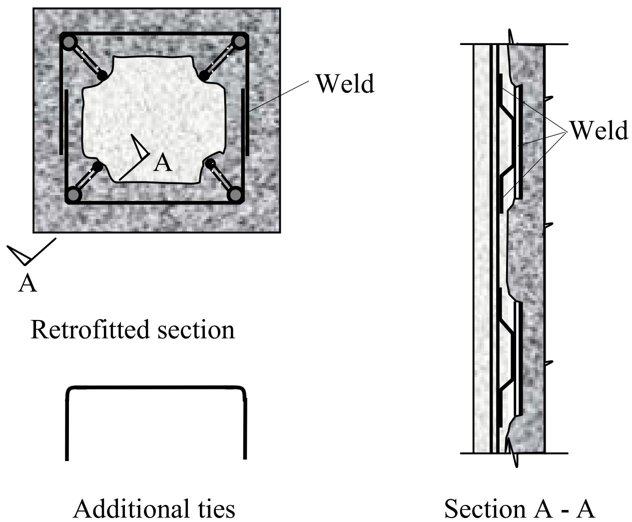
The retrofitting of deficient columns is essential to avoid collapse of a storey. Hence, it is more important than the retrofitting of beams. The columns are retrofitted to increase their flexural and shear strengths, to increase the deformation capacity near the beam-column joints and to strengthen the regions of faulty splicing of longitudinal bars. The columns in an open ground storey or next to openings should be prioritised for retrofitting. The retrofitting strategy is based on the "strong column weak beam" principle of seismic design. During retrofitting, it is preferred to relieve the columns of the existing gravity loads as much as possible, by propping the supported beams. The individual retrofit strategies are described next.

1. Concrete Jacketing

Concrete jacketing involves addition of a layer of concrete, longitudinal bars and closely spaced ties. The jacket increases both the flexural strength and shear strength of the column. Increase in ductility has been observed (Rodriguez and Park, 1994). If the thickness of the jacket is small there is no appreciable increase in stiffness. Circular jackets of ferro-cement have been found to be effective in enhancing the ductility. The disadvantage of concrete jacketing is the increase in the size of the column. The placement of ties at the beam-column joints is difficult, if not impossible (Stoppenhagen et al., 1995). Drilling holes in the existing beams damages the concrete, especially if the concrete is of poor quality. Although there are disadvantages, the use of concrete jacket is relatively cheap. It is important to note that with the increase in flexural capacity, the shear demand (based on flexural capacity) also increases. The additional ties are provided to meet the shear demand.

There are several techniques of providing a concrete jacket (Figure 9.9). A technique is selected based on the dimensions and required increase in the strength of the existing column, available space of placing the longitudinal bars. To increase the flexural strength, the additional longitudinal bars need to be anchored to the foundation and should be continuous through the floor slab. Usually the required bars are placed at the corners so as to avoid intercepting the beams which are framing in to the column. In addition, longitudinal bars may be placed along the sides of the column which are not continuous through the floor (Figure 9.9a). These bars provide lateral restraint to the new ties. A tie cannot be made of a single bar due to the obstruction in placement. It can be constructed of two bars properly anchored to the new longitudinal bars. It is preferred to have 135° hooks with adequate extension at the ends of the bars. An alternative arrangement to the drilling of holes near the joint, by confining the joint with steel angles has been tried by researchers. For an interior joint if there are beams framing in all the four sides, then the confining action of the beams is dependable.




Weld
A
A
Retrofitted section
Additional ties
Weld
Section A - A

(c)

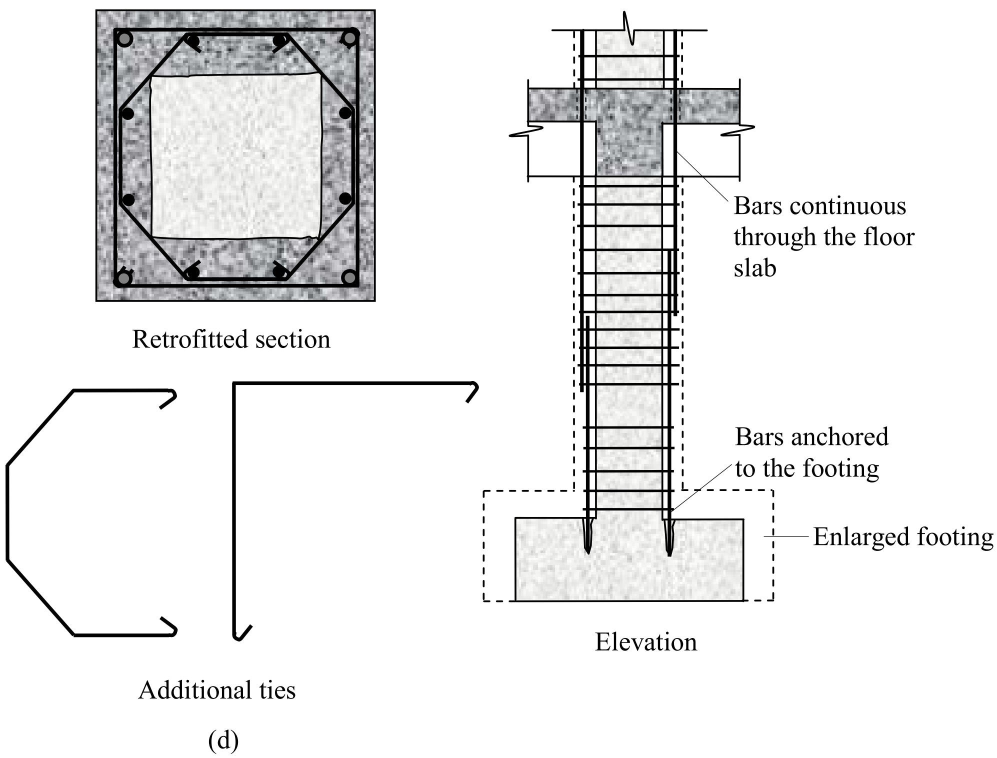


Figure 9.9 Techniques for concrete jacketing of columns

Since the thickness of the jacket is small, casting micro-concrete or the use of shotcrete are preferred to conventional concrete. To ensure the composite action of the existing and the new concrete, the options for preparing the surface of the existing concrete are hacking by chisel, roughening by wire buff or using bonding chemicals. Slant shear tests have shown that the preparation of the surface is adequate to develop the bond between the existing and new concrete. Inserting dowels in the existing column at a spacing of 300 to 500 mm enhances the composite action (Figure 9.9b). The dowels are attached by epoxy pumped in to the drilled holes. The length of insertion depends on the type of epoxy and the strength of the existing concrete. The later can be estimated through non-destructive or intrusive tests as described in Chapter 4. Based on the information of the epoxy from the manufacturer's catalogue and the evaluated concrete strength, the length of insertion is determined. Of course drilling holes damages the existing column. Moreover, it has been observed that the presence of dowels increases the disintegration of the concrete jacket during the formation of a plastic hinge under dynamic loads. If the jacket is all around the existing column, then the dowels can be avoided. The shrinkage of the new concrete generates friction with the existing concrete. If the jacket is only partially around the existing column, the existing bars can be exposed at locations. The new bars can be welded to the existing bars using Z- or U- shaped bent bars (Figure 9.9c). Of course it has been observed that welding of two different grades of steel increases corrosion.

The minimum specifications for the concrete jacket are as follows (Draft Code).

a) The strengths of the new materials must be equal to or greater than those of the existing column. The compressive strength of concrete in the jacket should be at least 5 MPa greater than that of the existing concrete.

b) For columns where extra longitudinal bars are not required for additional flexural capacity, a minimum of 12 mm diameter bars in the four corners and ties of 8 mm diameter should be provided.

c) The minimum thickness of the jacket should be 100 mm.

d) The minimum diameter of the ties should be 8 mm and should not be less than ⅓ of the diameter of the longitudinal bars. The angle of bent of the end of the ties should be 135º.

e) The centre-to-centre spacing of the ties should not exceed 200 mm. Preferably, the spacing should not exceed the thickness of the jacket. Close to the beam-column joints, for a height of ¼ the clear height of the column, the spacing should not exceed 100 mm.

A simplified analysis for the flexural strength of a retrofitted column can be done by the traditional method of interaction curves (SP 16: 1980, "Design Aids for Reinforced Concrete to IS 456: 1978, published by the Bureau of Indian Standards) assuming a composite section. The publication of the International Federation for Structural Concrete (FIB Bulletin 24, 2003) recommends such an analysis. Even if the grade of concrete in the jacket is higher, it can be considered to be same as that of the existing section. Such an analysis assumes that there is perfect bond between the new and old concrete. The yield moment and the ultimate flexural capacity can be conservatively limited to 90% of the calculated values. The increase in shear capacity can be calculated based on the amount of additional ties. For the requirement of confinement, only the additional ties are to be considered.

2. Steel Jacketing

Steel jacketing refers to encasing the column with steel plates and filling the gap with non-shrink grout. The jacket is effective to remedy inadequate shear strength and provide passive confinement to the column. Lateral confining pressure is induced in the concrete as it expands laterally. Since the plates cannot be anchored to the foundation and made continuous through the floor slab, steel jacketing is not used for enhancement of flexural strength. Also, the steel jacket is not designed to carry any axial load. If the shear capacity needs to be enhanced, the jacket is provided throughout the height of the column. A gap of about 25 to 50 mm is provided at the ends of the jacket so that the jacket does not carry any axial load. For enhancing the confinement of concrete and deformation capacity in the potential plastic hinge regions, the jacket is provided at the top and bottom of the column. Of course there is no significant increase in the stiffness of a jacketed column. Steel jacketing is also used to strengthen the region of faulty splicing of longitudinal bars. As a temporary measure after an earthquake, a steel jacket can be placed before an engineered scheme is implemented.

Circular jackets are more effective than rectangular jackets. A jacket is made up of two pieces of semi-circular steel plates which are welded at the site. A circular jacket can be considered equivalent to continuous hoop reinforcement. Of course circular jackets may not be suitable for columns in a building since the columns are mostly rectangular in cross-section. In a rectangular jacket, steel plates are welded to corner angles. Anchor bolts or through bolts may increase confinement but it involves drilling into the existing concrete. A simpler form of strengthening is to weld batten plates to the corner angles. This form is referred to as steel profile jacketing as opposed to the encasement provided by continuous plates. Figure 9.10 shows the different techniques of providing a steel jacket. The steel plates need protection against corrosion and fire.

The shear strength of the jacket (V_j) can be calculated by considering the jacket to act as a series of independent square ties of thickness and spacing t_{sj}, where t_{sj} is the thickness of the plates (Aboutaha et al., 1999). For rectangular columns,

$$V_j = A_{sj} \frac{f_{sj}\, d_{sj}}{s_{sj}} \tag{9.1}$$

Here, A_{sj} is the total area of the assumed square tie, $A_{sj} = 2t_{sj}^2$, f_{sj} is the allowable stress of the jacket, d_{sj} is the depth of the jacket (can be equated to the transverse depth of the column) and s_{sj} is the spacing between the square ties, $s_{sj} = t_{sj}$. The allowable stress of the jacket can be assumed to be half of its 'yield' stress. The required thickness t_{sj} can be calculated from the required value of V_j.

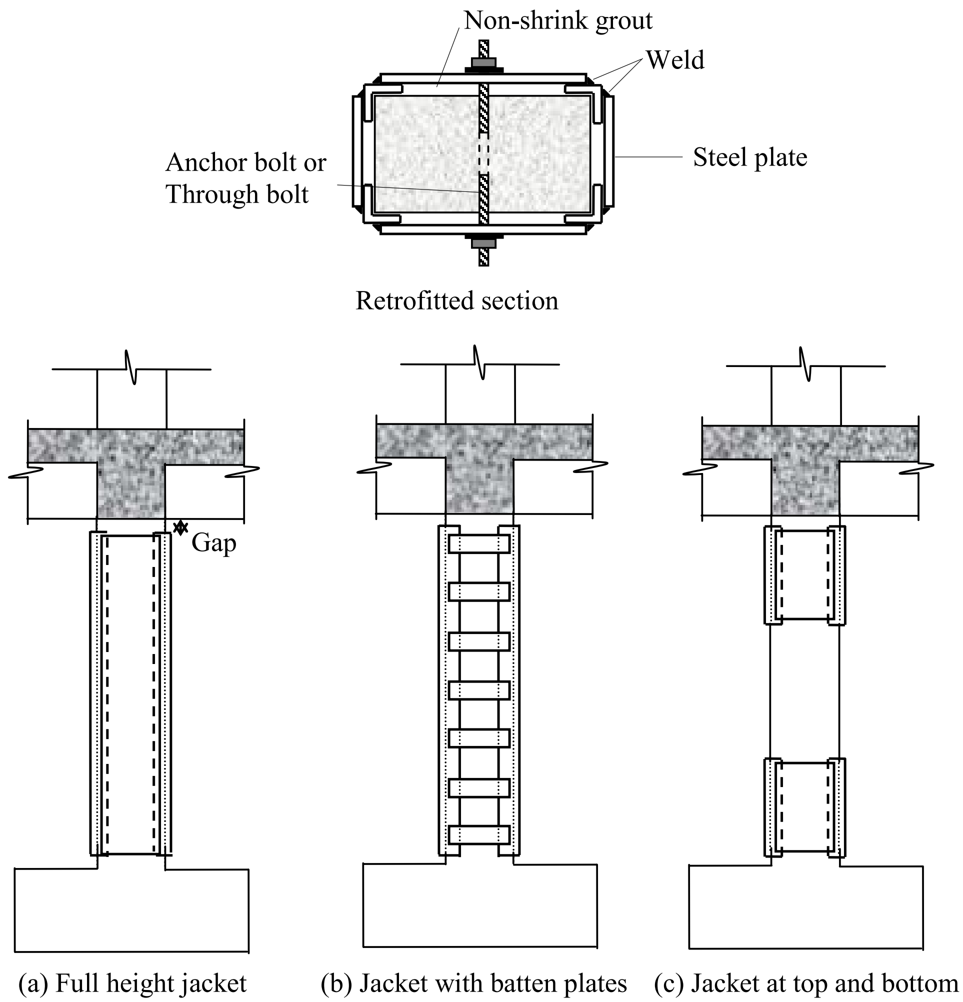


Figure 9.10 Techniques for steel jacketing of columns

3. Fibre Reinforced Polymer Sheet Wrapping

Fibre-reinforced polymer (FRP) has desirable physical properties like high tensile strength to weight ratio and corrosion resistance. FRP sheets are thin, light and flexible enough to be inserted behind pipes and other service ducts, thus facilitating installation. In retrofitting a column with FRP sheets, there is increase in ductility due to confinement without noticeable increase in the size. The main drawbacks of FRP are the high cost, brittle behaviour and inadequate fire resistance. The details of retrofitting using FRP sheets are covered in Chapter 13.

Beam Retrofitting

The beams are retrofitted to increase their positive flexural strength, shear strength and the deformation capacity near the beam-column joints. The lack of adequate bottom bars and their anchorage at the joints needs to be addressed. Usually the negative flexural capacity is not enhanced since the retrofitting should not make the beams stronger than the supporting columns. Of course the strengthening of beams may involve the retrofitting of the supporting columns. The individual retrofit strategies are described next.

1. Concrete Jacketing

Concrete is added to increase the flexural and shear strengths of a beam. The strengthening involves the placement of longitudinal bars and closely spaced stirrups. There are disadvantages in this traditional retrofit strategy. First, the drilling of holes in the existing concrete can weaken the section if the width is small and the concrete is not of good quality. Second, the new concrete requires proper bonding to the existing concrete. In the soffit of a beam, the bleed water from the new concrete creates a weak cement paste at the interface. If the new concrete is not placed all around, restrained shrinkage at the interface induces tensile stress in the new concrete. Third, addition of concrete increases the size and weight of the beam.

Instead of conventional concrete, fibre reinforced concrete can be used for retrofit. In addition to strength, this leads to the increase of energy absorption capacity. It is important to note that with the increase in flexural capacity, the shear demand (based on flexural capacity) also increases. Additional stirrups need to be provided to meet the shear demand.

There are a few options for concrete jacketing (Figure 9.11). The difficulty lies in anchoring the new bars. A technique is selected based on the deficiency and the available space for placing the bars. The techniques given in IS 13935: 1993 involves drilling holes in the existing beam. But drilling holes for the stirrups at closing spacing damages the beam, especially if the concrete is of poor quality. The additional longitudinal bars should be continuous through the beam-column joints. The bars are placed at the corners so as to avoid intercepting the transverse beams. Additional longitudinal bars may be placed at the sides for checking temperature and shrinkage cracks. These bars need not be continuous through the joints.

If the beam supports a masonry wall, then closed stirrups may not be possible. In such a situation the ends of U-stirrups may be threaded and anchored by nuts at the top surface of the slab (Figure 9.11a). If there is no wall above the beam or the beam supports a removable partition wall, a pair of U-stirrups can be welded to form a closed stirrup (Figure 9.11b). In the technique shown in Figure 9.11c, the cover concrete is removed and the new stirrups are welded to the existing stirrups. The technique shown in Figure 9.11d may be adequate for strengthening for gravity loads. But unless the slab is thick, the stirrups will not be properly anchored to sustain the dynamic seismic forces.

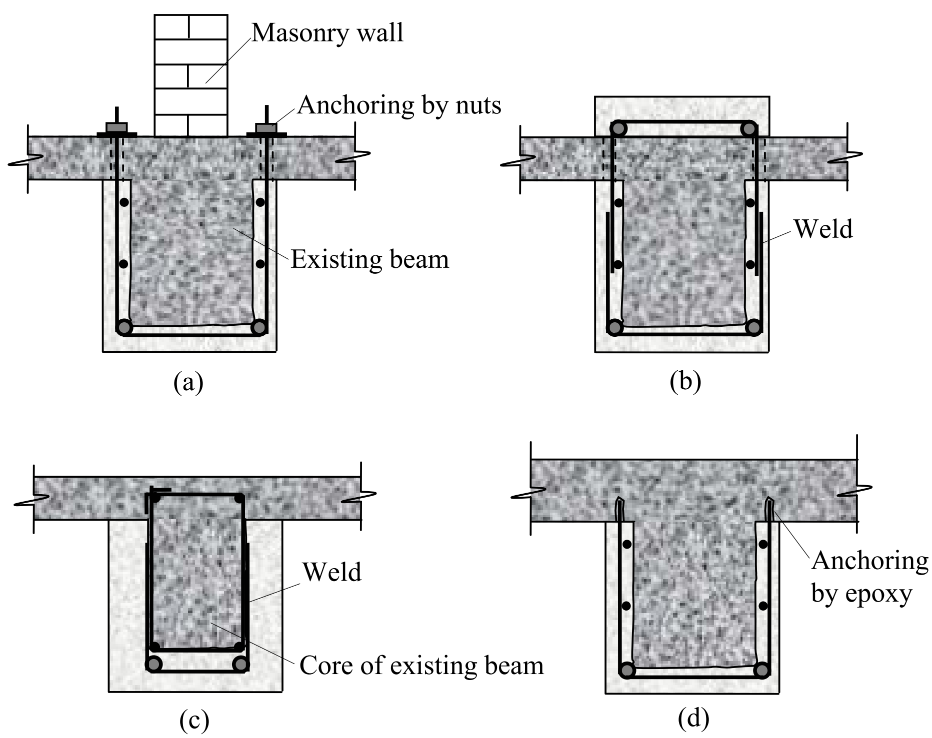


Figure 9.11 Techniques for concrete jacketing of beams

Similar to concrete jacketing of columns, the use of micro-concrete or shotcrete is preferred to conventional concrete. The surface of the existing concrete is to be prepared to ensure the composite action of the existing and the new concrete.

A simplified analysis for the flexural strength of a retrofitted section can be performed by the traditional method of beam analysis. Even if the grade of concrete in the jacket is higher, it can be considered to be same as that of the existing section. Such an analysis assumes a perfect bond between the new and existing concrete. For a rigorous analysis considering different grades of concrete, a layered approach is required.

2. Bonding Steel Plates

The technique of bonding mild steel plates to beams is used to improve their flexural and shear strengths. The addition of steel plate is rapid to apply, does not reduce the storey clear height significantly and can be applied while the building is in use. The plates are attached to the tension face of a beam to increase the flexural strength, whereas they are attached to the side face of a beam to increase the shear strength.

The plates can be attached by adhesives or bolts. The plates attached by adhesives are prone to premature debonding and hence, the beam tends to have a brittle failure. A beam with plates attached by bolts tends to have a ductile failure. But the use of bolts involves drilling in the existing concrete, which may weaken the section if the width is less or if the concrete is not of good quality. Any exposed steel plate is prone to corrosion and fire. Hence, adequate protection is required for beams retrofitted with steel plates.

The analysis for flexural strength of beams with plates bonded to the tension face is based on satisfying the equilibrium and compatibility equations and the constitutive relationships. It models the adhesive failure due to the stress concentration at the location of plate cut-off. The essential features of the model are as follows.

1. The steel plate is assumed to act integrally with the concrete beam and conventional beam theory is used to determine the flexural capacity.
2. The normal and shear stresses at the interface of concrete and the plate at the location of plate cut-off are calculated to check the failure of the adhesive.

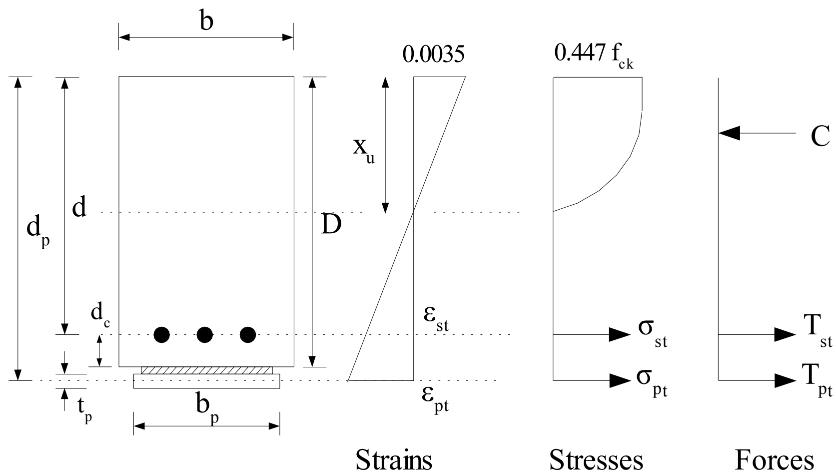


Figure 9.12 Strain and stress diagrams and internal forces of a tension-face plated beam

The strain and stress diagrams and internal forces at the ultimate limit state for a tension-face plated beam of rectangular cross-section are shown in Figure 9.12.

The depth of neutral axis (x_u) can be found out from the equilibrium equation as follows.

$$x_u = \frac{f_{st} A_{st} + f_{pt} b_p t_p}{0.36 f_{ck} b} \qquad \textbf{(9.2)}$$

The ultimate moment capacity (M_{uR}) of the plated section is given by the following expression.

$$M_{uR} = 0.36 f_{ck} b x_u \left(d - 0.416 x_u\right) + f_{pt} b_p d_p \left(d_c + t/2\right) \qquad \textbf{(9.3)}$$

Here,

b, D, d - width, overall depth and effective depth of the original section

b_p, t_p, d_p - width, thickness and effective depth of the steel plate

f_{ck} - characteristic cube strength of concrete (MPa)

f_{st}, f_{pt} - stresses in internal rebar and external plate, respectively, corresponding to the respective strains. These can be calculated from the compatibility equations and the constitutive relationships for steel of the rebar and plate. For an under-reinforced section, $f_{st} = f_y/1.15 = 0.87\,f_y$, where f_y, is the yield stress of the rebar. If the plate thickness is such that it is found to yield before the concrete crushes then $f_{st} = f_{yp}/1.1 = 0.9\,f_{yp}$, where f_{yp} is the yield stress of the plate. The material safety factor for rolled steel plates is 1.1 as per IS 800 (Draft), "Indian Standard Code of Practice for General Construction in Steel".

Since retrofitting is required mostly near the beam-column joints, the equations for a rectangular section are applicable. Of course a similar set of equations can be derived for a flanged section when the span region of a beam requires strengthening. The width of the plate (b_p) is limited to the width of the beam (b). The thickness of the plate (t_p) is limited such that the section does not become over-reinforced. The adhesive failure is checked by the following equation.

$$\tau_0 + \sigma_0 \tan 28^\circ \le c_{all} \qquad \textbf{(9.4)}$$

Here, τ_0 and σ_0 are the shear and normal stresses, respectively, at the interface of concrete and the plate at the location of plate cut-off. The allowable coefficient of cohesion for the adhesive is denoted as c_{all}. The expressions of τ_0 and σ_0 are based on the shear force at the location of plate cut-off, elastic modulus of the plate and the elastic and shear moduli of the adhesive (Ziraba et al., 1994).

The beams strengthened by plates bonded to the side face are subjected to the following modes of failure: fracture of bolts, buckling of plates and splitting of concrete. Barnes et al. (2001) derived expressions for the enhancement of shear capacity of such beams.

3. FRP Wrapping

Like steel plates, FRP laminates are attached to beams to increase their flexural and shear strengths. The details of retrofitting using FRP sheets are covered in Chapter 13.

Beam-Column Joint Retrofitting

The retrofitting of a beam-column joint aims to increase its shear capacity and effective confinement. Since access to a joint is not readily available, retrofitting a joint is difficult. The strengthening is carried out along with that for the adjacent columns and beams. The retrofitting should aim at the following improvements.

a) To prevent the pullout of any discontinuous bottom bars in the beams.

b) To reduce the deterioration of the joint under cyclic loading.

The methods of retrofit that have been investigated are as follows.

1. Concrete Jacketing

A joint can be strengthened by placing ties through drilled holes in the adjacent beams (Stoppenhagen et al., 1995). To avoid drilling holes in the beams, a steel cage can be fabricated around the additional vertical bars in the column to provide confinement (Alcocer and Jirsa, 1993). A simpler option is a concrete fillet at the joint to shift the potential hinge region of the beam away from the column face.

2. Steel Jacketing

If space is available, steel jacketing can be used to enhance the performance of joints (Ghobarah et al., 1997). A simpler option is to attach plates in the form of brackets at the soffits of the beams.

3. FRP Wrapping

In the use of FRP sheets to strengthen the joints, the considerations are number of layers, orientation and anchorage of the FRP sheets and the preparation of surface of the existing concrete.

<u>Wall Retrofitting</u>

A concrete shear wall can be retrofitted by adding new concrete with adequate boundary members (Seth, 2002). The new concrete can be added by shotcrete. For the composite action, dowels need to be provided between the existing and new concrete (Figure 9.13). The foundation of the wall is to be sufficiently strengthened to resist the overturning moment without rocking or uplift.

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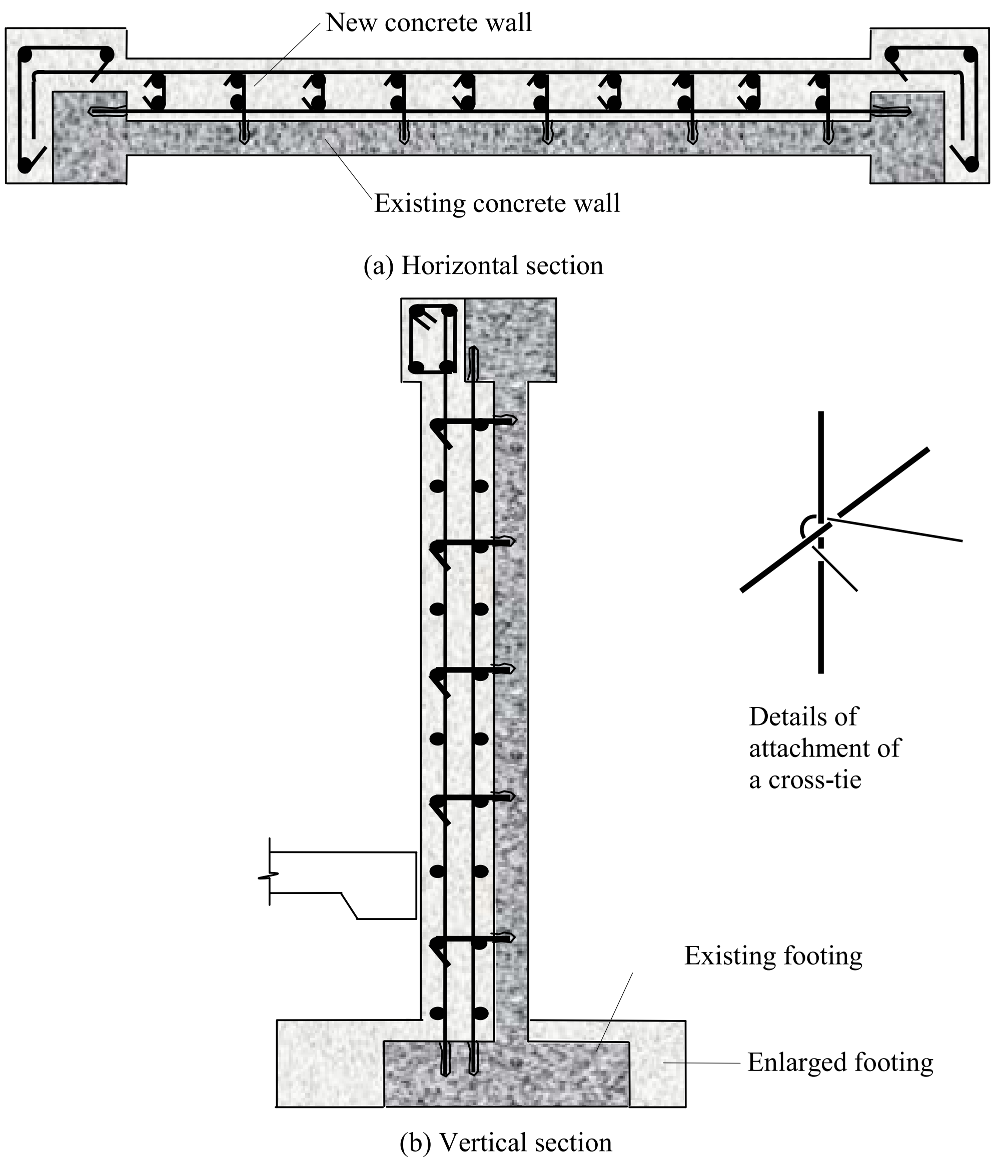


Figure 9.13 Strengthening a wall using concrete

A cross-tie connecting the layers of reinforcement on the two faces of the new wall should be hooked as shown in Figure 9.13. It should pass through diagonally opposite quadrants formed by the vertical and horizontal bars

For the masonry infill walls, whose failure can cause injury, additional concrete with wire mesh or FRP sheets can be used to strengthen for out-of-plane bending. Steel braces fitted to the RC frame have been used to check the out-of-plane bending of walls. Steel strips bolted to the walls have been investigated.

Table 9.2 Comparative evaluation of the local retrofit strategies

Retrofit strategy	Merits	Demerits	Comments
Concrete jacketing	• Increases flexural and shear strengths and ductility of the member • Easy to analyse • Compatible with original substrate	• Size of member increases • Anchoring of bars for flexural strength; involves drilling of holes in the existing concrete • Needs preparation of the surface of existing member	• Low cost • High disruption • Experience of traditional RC construction is adequate
Steel jacketing of columns	• Increases shear strength and ductility • Minimal increase in size	• Cannot be used for increasing the flexural strength • Needs protection against corrosion and fire	• Can be used as a temporary measure after an earthquake • Cost can be high • Low disruption • Needs skilled labour
Bonding steel plates to beams	• Increases either flexural or shear strengths • Minimal increase in size	• Use of bolts involves drilling in the existing concrete • Needs protection against corrosion and fire	• More suitable for strengthening against gravity loads • Cost can be high • Low disruption • Needs skilled labour
Fibre Reinforced Polymer wrapping	• Increases ductility • May increase flexural or shear strengths • Minimal increase in size • Rapid installation	• Needs protection against fire	• Cost can be high • Low disruption • Needs skilled labour

9.4 CONCLUDING REMARKS

1) Importance of condition assessment and seismic evaluation

Before undertaking seismic retrofit, it is essential to determine the condition and diagnose the deficiencies in a building. Condition assessment helps to determine the actual condition of the building as opposed to the information available from the construction documents. Seismic evaluation helps to identify the deficiencies of the building with respect to resistance to seismic forces. Based on the condition and deficiencies, repair and retrofit strategies are selected.

Considering the cost of retrofit, it is imperative to have seismic evaluations of a building both for the existing and retrofitted conditions to justify the selected retrofit strategies. When a new member is added to an existing building under a global retrofit strategy, the load transfer and the compatibility of deformation between the new and the existing elements are crucial. The load transfer should be judged from the analyses of the building. The compatibility of deformation should be ensured by proper detailing of the connections of the existing and new members.

2) Selection of a retrofit strategy

When a building is severely deficient for the design seismic forces, it is preferred to select a global retrofit strategy to strengthen and stiffen the structure. Next, if deficiencies still exist in the members, local retrofit strategies are to be selected. Beyond this recommendation it is not prudent to prescribe a retrofit strategy as a generic application. Each retrofit strategy has merits and demerits depending upon the project. A retrofit strategy is to be selected after careful considerations of the cost and constructability. Proper design of a retrofit strategy is essential. The failure mode in a member after retrofitting should not become brittle. A global retrofit strategy that involves a shift in either of the centre of mass or centre of rigidity, should be checked for torsional irregularity. Any alteration of the load path has to be carefully detected and any overstressed member has to be identified. Additional demand on the foundation has to be accounted for.

3) Quality of construction

Retrofit aims at overcoming the deficiencies of an existing building. The quality of construction for a successful retrofit scheme cannot be overemphasized. Any sort of patch work will be a wasted effort.

9.5 REFERENCES

1. ATC 40 (1996), “Seismic Evaluation and Retrofit of Concrete Buildings: Vol. 1”, Applied Technology Council, USA.
2. FEMA 172 (1992) , “NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings”, Building Seismic Safety Council, USA.
3. FIB Bulletin 24 (2003), “Seismic Assessment and Retrofit of Reinforced Concrete Buildings”, International Federation for Structural Concrete, Switzerland.
4. Draft Code, “Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings – Guidelines”, Bureau of Indian Standards.
5. IS 1893: 2002, “Criteria for Earthquake Resistant Design of Structures”, Part 1: General Provisions and Buildings, Bureau of Indian Standards.
6. IS 13920: 1993, “Indian Standard Code of Practice for Ductile Detailing of Reinforced Concrete Structures Subjected to Seismic Forces”, Bureau of Indian Standards.
7. IS 13935: 1993, “Indian Standard for Repair and Seismic Strengthening of Buildings – Guidelines”, Bureau of Indian Standards.
8. Aboutaha, R. S., Englehart, M. D., Jirsa, J. O. and Kreger, M. E. (1999), “Rehabilitation of Shear Critical Concrete Columns by Use of Rectangular Steel Jackets”, ACI Structural Journal, American Concrete Institute, January-February, Vol. 96, No. 1, pp. 68 – 78.
9. Alcocer, S. M. and Jirsa, J. O. (1993), “Strength of Reinforced Concrete Frame Connections Rehabilitated by Jacketing”, ACI Structural Journal, American Concrete Institute, May-June, Vol. 90, No. 3, pp. 249 – 261.
10. Barnes, R. A, Baglin, P. S., Mays, G. C. and Subedi, N. K. (2001), “External Steel Plate Systems for the Shear Strengthening of RC Beams”, Engineering Structures, Elsevier Publications, Vol. 23, pp. 1162 – 1176.
11. Basu, P. C. (2002), “Seismic Upgradation of Buildings: An Overview”, The Indian Concrete Journal, The Associated Cement Companies Ltd., August, pp. 461-475.
12. Ghobarah, A., Aziz, T. S. and Biddah, A. (1997), “Rehabilitation of Reinforced Concrete Frame Connections Using Corrugated Steel Jacketing”, ACI Structural Journal, American Concrete Institute, May-June, Vol. 94, No. 3, pp. 283-294.
13. Jain, S. K. (2001), “Seismic Strengthening of Existing Reinforced Concrete Buildings”, Repair and Rehabilitation, Compilation from The Indian Concrete Journal, The Associated Cement Companies Ltd., pp. 45 – 51.

14. Murty, C. V. R., Goel, R. K. and Goyal, A. (2002), "Reinforced Concrete Structures", Earthquake Spectra, Earthquake Engineering Research Institute, USA, July, Supplement A to Vol. 18, pp. 145-185.

15. Rodriguez M and Park.R (1994), "Seismic Load Test on Reinforced Concrete Columns Strengthened by Jacketing", ACI Structural Journal, Vol. 91, No.2, April – March, pp.150 – 159.

16. Seth, A. (2002), "Seismic Retrofitting by Conventional Methods", The Indian Concrete Journal, The Associated Cement Companies Ltd., August, pp. 489-495.

17. Stoppenhagen, D. R., Jirsa, J. O. and Wyllie, Jr., L. A. (1995), "Seismic Repair and Strengthening of a Severely Damaged Concrete Frame", ACI Structural Journal, American Concrete Institute, March – April, Vol. 92, No. 2, pp. 177-187.

18. Ziraba, Y. N., Baluch, M. H., Basunbul, I. A., Sharif, A. M., Azad, A. K. and Al-Sulaimani, G. J. (1994), "Guidelines toward the Design of RC Beams with External plates", ACI Structural Journal, American Concrete Institute, Nov.–Dec., Vol. 91, No. 6, pp.639–646.

10

RETROFIT OF STEEL BUILDINGS

10.1 OVERVIEW

Structural steel as a construction material is used extensively in single storey structures for industrial, storage, shopping and exhibition buildings. Structural steel enables large column free space in such single storey buildings. Structural steel is also used in tall multi-storeyed structures for residential, office and commercial buildings, although this is not very common in Indian construction practice. This chapter provides a brief description of the different types of structural systems used in single storey and multi-storeyed steel buildings.

The common deficiencies encountered in single storey buildings are the inadequate diaphragm action of the roofs, flexural and shear deficiencies of the columns, inadequate braces and poor detailing at the joints. For multi-storeyed buildings, besides the previous deficiencies, there can be plan or vertical irregularities. The retrofit strategies include strengthening individual members and joints, providing adequate braces and reducing irregularities. The retrofit strategies are explained separately for the single storey and multi-storeyed steel buildings. Brief descriptions of non-buckling braces and plate shear walls are also presented.

10.2 INTRODUCTION

This chapter deals with buildings that derive their main structural strength to resist lateral loads from structural steel members and systems. This does not include reinforced concrete (RC) buildings wherein the steel reinforcement bars provide strength and ductility, because concrete is weak in tension and relatively brittle. In India, structural steel is extensively used in single storey structures, such as industrial buildings, shopping malls, exhibition centres and storages structures. Its use in multi-storeyed building has been very minimal, in spite of its many advantages in structures subjected to seismic loads. Only recently, a few multi-storeyed buildings such as airport terminals are being constructed using structural steel. The structural steel used may consist of hot rolled steel, cold rolled steel or sections fabricated from steel plates. For the purpose of fabrication of sections and erection of the structural systems, bolting or welding are used. The use of riveting in modern construction is rare.

The design lateral loads of conventional single storey steel buildings are often governed by wind loads and not seismic load. Hence, when properly detailed, such buildings are rarely vulnerable to failure due to earthquake loads. Statistics indicate that the number of fatalities during earthquake due to failure of all types of steel buildings is significantly less compared to the other types of buildings. This stands as a testimony to the better performance of steel structures under seismic loads, compared to structures made of other materials.

Recent experiences in different countries have exposed the vulnerability of the connections in moment resisting frame systems for multi-storeyed steel buildings. Recommendations are available for retrofit of such vulnerable connections.

The use of structural steel in the design and retrofit of buildings subjected to earthquake is often economical and efficient, because of the following reasons.

- Structural steel members have higher strength-to-weight and stiffness-to-weight ratios and hence, the buildings attract less base shear under an earthquake.
- Structural steel members and systems exhibit ductile behaviour beyond elastic limit and hence, dissipate considerable energy before failure during an earthquake. The structural steel buildings are particularly effective under performance based design.
- Better quality control exercised in the production of the material, as well as fabrication and erection of the systems, ensures properties closer to the theoretical predictions.

- Structural steel can be used to retrofit existing deficient RC and steel buildings, obtaining the desired increase in strength, stiffness and ductility without increasing the dead weight appreciably. This is covered in Chapter 9, Retrofit of Reinforced Concrete Buildings.
- Dry nature of the retrofit using steel and minimal site work make retrofit using steel less intrusive in the functional requirements of the building, during retrofit.
- It is easy to design, plan and execute retrofit using steel because of ease of attachment of steel elements to existing members. Hence, retrofit using steel is usually amenable to fast track work.

In this chapter, first the single storey building systems, their deficiencies in resisting seismic loads and suggestions for retrofit are presented. Subsequently, multi-storeyed structural systems, their deficiencies and methods of retrofit are presented. In each case, the behaviour and design of the different types of structural system are discussed briefly, before reviewing the possible deficiencies and methods of retrofit.

The following Indian and international codes of practice give recommendations for evaluation, design and detailing of steel buildings to resist earthquake loads, which would be useful references for retrofit projects.

- IS 800 (Draft), “Code of Practice for General Construction in Steel”
- FEMA 172, “NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings”
- ASCE 31-03, “Seismic Evaluation of Existing Buildings”
- ANSI/AISC 341-05, “Seismic Provisions for Structural Steel Buildings”

10.3 STRUCTURAL SYSTEMS FOR SINGLE STOREY BUILDINGS

Some typical single storey buildings made of structural steel is shown in Figure 10.1. Typically, the members of the structural systems in such buildings perform the function of transfer of gravity load alone, lateral load alone or combination of gravity load and lateral load. The different structural systems and their behaviour under lateral loads are discussed in this section.

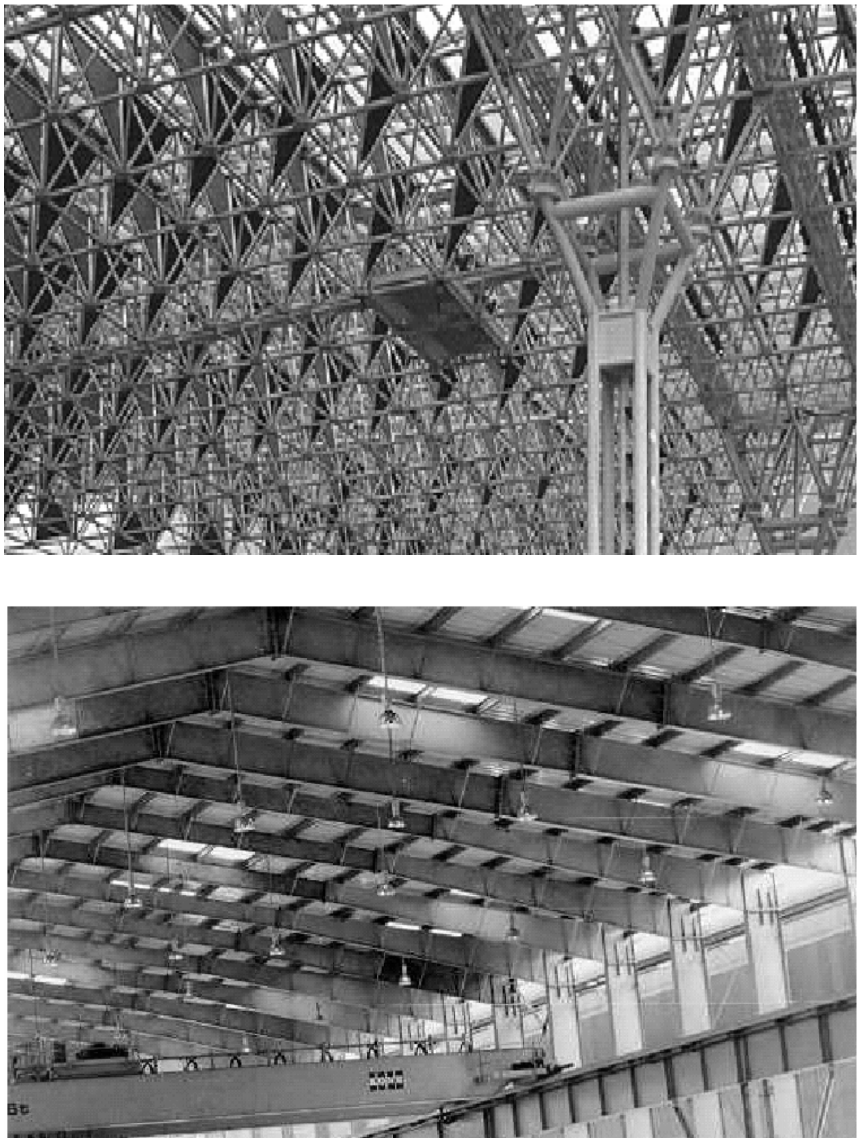


Figure 10.1 Typical single storey steel buildings

10.3.1 Gravity Load Resisting Systems

Usually purlins support the roof cladding and transfer gravity loads to the main structural members, which may be truss or rafter members of moment resisting frames. These, in turn, transfer the loads through columns to foundations. The columns may be RC or steel members. If the building is serviced by an electric overhead travelling crane, the gantry girder of the crane is also supported by such columns. In the case of buildings with light crane or no crane load, the steel columns may be members with a solid web such as I-sections. They may be laced or battened built-up columns in the case of buildings with heavy crane load or buildings with tall head room. Stepped built-up columns may be used to support the gantry and roof truss at different levels.

When the cladding material is light, frequently wind uplift (suction) force on the roof causes reversal of stresses in the gravity load system. This has to be accounted for, particularly with regard to instability of bottom chords of the trusses and bottom flanges of the rafters.

10.3.2 Lateral Load Resisting Systems

> The lateral loads on a single storey industrial building are due to wind, earthquake or crane surge. When the cladding is light and does not support any heavy equipment, usually the wind load governs the design of the lateral load resisting system. However, if the cladding consists of masonry walls and RC roof slabs, earthquake load can be the governing lateral load, especially in high seismic zones.

The lateral loads may be resisted by any one or combination of the following structural systems.

- Cantilever column system
- Moment resisting frame system
- Braced frame system
- Stressed skin system

A brief description of each of the system follows.

<u>a) Cantilever Column System</u>

Columns, acting as vertical cantilevers, fixed at the base and tied at the top by the gravity load carrying system of the roof, can resist the lateral loads (Figure 10.2). The foundations of such columns should be designed for the base moment of the cantilever, in addition to the vertical loads.

To reduce the bending moment at the base of a cantilever column, instead of providing a moment connection, frequently a knee brace is provided at the junction of the rafter and column.

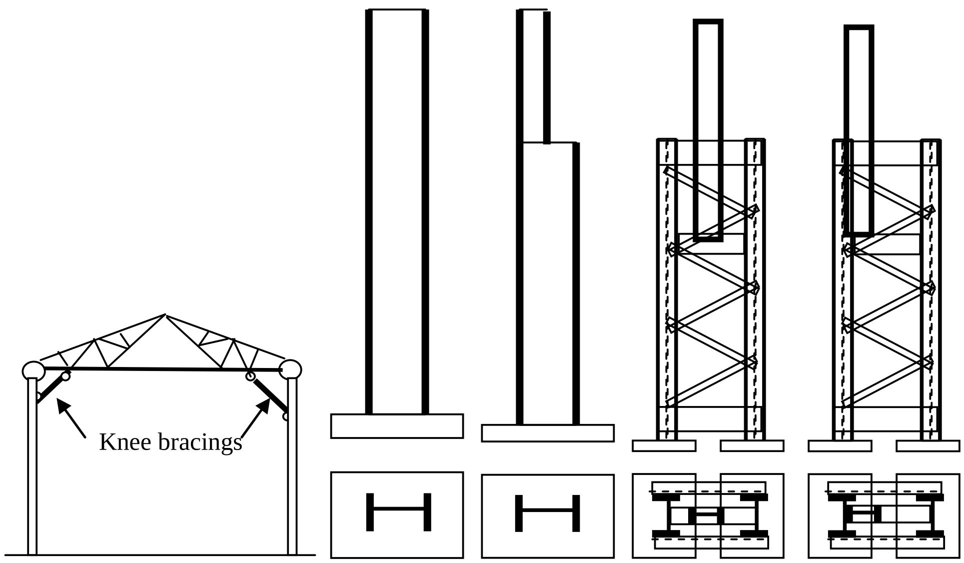


Figure 10.2 Cantilever column system

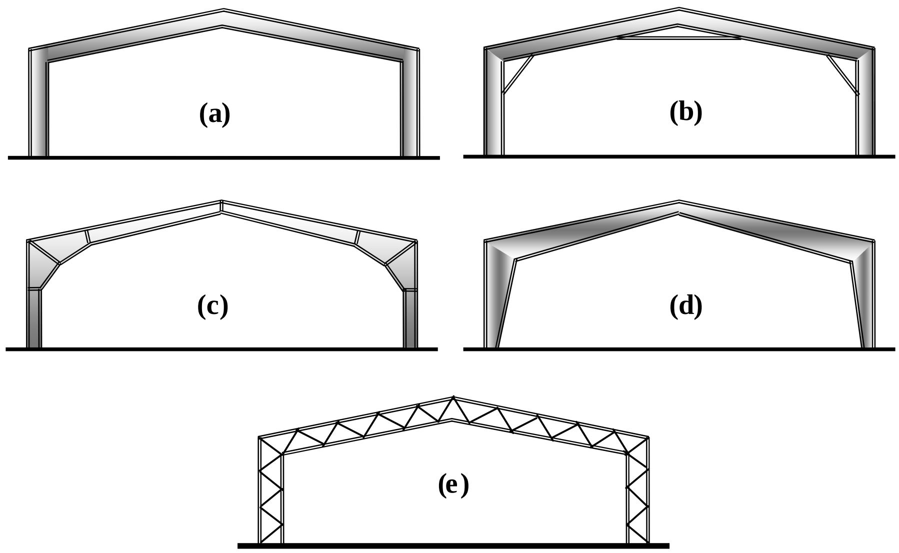


Figure 10.3 Moment resisting frame systems

b) Moment Resisting Frame System

The frames consist of columns (stanchions) and rafters that are connected at the eaves and crown for resisting moments. The members may be made of solid web I-sections in the case of smaller spans (Figure 10.3a) or may be latticed in the case of longer spans (Figure 10.3e). In order to increase the flexural strength at the regions of maximum moment, often the sections are tapered (Figures 10.3b, c, d). The frames with solid I-section exhibit good ductility, as long as the junctions between the rafters and columns are properly detailed.

c) Braced Frame System

This is the most common and probably the most economical way of achieving lateral load resistance in single storey steel buildings (Figure 10.4). The roof system consists of horizontal braces (usually X-type) either at the rafter level or tie level or both the levels of the trusses. The braces act as a horizontal truss to transfer the lateral load to the vertical bracing system in the perimeter walls. The perimeter columns in such buildings can be modelled as simply supported at the top and simply supported or fixed at the foundation. In the later case the columns behave as propped cantilevers.

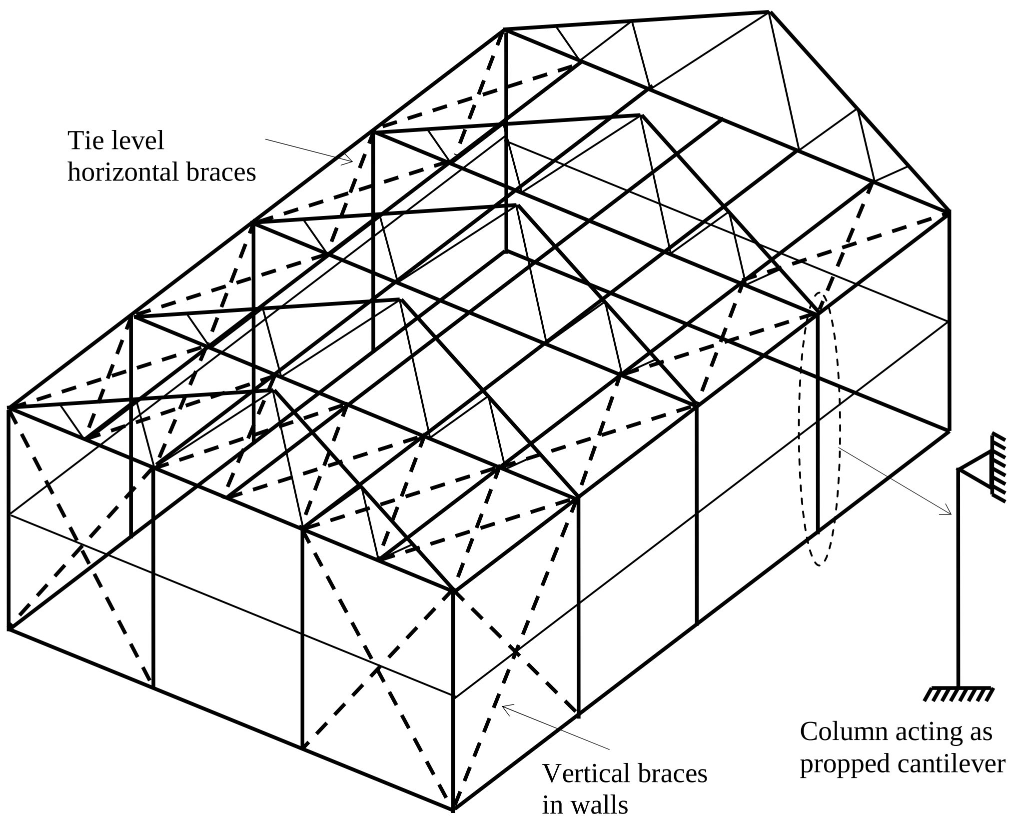


Figure 10.4 Braced frame system

d) Stressed Skin System

If the cladding of a single storey building consists of profiled metal decking properly attached to each other and the supporting members with adequate fasteners, then the cladding together with the supporting members can behave as a diaphragm to transfer in-plane shear, in addition to transferring loads normal to their plane. Such type of construction is referred to as stressed skin system (Figure 10.5). Instead of wall claddings, masonry infill between columns may be used to transfer the horizontal forces. The roof diaphragm along with the braces in the walls can be used without relying on the diaphragm action of the wall claddings. Since the diaphragm action of the claddings is usually flexible, this has to be accounted for while calculating the natural period of the structure.

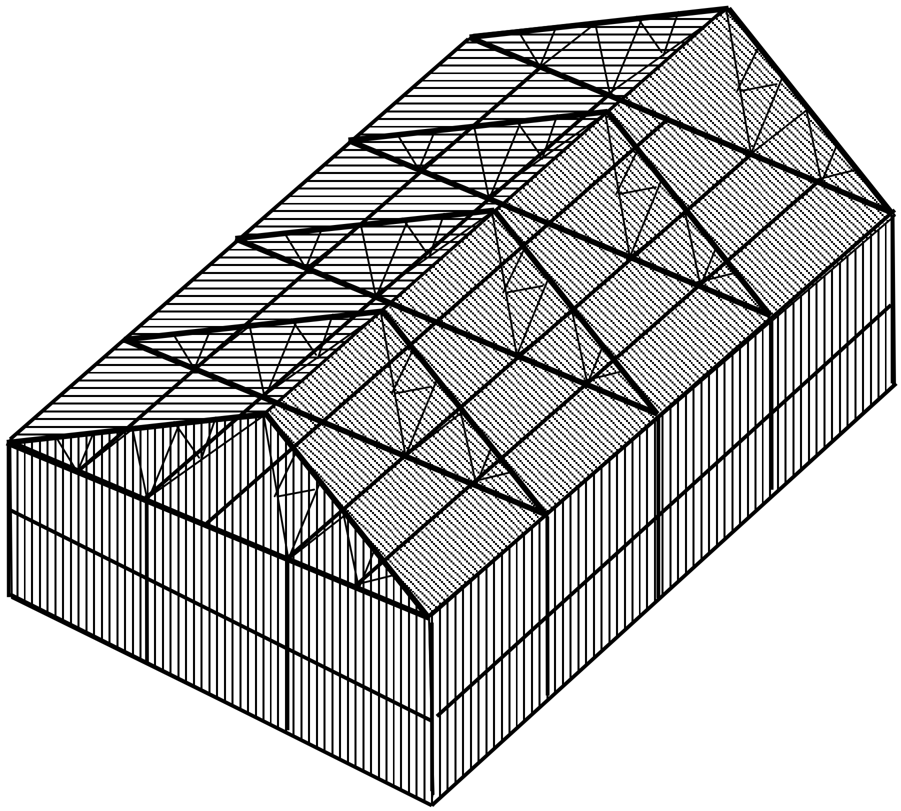


Figure 12.5 Stressed skin system

10.3.3 Recommendations for Seismic Analysis

As was mentioned already, single storey steel buildings are rarely vulnerable to earthquake forces because of their light cladding. The buildings can be evaluated for seismic loads as per IS 1893: 2002, using either the equivalent static method or the response spectrum method. These methods are explained in Chapter 8, Structural Analysis for Seismic Retrofit.

Of course, the recommendation for calculating the time period based on the height ($T = 0.085\ h^{0.75}$) tends to under-estimate the period and over-estimate the seismic forces for single storey steel buildings. This is due to the neglect of the flexibility of the roof diaphragms and the walls in these buildings, which also have large lateral dimension compared to the height. The period can be more realistically evaluated by the following methods.

- Modelling the diaphragms as flexible to obtain longer periods for the structure.
- Utilizing the energy dissipation capacity of flexible diaphragms (instead of treating the diaphragms to remain elastic) at the lateral load capacity of the vertical bracing system.

In place of tedious and time consuming models, the following formula can be used to arrive at the period of single storey steel buildings with light cladding more accurately (Lamarche et al., 2004).

$$T = 0.01L^{0.75} \tag{10.1}$$

where, L is the largest plan dimension of the building. Once the time period is more accurately ascertained, the analysis procedures can be used to calculate the design forces in the members.

10.4 DEFICIENCIES AND RETROFIT STRATEGIES FOR SINGLE STOREY BUILDINGS

The possible structural deficiencies in single storey steel buildings for resisting seismic forces and the retrofit strategies for the same are reviewed in this section. Many of these strategies may be used to retrofit against wind loads. While evaluating any steel structure for retrofit, it is necessary to evaluate the current condition. This is covered in Chapter 3, Rapid Visual Screening, Data Collection and Preliminary Evaluation and Chapter 4, Condition Assessment of Buildings.

10.4.1 Cantilever Column System

> Frequently the cantilever columns are mistakenly designed as propped cantilever columns, when there is no adequate bracing at the top to provide the lateral support. Such columns are inadequate to resist the moment due to lateral loads.

When the columns are not adequately designed to resist the moment due to lateral loads, one of the following methods can be used to retrofit the columns.

- Provide a horizontal bracing system at the tie level of the roof and attach it to the columns. The braces will provide a prop at the top of the columns and reduce the bending moment due to lateral load (Figure 10.4).
- If the flexural strength of the column is inadequate due to the low lateral buckling strength, additional lateral supports can be provided to the critical flange (usually the inner flange which is not connected to the sheeting rails / girts) by providing lateral diagonal supports emanating from the girts (Figure 10.6a).
- If the support is not adequate to prevent lateral buckling, or if the inadequacy of flexural strength is governed by material failure, then additional plates may be welded or bolted to the flanges of the columns (Figure 10.6b).

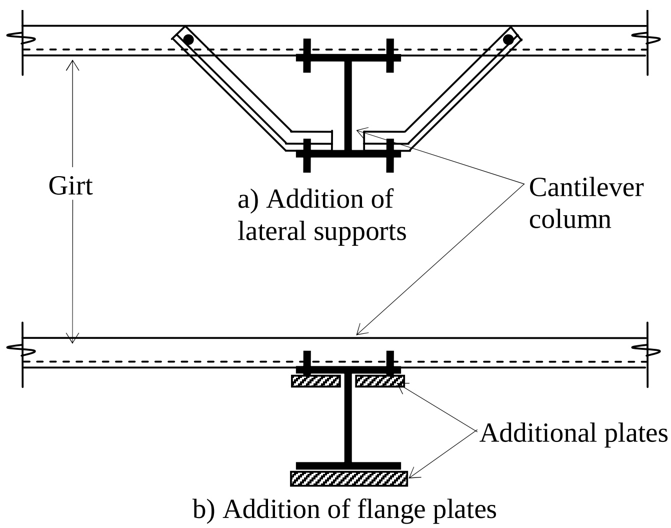


Figure 10.6 Strengthening of cantilever columns shown in plan

- The bending moment in a cantilever column can be reduced by creating a moment connection between the structural system of the roof (trusses / rafters) and the column by providing a knee brace (Figure 10.2). With this the column behaves as a member restrained partially against rotation at the top and bottom and hence, experiences reduced moment as compared to the cantilever column. In this method, in addition to designing the knee brace and its connection, the existing structural system of the roof should be checked for the additional forces due to the knee brace.

Further, the foundation/footing supporting a column may be inadequate to carry the moment. In such a case, if one of the above methods reduces the moment at the base of the column, then the foundation may be adequate. Otherwise the foundation may have to be strengthened by under-pinning, as discussed in Chapter 12, Retrofit of Foundations.

10.4.2 Moment Resisting Frame System

The moment resisting frames rely on the formation of plastic hinges at the maximum moment section to dissipate energy during an earthquake. In multi-storeyed buildings, it is desirable that such hinges form in the beams and not in the columns (strong column weak beam concept) to avoid instability. In a single storey building this restriction need not be imposed. However, the premature failure in the connections due to poor ductility of the welds should be avoided. It may be noted that the web in the column at the joint is subjected to excessive shear due to the local moment transfer from the beam (Figure 10.7a). In order to overcome the inadequacy of the web, extension plates in line with the flanges along with either a diagonal stiffener or doubler plate may be welded (Figure 10.7b).

To reduce the stresses in the welds connecting the beam to the column (to avoid their brittle fracture), a haunch can be added at these locations with ductile weld detail (Figure 10.7c). Alternatively, reducing the capacity of the beam adjacent to the junction with a transition notch will force the hinging at the reduced section in the beam, away from the joint (Figure 10.7d). This is known as the *dog bone connection* and has been effectively used in multi-storeyed buildings.

A self-centering post-tensioned (PT) moment resisting connection can be introduced to increase the energy dissipation capacity of the joint. This involves introducing PT rods in the joint at the level of neutral axis of the beam and energy dissipation devices away from the neutral axis (Figure 10.3e). Introduction of buckling restrained braces (Section 10.6.1) as the knee bracing is another way to improve the energy dissipation.

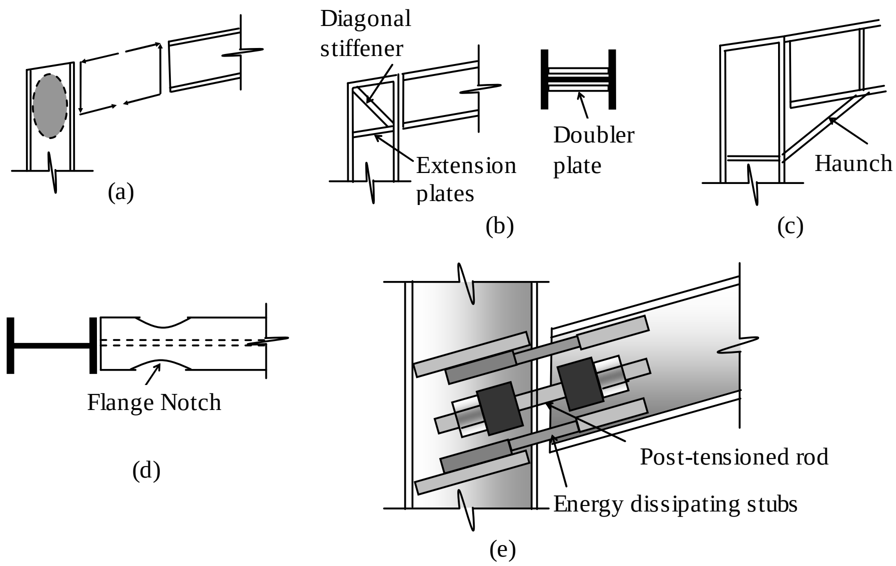


Figure 10.7 Moment resisting frame joint retrofit

10.4.3 Braced Frame System

Braced frame buildings may be vulnerable to failure during earthquake either due to the inadequate strength or ductility of the members or its connections, as explained below.

- In X-bracing, one of the diagonal members is under compression at any time. Hence, these members may undergo buckling under compression and yielding under tension alternatively. The plastic hinge formed under buckling cannot dissipate much energy and the member may fail within a few cycles at such locations. This problem may be overcome by providing buckling restrained braces (Section 10.6.1). Alternatively, very slender braces which undergo only elastic buckling in compression and do not yield in tension may be used. In this case only the member under tension can be considered to be effective at any given time. In order to decrease the forces in the inadequately designed braces, the cladding can be mobilised as a stressed skin system to resist part of the lateral load.
- Due to large rotation at the connection during buckling, the end connections of the braces may fail prematurely. Figure 10.8 shows the failure of a bracing system in an aircraft

hangar due to poor detailing of the end connection. The end connections may be modified to have adequate strength and ductility as per the design provisions of IS 800 (Draft). Very often the end connection fails before the brace.

The strength of the gusset plate in a bolted connection may be evaluated by the modified Whitmore–Thornton method (Yam and Cheng, 2002). In this method, first the tensile strength is obtained by calculating the yield strength of a segment of the plate of effective width b_w (Figure 12.9, $\theta = 30°$). Next, the compressive strength is evaluated based on the calculations of another segment of the plate defined by $\theta = 45°$. The corresponding width of the segment is b_{eff} and the effective length is 0.65 times the largest of the lengths L_1, L_2, L_3 as shown in the figure. Further, in order to ensure ductility under cyclic loading, the flexural capacity of the gusset plate for out-of-plane bending is made equal to or larger than the out-of-plane bending capacity of the brace.

Figure 10.8 Failure of the end connections of braces (circled)

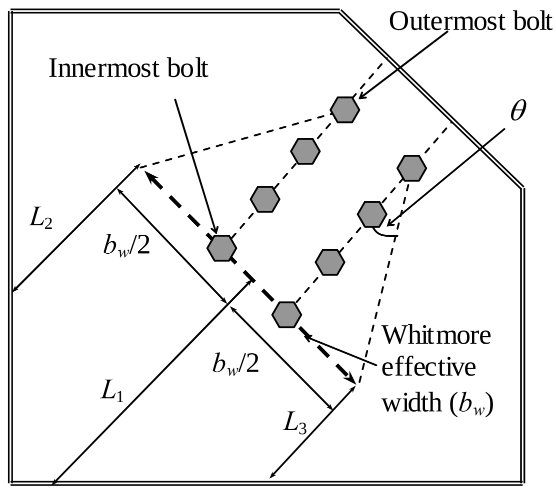


Figure 10.9 Analysis of a gusset plate

- In a V- or inverted V- bracing (Chevron type) system, the member under compression buckles beyond a certain load, whereas the member under tension continues to carry additional load. This causes an unbalanced transverse load on the horizontal beam, which is not normally designed for it. In order to overcome the problem, either the beam may be adequately strengthened for the transverse load. Alternatively, the bracing system may be converted to two-storeyed X-bracing system, by reversing the orientation of V at every alternate storey.

10.4.4 Stressed Skin System

The lateral load resistance of the stressed skin system may be inadequate due to too large spacing of the perimeter fasteners. This can be improved by providing additional fasteners at closer spacing, so that the full capacity of the cladding is mobilized. If the lateral load resistance is still inadequate, a parallel bracing system can be introduced.

10.4.5 Miscellaneous Deficiencies

A low-rise building may have excessive mass either due to heavy cladding or due to addition of storeys. Reduction in heavy cladding weight by replacing with light cladding, replacement of heavy interior partitions with light ones or with braces, removal of heavy storage and equipment loading from the roof and demolition of unaccounted storeys can considerably reduce the seismic loads.

Some buildings suffer from the eccentricities between the centres of rigidity and the centres of mass of the floors which causes torsion. Additional braces can be provided at appropriate locations to minimize the eccentricities. If the torsion is due to irregular configuration of the building, the plan shape may be sub divided into a number of regular independent shapes with adequate seismic joints in between them to prevent pounding under seismic loads.

10.5 STRUCTURAL SYSTEM FOR MULTI-STOREYED BUILDINGS

Multi-storeyed steel buildings are uncommon in India, even though they are very effective in resisting seismic loads. Therefore, the material presented in this section is based on the experiences from other countries. The different types of load resisting systems used in multi-storeyed buildings are initially reviewed.

10.5.1 Gravity Load Resisting System

The gravity load resisting system usually consists of either RC slabs or composite steel decks supported by solid web or open web steel joists. Often, shear studs are welded to the top flange of the joists and girders to create the composite action between the concrete and the steel deck. The main girders support and transfer the load from the joists to the columns.

10.5.2 Lateral Load Resisting System

Moment resisting frames, braced frames, shear walls or lift / stair core walls are the lateral load resisting systems in multi-storeyed buildings. These may be used individually or in combination to resist the lateral loads. IS 1893: 2002 in conjunction with IS 800 (Draft), can be used to design and detail these structural systems. The behaviour of each system under seismic loads, as well as recommendations for its design, is briefly discussed.

a) Moment Resisting Frames

The moment resisting frames (MRF) are fairly flexible and consequently attract lesser inertia forces due to earthquake. But due to their lower stiffness, they may exhibit greater drift under seismic loads. Frequently, masonry infill walls are used in the frames. The stiffening effect of the infill walls should be considered in the analysis and design of such frames.

> The frames should be designed based on the concept of strong columns and weak beams, so that the plastic hinges are formed in the beams before they can form in the columns. Else, the weakening of the columns may lead to premature instability and overall failure of the structure.

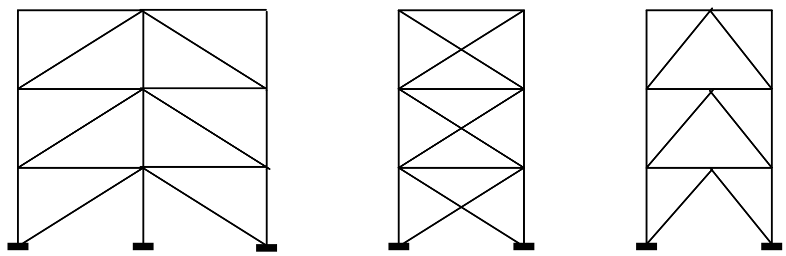
For the lateral load resistance, the beam-column joints in the frames are designed to transfer moment using either rigid or semi-rigid connections. To ensure the formation of the plastic hinge away from the beam-column joint, the widths of the flanges of the beam can be reduced to create a *dog bone connection* (Figure 10.7d).

The recent earthquakes have revealed inadequate performance of the frames due to brittle fracture of the welds. Recommendations are given in IS 800 (Draft) for the seismic detailing of two types of frames, ordinary MRF and special MRF.

b) Braced Frames

Frames may be braced concentrically or eccentrically. The concentric bracing increases the lateral stiffness of the frame, thus increasing the seismic forces. Of course there is a reduction of the lateral drift. Concentrically braced frames (CBF) may be of different types as shown in Figure 10.10. In X-bracing, at least one of the braces in each floor is under tension and hence it is preferred to the diagonal bracing. The diagonal bracing should be provided in pairs of frames, with the diagonals in the two frames inclined in opposite directions. In case of V- or inverted V- (Chevron type) bracing, the horizontal beam may experience an unbalanced transverse force due to the buckling of the brace under compression. Hence, it is should be adequately designed for the same. IS 800 (Draft) covers the design and detailing of ordinary CBFs and special CBFs.

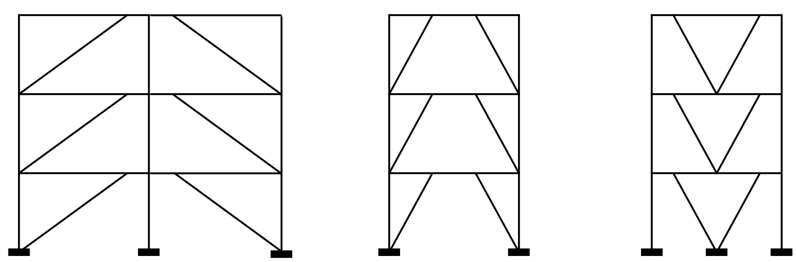
While the braces decrease the bending moments and shear forces in the columns and beams, they increase the axial forces in the columns. Any foundation uplift due to the tension in the column under lateral loads should be avoided by ensuring adequate compression due to gravity loads. The concentric braces usually have lower energy dissipation capacity, especially under the compression range of the cyclic loading.



(i) Diagonal braces (ii) X-braces (iii) Chevron braces

Figure 10.10 Types of concentric braced frames

To reduce the lateral stiffness of the structure and improve the energy dissipation capacity, eccentrically braced frames (EBFs) have been used (Figure 10.11). Due to the eccentric connection of the braces to the beams, the lateral stiffness also depends on the flexural stiffness of the beams. The vertical component of the force of a brace causes a concentrated transverse load on a beam. Under this load, plastic hinge is formed in the stub length of the beam. The stub beam is designed to have adequate ductility and thus dissipate energy. Of course, the stub beam should be designed to avoid the formation of a shear hinge. Although EBFs attract lesser base shear compared to CBFs, they under go larger lateral drift.



(i) Diagonal braces (ii) Split braces (iii) V braces

Figure 10.11 Types of eccentric braced frames (EBFs)

10.6 RETROFIT STRATEGIES FOR MULTI-STOREYED BUILDINGS

The deficiencies in multi-storeyed buildings can be in the configuration or in the capacities of individual members. The deficiencies in the configuration are referred to as irregularities. The definitions of plan and vertical irregularities are given in IS 1893: 2002. These are explained in Chapter 2.

The retrofit strategies for reinforced concrete multi-storeyed buildings are discussed in Chapter 9. Some of them are applicable for steel buildings also. These are not covered in this chapter. Only the special strategies related with steel buildings are mentioned.

10.6.1 Non-Buckling Braces

Conventional braces tend to buckle under the compression cycle of the seismic load, thus dissipating little energy under compression. This causes pinching of the hysteresis loop and failure of the braces within a few cycles, due to the formation of plastic hinge close to middle. The use of non-buckling braces bypasses this problem. These braces are also known as buckling restrained braces or unbonded braces.

The non-buckling brace is an innovative, patented concept (Indian patent No. 155036, dated 30.4.81 and United States patent No. 5175972, dated 5.1.93), which overcomes the problem of buckling and low energy dissipation of regular braces. In this type of braces, the requirements of adequate strength to resist compression and adequate rigidity to avoid buckling, have been assigned separately to a core and a sleeve. First, a suitable section (made of flats or angles) is selected for the core. The core is non-prismatic, that is the cross-section is not constant throughout the length (Figure 10.12). The projection of the core beyond the sleeve should have larger area such that the core does not yield or buckle in that region. By appropriately flaring of the area of the core, the desired strength and stiffness of the brace can be obtained independently. The sleeve is in the form of a cylinder. The space between the core and the sleeve is filled with inert filler such as cement mortar. The bond between the core and the grout is eliminated to ensure that no part of axial force is transferred to the sleeve.

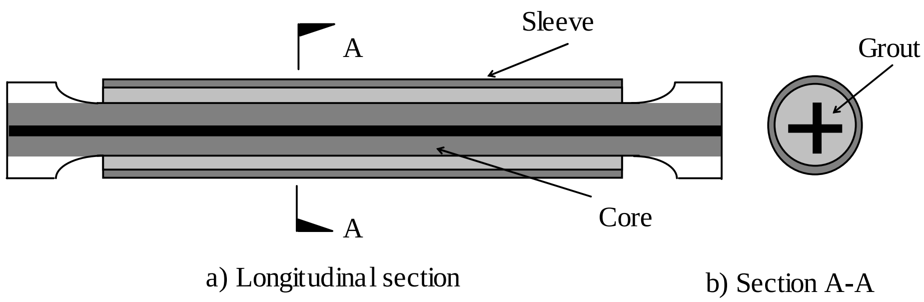


Figure 10.12 A non-buckling brace

In the frames with non-buckling braces, the structure is expected to remain elastic under the design seismic loads and all the yielding (seismic damage) is expected to occur in the non-buckling braces. This is an effective and economical way of achieving a structure with adequate lateral stiffness under moderate seismic loads and large energy dissipation under extreme seismic loads (Kalyanaraman et. al. 1998a, 1998b). Careful proportioning of the areas of the cores of the braces at different storeys can enable dissipation of energy and control inter-storey drift. The connection between the core and the gusset plate may be made using slip resistant (friction grip) bolts. The gusset plates can be designed as discussed in Section 10.4.3.

Since the compressive strength of the core is slightly larger than the tensile strength for the same level of plastic deformation, the V- and inverted V- type of bracing configuration generates an unbalanced transverse force on the horizontal beam. The horizontal beam should be adequately designed for the transverse force or the two configurations should be combined to achieve an X-configuration over two storeys.

Guidelines for the calculations of non-buckling braces are given for the following methods of analysis.

- Equivalent static analysis (linear static analysis)
- Response spectrum method (linear dynamic analysis)
- Push over analysis (non-linear static analysis).

These methods are explained in Chapter 8, Structural Analysis for Seismic Retrofit.

a) Equivalent static analysis

The steps follow the provisions of IS 1893: 2002.

- Calculate the seismic weight and approximate time periods of the building (Clause 7.6).
- Calculate the design horizontal seismic coefficient A_h (Clause 6.4.2) assuming an R value of 4.5, and calculate the base shear V_B.
- Distribute the lateral forces to all the levels (Clause 7.7.1).
- Calculate the shear demand in the i^{th} storey as the sum of the lateral forces acting at all the floors above the storey. If the demand exceeds the capacity of the existing braces, then the additional braces are to be designed for the shear difference, say V_{Di}.
- Calculate the axial force in the non-buckling braces (F_{br}) as follows.

$$F_{br} = \left(\frac{V_{Di}}{N_{br}}\right)\left(\frac{L_{br}}{s}\right) \qquad \textbf{(10.2)}$$

Here,

L_{br} = average length of the braces

N_{br} = number of additional braces in the storey

s = average span (horizontal projected length) of a brace.

The above procedure is based on hand calculations. The individual forces can also be found out by a computational model. The development of a model is discussed in Chapter 8. The required areas for the core and sleeve of a brace are calculated corresponding to the design force. The strength and stiffness of the braces can be altered by the parameters defining the non-prismatic core, to obtain the desired level of storey drifts.

b) Response spectrum method

The design steps are as follows.

- Choose trial sections for the braces as per the equivalent static analysis discussed above.
- Define the response spectrum (Figure 2 of IS 1893: 2002).
- Perform the analysis using a modal superposition method.
- Check the braces for the design forces. Repeat the steps until the sections are adequate.

c) Pushover analysis

In this method, the energy dissipation concept of non-buckling braces can be properly utilised to develop a damage controlled design. The steps can be divided into two stages.

i. Analysis of the existing building
ii. Analysis and design of the retrofit scheme.

i. Analysis of the existing building

- Develop the computational model and the assign the lateral force distribution.
- Define and assign the load versus deformation hinge properties for the elements.
- Perform the pushover analysis.

ii. Analysis and design of the retrofit scheme

- Choose appropriate locations for the braces. The braces should reduce the torsion of the building. For regular buildings, it is better to provide braces symmetrically to avoid shifting the centre of stiffness.
- Choose trial sections for the braces. Define and assign appropriate load versus deformation hinge properties for the braces.
  - It is possible to design the areas of the cores in such a way that all the braces in a storey start yielding at the same step of the pushover analysis.
- Perform the pushover analysis for the building model with the retrofit scheme. At the performance point, the roof displacement and inter-storey drifts for the building should be under the permissible limits. Repeat the steps if necessary.
- Wherever the V- type or the inverted V- type braces are used, check the horizontal beams to which the braces are attached, for the unbalanced transverse forces. Check the columns and foundations for the design forces.

10.6.2 Plate Shear Walls

Plate shear walls may be used to resist the lateral forces and dissipate energy (Berman and Bruneau, 2003). This consists of a steel plate attached to beams at the top and bottom and columns at the two sides (Figure 10.13). The beams and columns are referred to as the boundary elements. The boundary elements, particularly the columns can be designed to remain elastic till the plate yields.

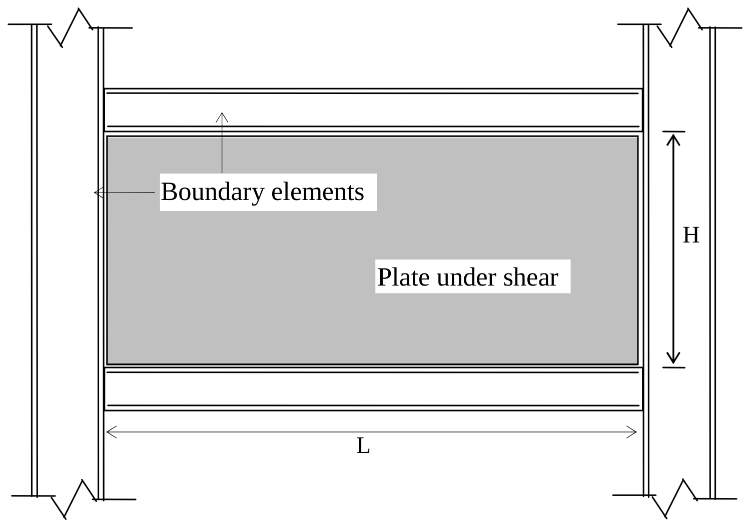


Figure 10.13 Plate shear wall

The design shear strength of a plate shear wall is evaluated as follows.

$$V = 0.42\, f_y\, t_p\, L_{cf}\, \sin 2\alpha \qquad \textbf{(10.3)}$$

Here,

f_y = yield strength of the plate

t_p = thickness of the plate.

The value of α is given by the following equation.

$$\tan^4 \alpha = \frac{H \dfrac{t_p L}{2A_v}}{H\, t_p\, h\left[\dfrac{1}{A_h} + \dfrac{h^3}{360 I_v L}\right]} \qquad \textbf{(10.4)}$$

Here,

A_h = area of a beam

A_v = area of a column

H = height of the plate

I_v = moment of inertia of a column about an axis perpendicular to the plane of the plate
L = length of the plate.

The aspect ratio of the panel should satisfy $0.8 < L/H \leq 2.5$.

10.7 MAINTENANCE

Steel structures can perform without degradation even under severe coastal condition provided they are protected properly against direct exposure of steel to alternate wetting and atmospheric oxygen. Steel structures require maintenance to ensure that the corrosion protection measures are in place all the time. The protection measures involve exposed surface preparation and application of coatings, which can serve as long as 25 years without any further treatment (IS 800 (Draft)). However, claddings in the buildings may leak and consequently allow salt laden moisture to come into direct contact with steel, thus causing corrosion. Once steel starts corroding, simply covering the corroded area with coatings is not adequate. It is necessary to remove the oxides of steel and prepare the surface adequately before applying the necessary coatings. When corrosion is prevented by such maintenance measures, steel structures can have a long life.

10.8 SUMMARY

First, the structural systems of steel buildings are explained. Next, two types of steel buildings are described: single storey industrial sheds and multi-storeyed buildings. For each type, the lateral load resisting systems are reviewed. The possible deficiencies in resisting seismic loads and the retrofit strategies are discussed. The design of non-buckling braces and plate shear walls are explained.

10.9 REFERENCES

1. ANSI/AISC 341-05 (2005), “Seismic Provisions for Structural Steel Buildings”, American Institute of Steel Construction, Inc.
2. ASCE 31-03 (2003), “Seismic Evaluation of Existing Buildings”, American Society of Civil Engineers, USA.
3. FEMA 172 (1992), “NEHRP Handbook of Techniques for the Seismic Rehabilitation of Existing Buildings”, Building Seismic Safety Council, USA.

4. IS 800 (Draft), "Indian Standard Code of Practice for General Construction in Steel", Bureau of Indian Standards.
5. IS 1893: 2002, "Criteria for Earthquake Resistant Design of Structures", Part 1: General Provisions and Buildings, Bureau of Indian Standards.
6. Berman, J. and Bruneau, M. (2003), "Plastic Analysis and Design of Steel Plate Shear Walls", Journal of Structural Engineering, American Society of Civil Engineers, Vol. 129, No. 11, pp. 1448 – 1456.
7. Kalyanaraman V., Sridhara B. N. and Ramachandran B., (1998a) "A Sleeved Bracing System for Earthquake Resistant Design of Tall Buildings", Proceedings of the 11th Symposium on Earthquake Engineering, University of Roorkee, pp 713 – 720.
8. Kalyanaraman, V., Sridhara, B. N., Thairani, V., (1998b), "Core Loaded Earthquake Resistant Bracing System", Journal of Constructional Steel Research, Vol. 46, No. 1-3, pp 215.
9. Lamarche C. P., Proulx J., Paultre, P. (2004), "An Experimental Investigation of the Dynamic Characteristics of Low Rise Steel Structures", Fifth Specialty Conference of the Canadian Society of Civil Engineers, Saskatchwan, Canada.
10. Yam M. C. H. and Cheng J. J. R. (2002), "Behavior and Design of Gusset Plate Connections in Compression", Journal of Constructional Steel Research, Vol. 58, pp. 1143–1159.

11

MITIGATION OF GEOTECHNICAL SEISMIC HAZARDS

11.1 OVERVIEW

The geotechnical seismic hazards include ground cracks, heave, liquefaction, failure of slopes and tsunami. In order to evaluate the hazard at a particular site, it is necessary to undertake a site characterisation. The site characterisation is presented in three sub-topics: study of seismicity, subsurface investigation and dynamic soil properties. The study of seismicity identifies the sources of ground motion, evaluates their potential for strong ground shaking. The faults near a site are identified. For subsurface investigation, the common field tests are explained. Among dynamic soil properties, the shear modulus is explained. Based on the site characterisation, classification of a site and a site-specific response analysis are undertaken.

During an earthquake, the cyclic stresses in the soil due to the propagation of shear waves from the bed rock, cause an increase in the pore water pressure. The soil loses its shear strength and acts similar to a viscous liquid. This phenomenon is termed as the liquefaction of soil. The liquefaction potential at a site is commonly expressed in terms of a factor of safety. The methods of evaluating the liquefaction potential based on the standard penetration tests and the cone penetration tests are briefly explained.

The unfavourable soil conditions can be mitigated by certain measures. The ground improvement techniques are divided into four categories: densification, reinforcement, grouting/mixing and drainage techniques. Among these techniques, densification is probably the most commonly used technique. The strengthening of slopes includes the use of geo-grids, rock anchors, gabions and stone columns. These techniques are explained with illustrations.

11.2 INTRODUCTION

A proper understanding of geotechnical earthquake engineering principles is necessary for rational seismic resistant design and retrofit of buildings. Past earthquakes have shown that the effects of strong ground shaking are diverse. During the 1999 Turkey earthquake, a building collapsed due to liquefaction of the ground with little or no damage to the structure itself. In contrast, during the 1999 Taiwan earthquake, a building was heavily damaged due to the intense shaking of a portion of the building.

This chapter discusses the types of hazards, characterisation of sites based on local effects, response of site during earthquakes, evaluation of liquefaction potential and mitigation of the hazard.

11.3 TYPES OF HAZARD

The hazards associated with ground displacements are the development of ground cracks and faults, ground heave, liquefaction-induced sinking of foundations and lateral spread of soil, failure of slopes, failure of retaining walls and generation of tsunami. Each of the hazards is briefly discussed.

1. Ground cracks range from small fissures to long and deep faults. The cracks can transect roads, railway tracks, dams, etc., and can cause widespread damage in buildings. Important infrastructure facilities such as power cables, telephone and high speed data cables, water, sewage, gas and oil pipelines can also be damaged.
2. Ground heave refers to the uplift of the soil along a fault line. Figure 11.1 shows the condition of a sprinting track after the 1999 Taiwan earthquake.
3. Liquefaction is the temporary loss of the strength of the soil and subsequent fluidization during ground shaking. The reduction in bearing capacity during liquefaction can lead to the sinking of building foundations causing varying degrees of damage to the superstructure. The phenomenon is further discussed subsequently.

4. Slopes of soil and rock can fail during an earthquake, particularly those that are marginally stable before the earthquake.
5. Tsunami refers to the waves generated when the fault of an earthquake is located at the sea-bed and there is an upward displacement of the sea-bed.

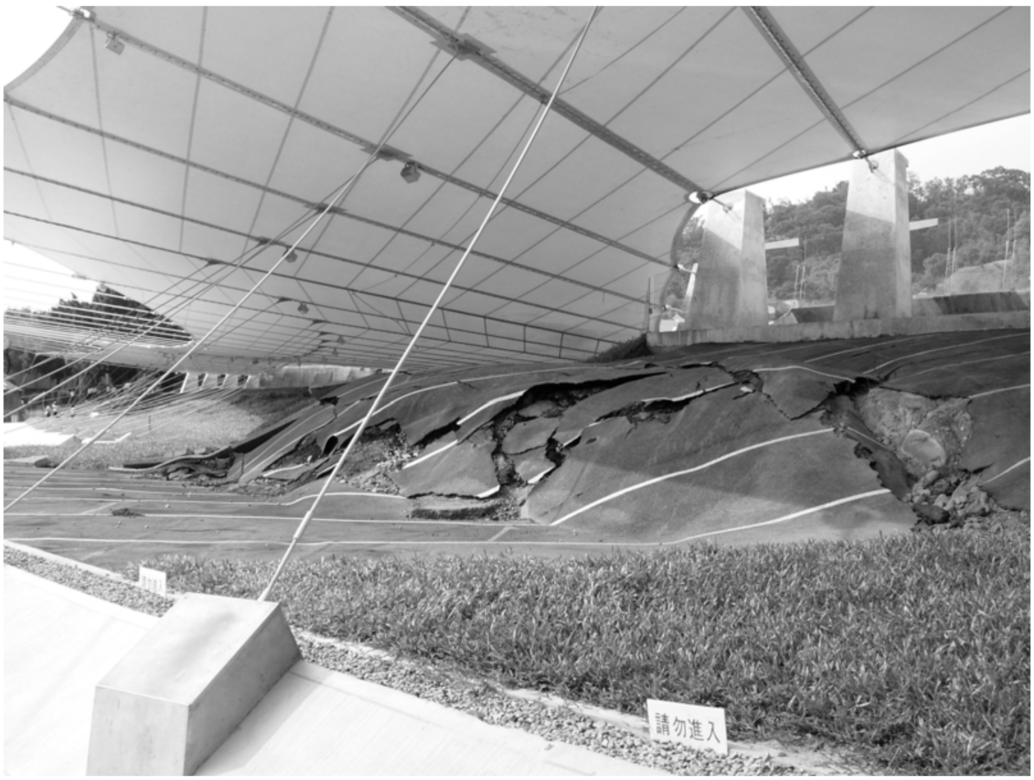


Figure 11.1 Ground heave during Chi Chi earthquake in Taiwan, 1999

11.4 SITE CHARACTERISATION

One of the important steps in the evaluation of geotechnical seismic hazard is site characterization. The information is used to develop site specific response spectrum and to select methods to mitigate the hazards. The scope of the investigation depends on the seismicity of the area, soil conditions at the site and the proposed/existing structure. The investigation involves acquisition, synthesis and interpretation of information about the fault lines, local soil conditions, potential of large-scale liquefaction, mudflows, subsidence and landslides.

11.4.1 Study of Seismicity

The objective of a seismicity study is to quantify the level and characteristics of ground motion at a site. The study involves detailed examination of available geological data and past seismic activities (epicentre, magnitude and intensity database) to establish patterns and to locate possible sources of earthquakes and their associated mechanisms. Subsequently, typical ground motion time histories called synthetic/scenario earthquakes, are developed. The process by which the parameters of the ground motion time histories are established is termed as the seismic hazard analysis. A seismic hazard analysis involves the following steps.

i. Identification of the sources capable of generating strong ground motion at the site
ii. Evaluation of the potential of each source, which is referred to as seismic source characterisation
iii. Evaluation of the intensity of the design ground motion at the site.

The data on past seismic activities in India can be obtained from Geological Survey of India and Indian Meteorological Department. The study of seismicity is used in seismic micro-zonation. The later refers to the identification of faults and local soil conditions in a certain area. The information is used in the seismic design of buildings and seismic hazard assessment.

11.4.2 Subsurface Investigation

Subsurface investigation is an important step to identify all relevant information about the local soil conditions. Exploration can be conducted up to the depths recommended in the following table or to bedrock / hard soil layer encountered within these depths.

Table 11.1 Typical values of depths of exploration

Objectives	Tall buildings or structures with deep excavated foundations	Less important buildings
Determination of properties for dynamic response of soil	50 m	20 m
Assessment of liquefaction potential	15 m	10 m
Detection of faults	50 m	30 m

The thickness and properties of soil layers are obtained through sampling and testing in a laboratory or by field tests. The common soil parameters include water content, specific gravity, grain size analysis, Atterberg limits and the compressive strength. In addition to these parameters, the stiffness and shear wave velocity need to be determined. The following field tests are conducted to determine the in-situ properties of soil.

1. Standard penetration test
2. Seismic cross-hole test
3. Seismic up-hole (down-hole) test
4. Spectral analysis of surface waves test

Standard penetration test

The standard penetration test (SPT) consists of driving a standard sampler through a specified depth in a borehole. The driving is achieved by dropping a standard mass through a certain height on the sampler. The number of blows (N) to drive the sampler is recorded. The procedure of the test is given in IS 2131: 1981. The value of N is a measure of stiffness of the soil.

Seismic cross-hole test

The seismic cross-hole test determines the velocities of compression wave (P-wave) and shear wave (S-wave) of soil at a certain depth. The test involves the generation of stress waves at a certain borehole and detecting the waves by a receiver in another borehole. The durations of travel of the waves between the boreholes are recorded. Details of the test is given in ASTM D4428-00. The calculated shear wave velocity (v_s) is used to determine the shear modulus of the soil layer. Figure 11.2 shows the wave velocity profiles at the Prototype Fast Breeder Reactor (PFBR) site in Kalpakkam, Tamil Nadu.

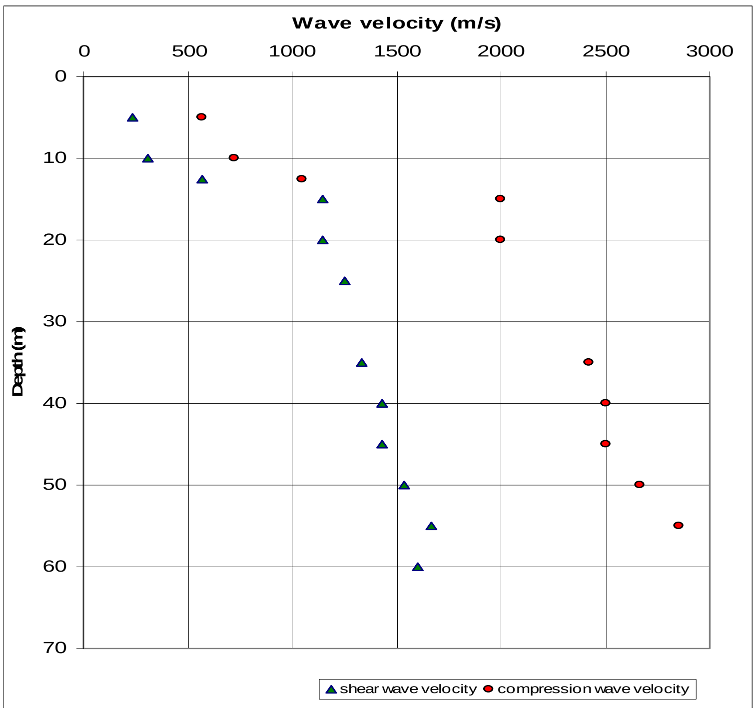


Figure 11.2 Shear wave and compression wave velocity profiles at the PFBR site, Kalpakkam

Seismic up-hole (down-hole) test

The seismic up-hole or down-hole tests are similar to the cross-hole test. But only one borehole is required to perform these tests. In the up-hole method, the receiver is placed at the surface, and the shear waves are generated at different depths within the borehole. On the contrary, in the down-hole method, the excitation is applied at the surface and one or more receivers are placed at different depths within the borehole (Figure 11.3).

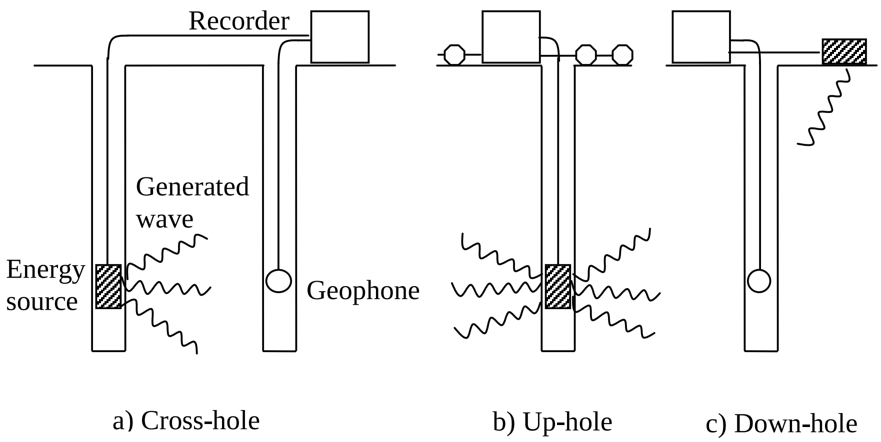


Figure 11.3 Schematic diagrams of cross-hole, up-hole and down-hole tests

Spectral analysis of surface waves (SASW) test

Spectral analysis of surface waves test is a recently developed test, that does not require boreholes. The test uses two receivers that are spread out along a line at the surface and the source of energy is a hammer or tamper also at the surface. The output of both the receivers is used to develop the shear wave velocity profile of the soil. The details of the test and other tests are provided by Bowles (2001) and Ansal (2004).

11.4.3 Dynamic Soil Properties

The behaviour of soils under dynamic load plays a crucial role in the determination of ground displacements. The soil properties required in a dynamic analysis are the shear modulus, Poisson's ratio, unit weight and damping. Here, only the shear modulus is explained.

Dynamic Shear Modulus

The dynamic shear modulus (G) can be determined in the laboratory by the resonant column test. For in-situ evaluation, the shear wave velocity (v_s) obtained from the tests mentioned earlier can be used. The equation relating the two quantities is as follows.

$$G = \rho\, v_s^2 \tag{11.1}$$

Here, ρ is the density of soil. In absence of field tests, empirical relationships can be used to calculate the value of G (Bowles, 2001). Table 11.2 provides the typical ranges of G for several generic soil types.

Table 11.2 Typical values of dynamic shear modulus of soil

Type of soil	***G* (kPa)**
Soft clays	2,750 – 13,750
Firm clays	6,900 – 34,500
Silty sands	27,600 – 138,000
Dense sands and Gravel	69,000 – 345,000

11.5 MODELLING OF SITE EFFECTS

The soil profile at a site can have a profound effect on the ground motion. The local soil conditions affect the intensity, frequency content, and duration of the motion at the ground surface or at level of the foundation. The motion at the bedrock can be amplified several times due to the local soil profile.

11.5.1 Types of Site Effect

The site effects on the ground motion include the following.

1. The response of shallow sediments (local response effect)
2. The effect of deep sediments (basin response effect)
3. The effect of irregular surface topography (topographic effect)

Among the three site effects, the local response effect is first considered in practice. For major projects and critical facilities, and for deep deposits of soft clay or other special soils, the other two effects should also be studied.

11.5.2 Site classification

The classification of sites based on the local soil conditions is used in the modelling of site effects in the seismic design of buildings. As per ASCE 7-02, the site classification is based on the average shear wave velocity in the upper 30 m of the soil ($\bar{v}_s$). The average shear wave velocity for the layers is determined from the equation given below.

$$\bar{v}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}} \tag{11.2}$$

Here,

d_i = thickness of layer i (m)
n = number of distinct layers of soil in the upper 30 m
v_{si} = shear wave velocity in layer i (m/s).

The six site classes as per ASCE 7-02 are reproduced in Table 11.3.

Table 11.3 Site classification

Site class	**Nomenclature**	$\bar{v}_s$ **in m/s**
A	Hard Rock	>1500
B	Rock	760 to 1500
C	Very dense soil and soft rock	370 to 760
D	Stiff Soil	180 to 370
E	Soil	<180
F	The sites requiring specific evaluation are classified here.	

Based on the site class, the response spectrum is modified with coefficients.

11.5.3 Site-Specific Response Analysis

The methods of analysis of a building for seismic forces are traditionally based on a design response spectrum. The methods and the design response spectrum are explained in Chapter 8, Structural Analysis for Seismic Retrofit. But for important structures it is preferred to do the analysis based on a site-specific response spectrum. A site-specific response spectrum is developed based on the seismicity at the site, the soil conditions and soil attenuation relationships.

The site-specific response spectrum requires the information of the magnitude of earthquake and the acceleration of the bed rock expected at the site. These quantities are evaluated from a seismic hazard analysis. Next, for evaluating the effect of local soil conditions on the ground motion several methods are now available. Most of these methods are based on the assumption that the main response in a soil deposit is caused by the upward propagation of shear waves from the underlying rock formation through the horizontal soil layers (Figure 11.4). Methods based on this one-dimensional wave propagation theory have given results in fair agreement with field observations for a number of cases.

The computer program SHAKE (developed at the Earthquake Engineering Research Center, University of California Berkeley, USA) is based on the one-dimensional wave propagation theory. If the ground motions are known or specified at Point A, and the properties of the soil layers are provided, the SHAKE program can be used to compute the motion at the base of the soil column (Point A$^{/}$). The program can then find the motion at any other point (say Point B), which is the top of another soil column. There are some other programs catered to specific analyses.

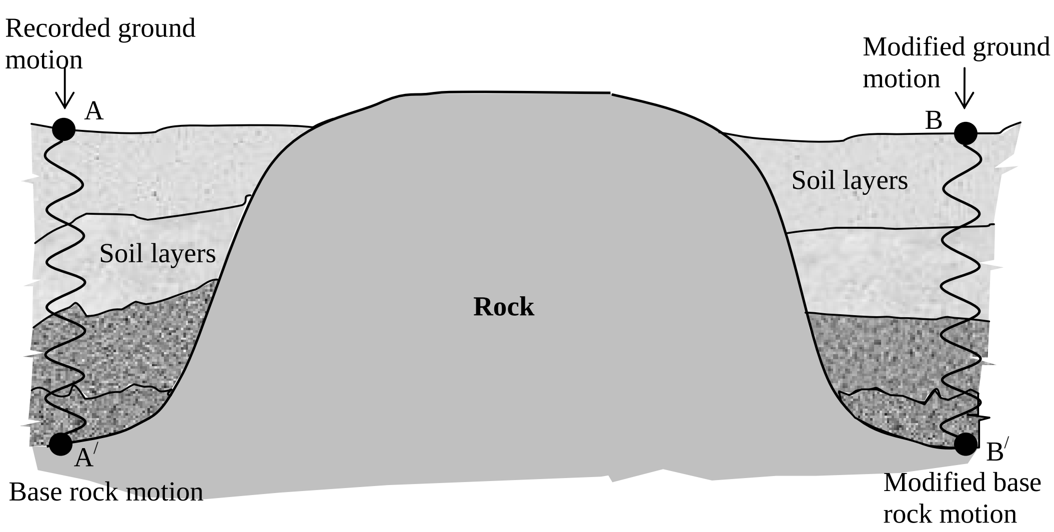


Figure 11.4 Schematic representation of computing the effect of local soil conditions

11.6 LIQUEFACTION

11.6.1 General

During an earthquake, the stresses induced in saturated granular soil due to the propagation of shear waves cause reduction of the volume of soil and an increase in the pore water pressure. In certain cases, the pore water pressure exceeds the existing effective overburden pressure and hence, the soil loses its shear strength and acts similar to a viscous liquid. This phenomenon is termed as the liquefaction of soil. In a built environment, it leads to failure of the foundation of buildings, tilting of retaining walls, damage of the service pipelines, fracture of pavements, etc.

The increase in pore water pressure leads to upward flow of water to the ground surface, where it can emerge in the form of sand boils or mud spouts.

11.6.2 Factors Affecting Liquefaction

The three primary factors controlling liquefaction are as follows.

1. Characteristics of ground motion: The character of ground motion (amplitude and frequency content) controls the development of shear strains that cause liquefaction. Even for the same acceleration, higher magnitude earthquakes are more damaging because of the higher number of cycles of strain.
2. Type of soil: The soils which are susceptible to liquefaction are clay free deposits of sand and silt. The potential for liquefaction depends on the looseness of the soil, amount of clay (cementing material), the particle size gradation and the restriction to drainage. Well-graded soils are generally less susceptible to liquefaction than poorly graded soils. Soils with rounded particle shapes densify more easily and hence, are less susceptible to liquefaction than angular grained soils.
3. In-situ stress condition of the soil: A granular soil is susceptible to liquefaction when it is saturated in water. This occurs in low lying areas near rivers, lakes or oceans, where the ground water level is high, say within 10 m of the ground surface.

11.6.3 Evaluation of Liquefaction Potential

Because of difficulties in analytically modeling soil conditions at liquefiable sites, the use of empirical methods is popular in conventional engineering practice. Two approaches are used to estimate the liquefaction potential, one based on standard penetration tests (SPT) and the other based on the cone penetration tests (CPT). The data from these tests provide a measure of the resistance to liquefaction in a certain layer of soil, which is termed as the cyclic resistance ratio (CRR). A cyclic stress ratio (CSR) is evaluated as the measure of stress during an earthquake on the layer under consideration. The ratio of CRR to CSR gives a factor of safety, which is a measure of the liquefaction potential of the soil layer.

<u>Method based on Standard Penetration Test</u>

The basic procedure of this method is as follows.

1. The standard penetration test count (N) is evaluated by field test as per IS 2131: 1981.
2. The count N is corrected to normalise for an effective overburden pressure of 100 kPa and to 60 percent of the energy input by the equipment. The corrected blow count is represented as $(N_1)_{60}$.
3. The CRR is estimated from $(N_1)_{60}$ using a correlation chart.
4. The factor of safety against liquefaction is defined as the ratio of CRR to CSR.

Table 11.4 can be used to have a qualitative estimate of the liquefaction potential from values of $(N_1)_{60}$.

Table 11.4 Liquefaction potential based on corrected SPT count

Values of $(N_1)_{60}$	**Liquefaction potential**
0 to 20	High
20 to 30	Intermediate
Greater than 30	Insignificant

<u>Method based on Cone Penetration Test</u>

The basic procedure of this method is similar to that based on SPT.

1. The cone penetration resistance (q_c) is evaluated by field test as per IS 4968: 1976 (Part 3).

2. The value of resistance is corrected to normalise for an effective overburden pressure of 100 kPa and also based on content of fine particles. The corrected resistance is represented as q_{c1}.
3. The CRR is estimated from q_{c1} using a correlation chart.
4. The factor of safety against liquefaction is defined as the ratio of CRR to CSR.

Further details of the methods are provided by Kramer (1996). A liquefaction analysis would typically not be needed for those sites having peak ground acceleration less than 0.10g or magnitude of earthquake less than 5.

11.7 GROUND IMPROVEMENT TECHNIQUES

11.7.1 General

For a new construction the liquefaction potential can be mitigated by avoiding construction at susceptible soils or designing the foundation appropriately. For an existing building, the liquefaction potential can be mitigated by ground improvement techniques. These techniques aim to densify and hence increase the strength and stiffness of the soil, and/or improve the drainage characteristics. The selection of a technique depends on the site and available equipment. This section briefly describes the available options.

11.7.2 Densification Techniques

> Densification is one of the most effective methods of improvement of soil characteristics for mitigation of seismic hazards. Of course for an existing building, densification is limited in application. Although the amplitude of ground displacement may decrease after improvement, the acceleration may be greater as compared to that in the existing condition.

The methods of densification include vibro-flotation, dynamic compaction, blasting and compaction grouting.

(a) Vibro-flotation

The vibro-flotation technique uses a probe that is vibrated through soil layers in a grid pattern to densify the soil. In addition a vertical borehole may be drilled, which is filled with compacted gravel or crushed rock to dissipate the pore water pressure during an earthquake. (Figure 11.5). The vertical columns should be closely spaced for effective dissipation of pore water pressure.

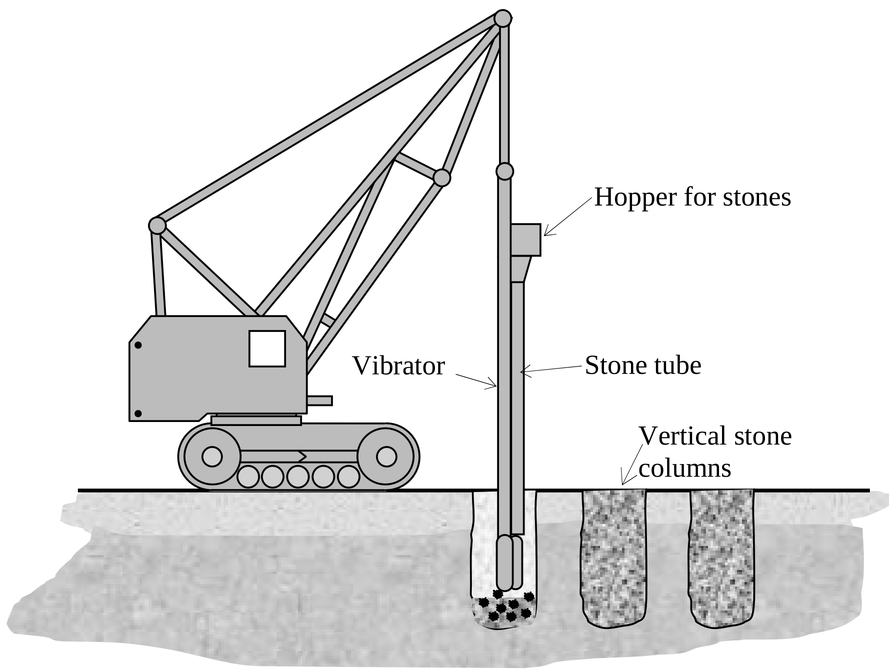


Figure 11.5 Vibro-flotation technique

(b) <u>Dynamic compaction.</u>

Dynamic compaction is performed by repeatedly dropping a heavy weight (10 to 20 tons through a height of 15 to 20 m) in a grid pattern on the ground surface (Figure 11.6). The impact causes local liquefaction followed by drainage of the water and densification of the soil. This method is effective to densify granular soil up to a depth of 9 to 12 m. The process requires filling of the craters formed by the impact and re-leveling of the ground surface.

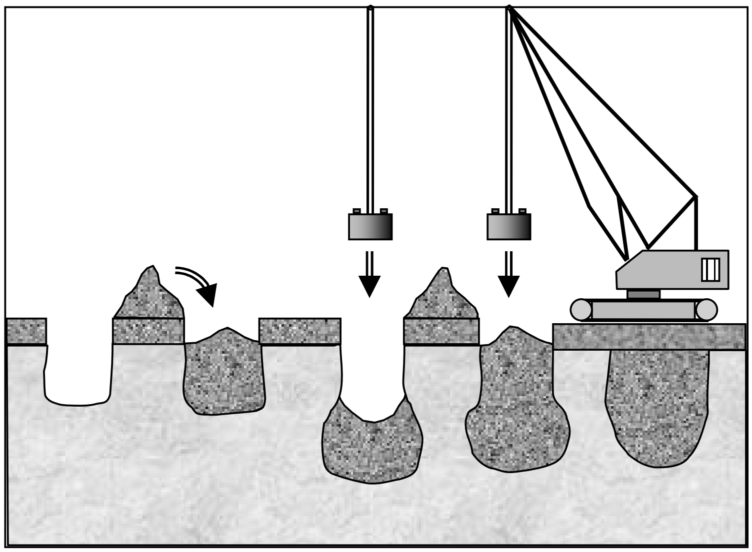


Figure 11.6 Dynamic compaction technique

Most contractors prefer to use any one method because they have experience with it and have invested in the equipment. Each method works best in certain soils and poorly in others. Therefore, the method to be used in a certain situation has to be carefully chosen.

(c) Blasting

Blasting is used to densify loose granular soils. This technique is suitable for loose sand with less than 20 percent silt and 5 percent clay. It involves the detonation of multiple explosive charges vertically spaced 3 to 6 m apart in drilled boreholes. After detonation, gas and water are expelled which leads to the densification of the soil. The technique is not applicable near existing structures. Figure 11.7 shows a schematic representation of this method.

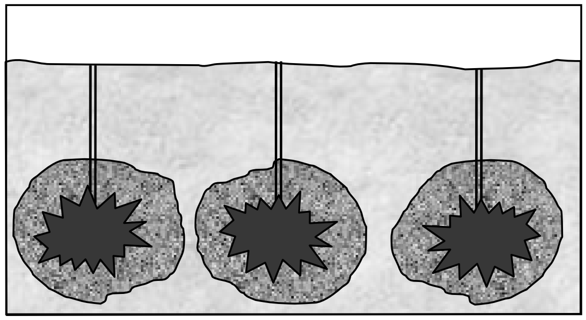


Figure 11.7 Soil densification by blasting

(d) Compaction Grouting

Compaction grouting is most suitable for existing buildings. It involves injection of low slump (less than 75 mm) mortar-type grout into soils at high pressures (3 to 4 MPa). The soil is displaced and densified by the formation of a bulb of the grout (Figure 11.8). Compaction grouting has been used in inclined position and up to depths of 30 m.

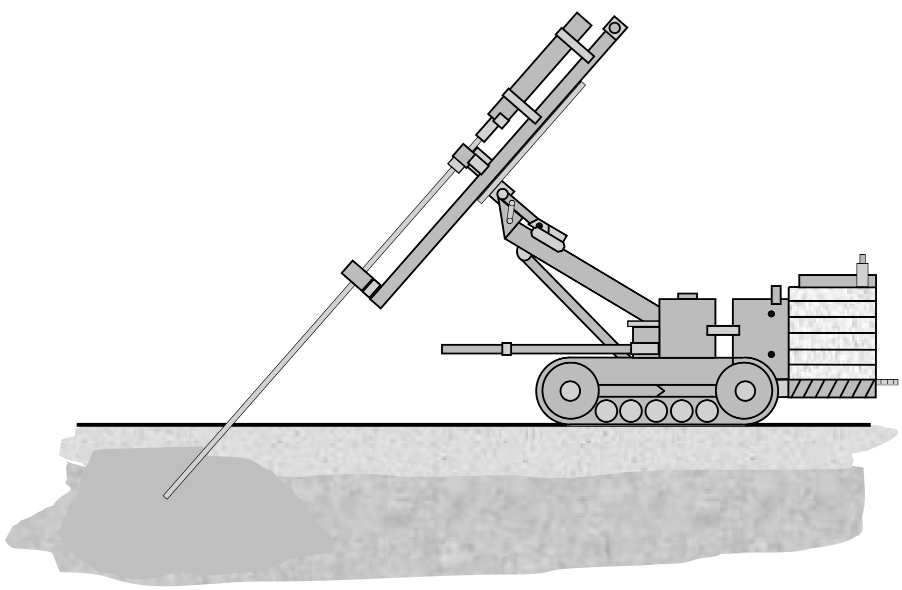


Figure 11.8 Compaction grouting technique

For an existing building, holes are required to be drilled along the perimeter of the foundation, both outside as well as within the building area. Depending on the permeability of the soil and capacity of the grout equipment, the lateral spread of the grout into the soil shall be established by field trials. This will decide the centre to centre distance to be adopted between the grout holes. A typical arrangement of grouting pattern is shown in Figure 11.9. If the size of the footing is large compared to the lateral spread of grout, it may become necessary to drill the holes through the footing itself or to use inclined drilling to ensure that the entire zone below the foundation is improved.

There are no theoretical relationships to estimate the shear strength achieved after grouting based on the spacing of grout holes and grout pressure. It is therefore important to establish the same by field trials. The approximate depth of treatment is usually twice the width of the footing below the foundation level. The plan area to be treated shall cover a distance of half the width of foundation beyond the edge of the foundation.

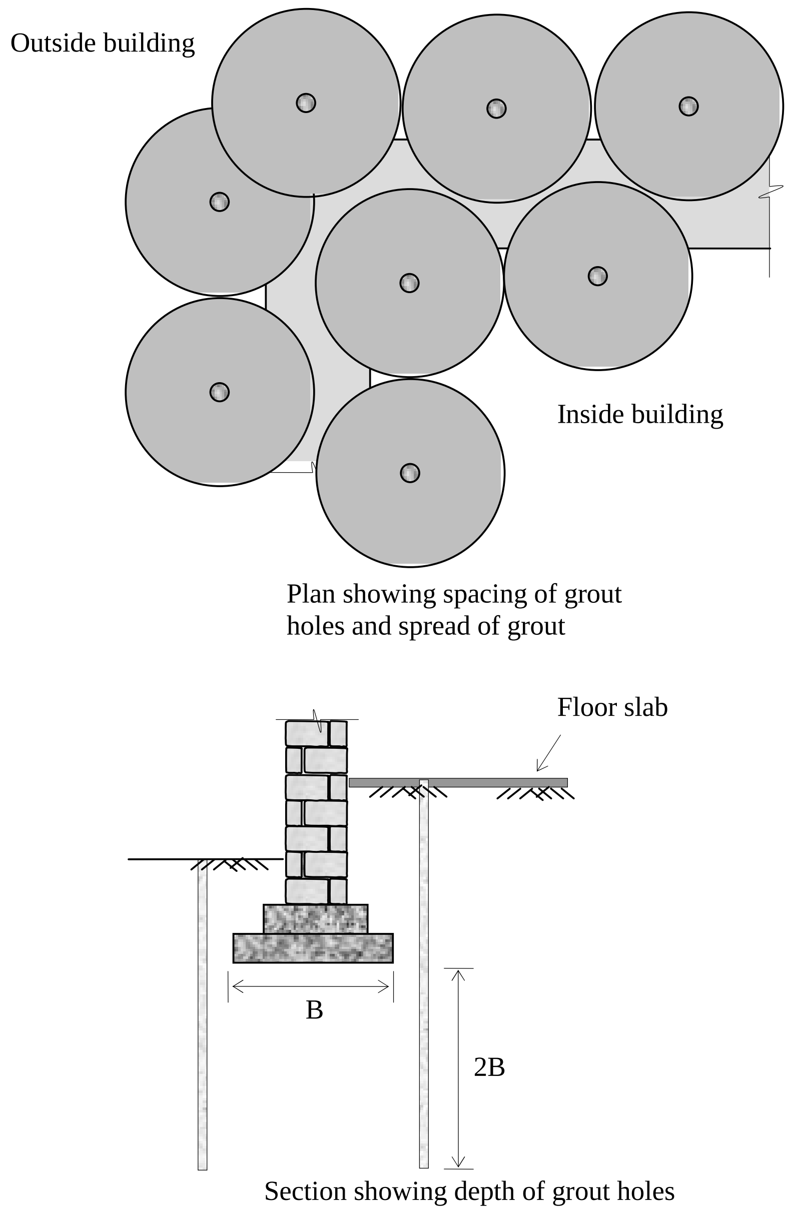


Figure 11.9 Typical arrangement of grout holes

11.7.3 Reinforcement Techniques

In some cases, it is possible to improve the strength and stiffness of an existing soil deposit by installing discrete inclusions that reinforce the soil. These inclusions may consist of structural members of steel, concrete, or timber or geo-materials such as densified gravel.

(a) Stone columns

Columns of gravel installed in soil are known as stone columns. They are commonly used for improvement of liquefiable soil deposits. Stone columns are constructed by introducing gravel during the process of vibro-floatation. Three different mechanisms are involved in the improvement of the soil deposits. First, the stone columns improve the deposit by virtue of their high density, strength and stiffness. Second, they provide closely spaced drainage spaces that inhibit the development of high pore water pressure during an earthquake. Third, the process of installation densifies and increases the lateral stresses in the surrounding soil.

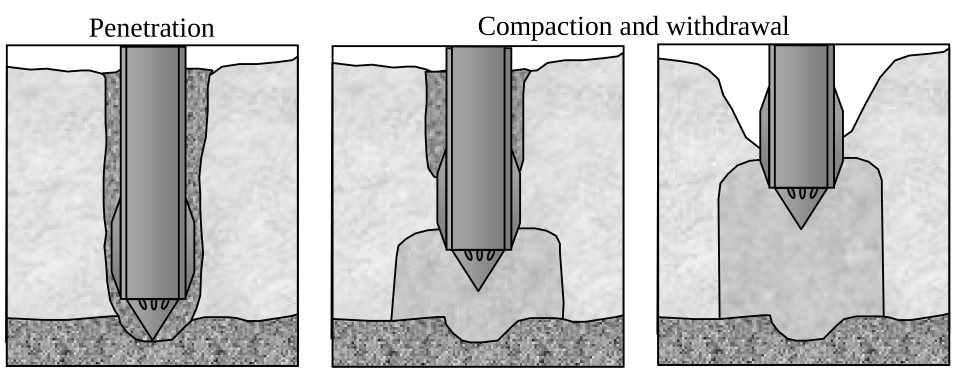


Figure 11.10 Sand compaction pile

(b) Compaction piles

Compaction piles are usually prestressed concrete or timber piles that are driven into loose sand or gravel deposit in a grid pattern. Compaction piles improve the soil deposit by the following mechanisms. First, the flexural strength of the piles provides resistance to soil movement. Second, the vibrations and displacements produced during their installation densify and increase the lateral stresses in the soil surrounding the piles. Relative densities up to 75 to 80 percent are usually achieved in the zones between the piles.

11.7.4 Grouting and Mixing Techniques

The engineering characteristics of many soil deposits can be improved by injecting or mixing cementitious materials into the soil. These materials strengthen the contacts between soil grains and fill the void space between the grains. The techniques are expensive but can often be accomplished with minimal settlement or vibration.

> The grouting techniques involve the injection of cementitious materials into the voids and fractures in the soil. The particle structure of the majority of the soil remains intact. On the contrary, mixing techniques introduce cementitious materials by physically mixing them with the soil, thus completely disturbing the particle structure of the soil.

(a) Grouting

Permeation grouting involves the injection of low viscosity liquid grout into the voids of the soil. Particulate grouts include aqueous suspensions of cement, fly ash, bentonite, microfine cement or combination of them. Chemical grouts include silica and lignin gels, or phenolic and acrylic resins. The suitability of the grouts is determined based on the grain size of the soil. The presence of fines can significantly reduce the effectiveness of permeation grouting.

In intrusion grouting, grout of higher viscosity is injected under pressure to cause controlled fracturing of the soil. The improvement results from the increased stiffness and strength of the soil mass after hardening of the grout.

(b) Mixing

The cementitious material is mixed with the soil using hollow stem auger and paddle arrangement. The strength of the soil-cement mixture depends on the type of grout, type of soil and degree of mixing. After the mixing there is a uniform column of soil-cement.

11.7.5 Drainage techniques

The build up of pore water pressure during an earthquake can be mitigated by providing drainage channels. This can be achieved by installing columns of stone, gravel or any synthetic material. Unacceptable movements of slopes, embankments, retaining structures and foundations can be eliminated or reduced by lowering the ground water table.

11.7.6 Preservation of Moisture

Lightly loaded structures founded on conventional shallow strip footing over highly plastic clays can be strengthened by moisture preservation techniques. Under drought conditions, deficiency in moisture causes shrinkage of the soil, leading to possible tilting of the external walls. The intervention in such cases may be undertaken in the following four stages.

i. Environmental control

ii. Soil stabilization

iii. Moisture preservation

iv. Underpinning.

Here, the moisture preservation method is briefly described. The data regarding anticipated moisture deficiency below ground level is shown in Figure 11.11 (a). The proposed moisture preservation scheme is shown in Figure 11.11 (b). The scheme uses geo-fabric encapsulated brick bat filled peripheral drains. This is to achieve periodic self-saturation with rain water discharge from the roof. The scheme ensures preservation of moisture up to 3 to 4 m depth. The encapsulation of brick bat material with geo-fabric prevents clogging and makes the drain work with maximum efficiency. The peripheral drains are located as close as possible to the edge of wall and are spaced at 1.5 to 2m.

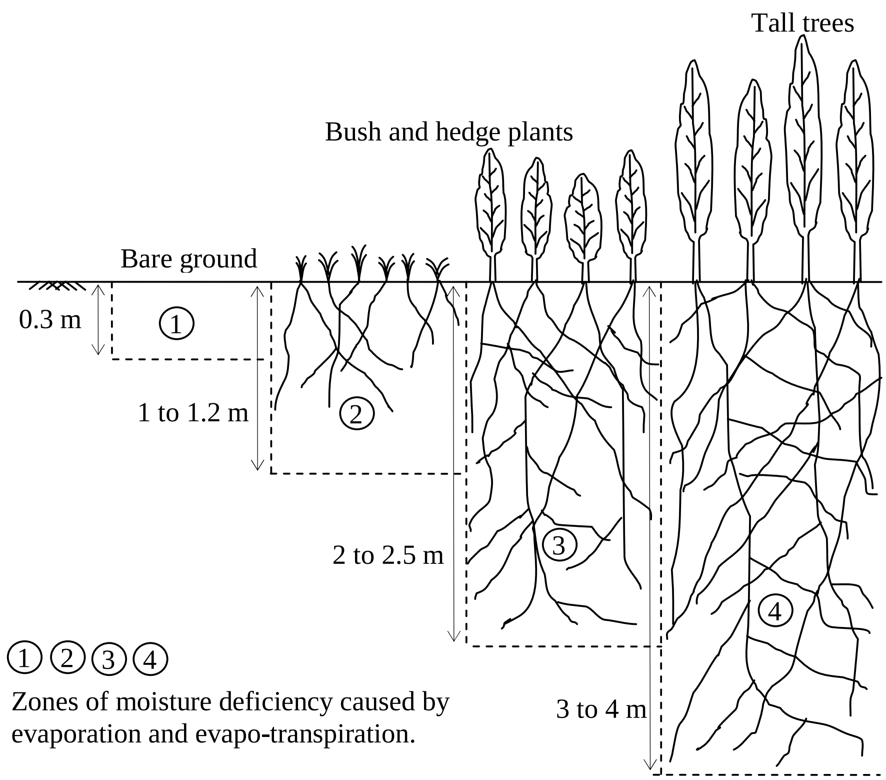


Figure 11.11 a Anticipated depth of moisture deficiency

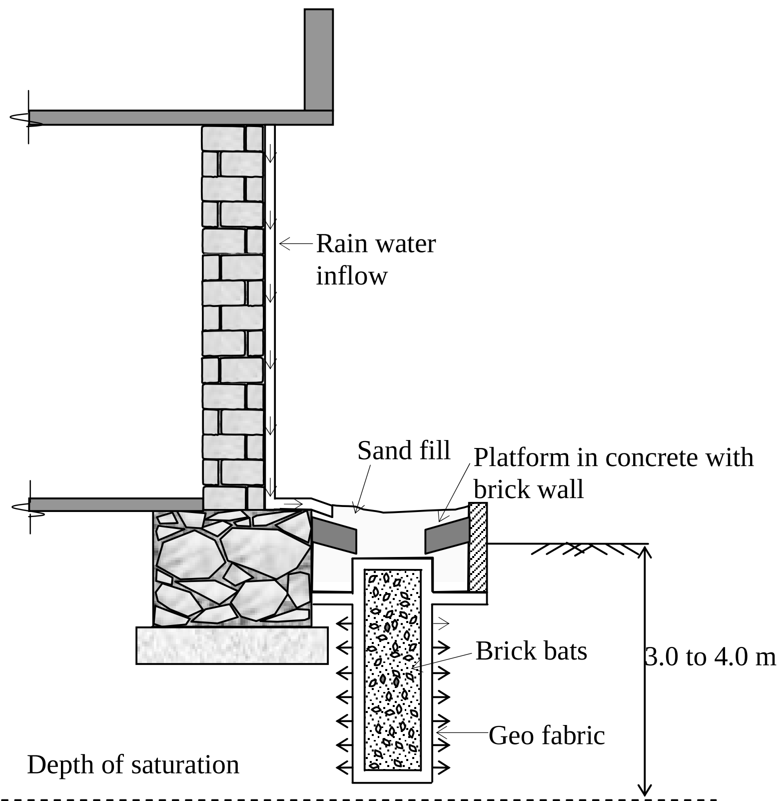
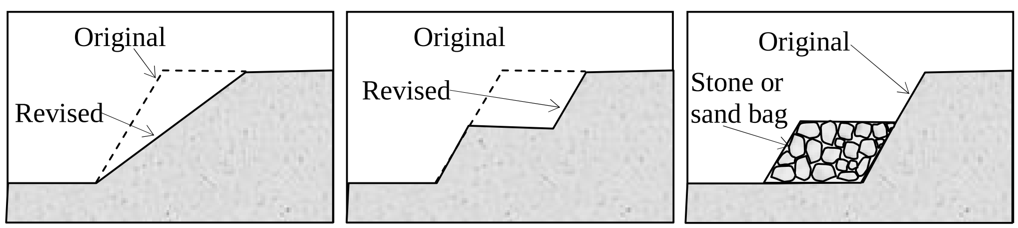


Figure 11.11 b Moisture preservation scheme

11.8 STRENGTHENING OF UNSTABLE SLOPES

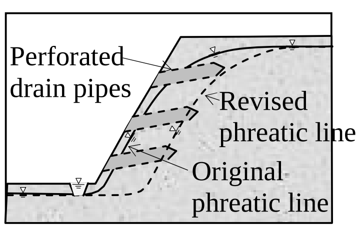
In several cases, earthquake induced land slides result in damage to the houses and other constructed facilities with loss of lives. The highways get blocked by the debris of the collapsed soil, resulting in hardship during relief operations. It is therefore very important to check the stability of slopes close to the important roads and structures. The slope stability analysis should consider appropriate earthquake forces. If the factor of safety is less than one, the slope needs to be suitably strengthened. The following are different ways of improving the unstable slopes (Figure 11.12).

i. Change the geometry of the slope by one of the following ways.
   a. Provide flatter slope
   b. Provide berms at suitable locations
   c. Deposition of soil or stones at the toe of the slope to increase the stabilizing moment
ii. Provide internal drain pipes to lower the phreatic line
iii. Provide protection with geo-grid anchored into the slope.
iv. Provide horizontal support using stone filled gabions
v. Provide prestressed rock anchors through the unstable mass which will increase the normal stress on the failure plane (IS 14448: 1997).
vi. Provide suitable ground improvement technique such as stone columns to prevent deep seated base failure.

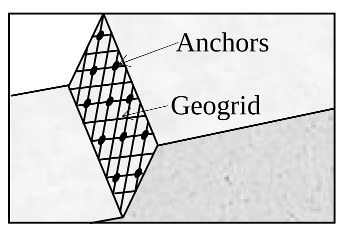


(i) Flatter slope (ii) Berms (iii) Sand bags or stones

(a) Changing the geometry of the slope



(b) Internal drains



(c) Geogrids

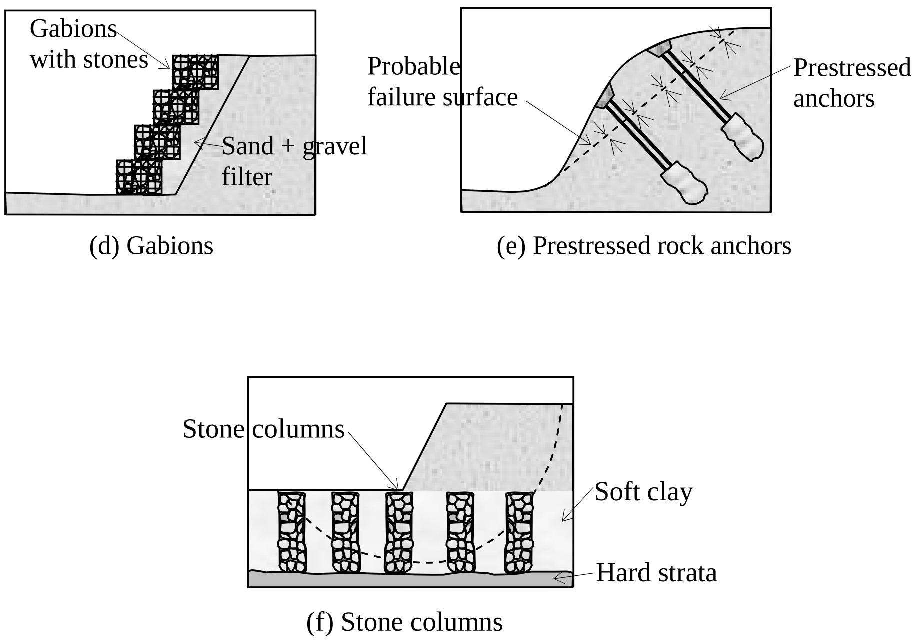


Figure 11.12 Strengthening of unstable slopes

11.9 SUMMARY

In this chapter first the types of geotechnical hazards are explained. The site characterisation explains the study of seismicity, subsurface investigation and dynamic properties of soils. The common field tests such as standard penetration test, seismic cross-hole, up-hole and down-hole tests and spectral analysis of surface waves test are briefly discussed.

Under the modelling of site effects, the types of site effect, site classification and site-specific response spectrum are explained. Next, the phenomenon of liquefaction, the factors and the evaluation of liquefaction potential are elucidated. Finally, the ground improvement techniques and strengthening of slopes are illustrated.

11.10 REFERENCES

1. ASCE 7-02 (2002), "Minimum Design Loads for Buildings and Other Structures", American Society of Civil Engineers.
2. ASTM D4428-00 (2000). "Standard Test Methods for Cross-hole Seismic Testing", American Society for Testing and Materials, USA.
3. IS 2131: 1981, "Indian Standard Method for Standard Penetration Test for Soils", Bureau of Indian Standards.
4. IS 4968: 1976, "Indian Standard Method for Subsurface Sounding for Soils, Part 3: Static Cone Penetration Test", Bureau of Indian Standards.
5. IS 14448: 1997, "Indian Standard Code of Practice for Reinforcement of Rock Slopes with Plane Wedge Failure", Bureau of Indian Standards.
6. Ansal, A. (2004). "Recent Advances in Earthquake Geotechnical Engineering and Microzonation", Kluwer Academic Publishers.
7. Boominathan, A. (2002). "Seismic Site Characterization for Nuclear Structures and Power Plants", *Current Science, Vol. 87, No. 10*, 1388–1397.
8. Bowles, J. E. (2001), "Foundation Analysis and Design", Fifth Edition, McGraw-Hill, Inc.
9. Dowrick, D. J. (2003). "Earthquake Risk Reduction", John Wiley and Sons.
10. Kramer, S. L., (1996). "Geotechnical Earthquake Engineering", Pearson Education Pvt. Ltd.
11. Moseley, M.P, and Kirsch, K, (2004). "Ground Improvement", 2nd edition, Spon Press.
12. Port and Harbour Research Institute (1997), "Handbook on Liquefaction Remediation of Reclaimed Land", A. A. Balkema.

12

RETROFIT OF FOUNDATIONS

12.1 OVERVIEW

The loads from a building get transmitted to the soil through the foundation. A seismic retrofit of a building includes strengthening of inadequate foundations or supplementing with new foundation. This chapter covers the important aspects of deficiencies of foundation, analysis and assessment of foundation, the types of intervention to strengthen the foundation and the methods of execution. The types of intervention include strengthening rubble masonry foundation, enlarging the area of reinforced concrete footing, underpinning the foundation, drilling micro-piles, strengthening of piles and base plates. The descriptions are supported with schematic sketches for clarity.

The methods of execution briefly describe the types of shoring, temporary supports and underpinning. The methods of improvement of the ground are separately covered in the chapter on Mitigation of Geotechnical Seismic Hazards.

12.2 INTRODUCTION

All buildings have to be supported on adequate foundations. The foundations transmit the loads to the soil underneath. Hence, the safety of the buildings depends critically on the foundations. A builder has options for the material and construction of the building, such as masonry, timber, concrete or steel. However, in many cases, there is little choice with regard to the site of the building. The site may have unsatisfactory soil condition. In a few cases, the soil condition may be improved by ground improvement techniques. But in most cases, for economy, the prevailing soil condition is adopted with a suitable foundation.

> Soil has a wide range of characteristics depending on the nature of formation. The soil that forms at the location of the parent rock (residual soil) is different from that forming at a different location (transported soil). Even at a given site, the soils taken from two locations from the same stratum, can show widely varying properties. The properties are also affected considerably by environmental changes and vibrations during an earthquake. Therefore, it is important to investigate the site before choosing a suitable foundation for a new building or retrofitting the foundation for an existing building.

Some types of clay which are very hard when dry, lose their bearing capacity when wet. In India, vast areas have black cotton soil which contains expansive clay mineral. The later is responsible for excessive swelling and shrinkage. In the coastal areas, sandy soil with high water table is susceptible to liquefaction. The chapter on Mitigation of Geotechnical Seismic Hazards briefly covers site investigation, liquefaction and measures for ground improvement.

12.2.1 Types of Foundation

The foundations of structures can broadly be grouped as either shallow or deep. The essential features of these two types of foundation are discussed.

<u>Shallow Foundations</u>

- Column or wall footings and rafts are classified as shallow foundations. A shallow foundation transmits the loads from the structure to a soil stratum at a relatively small depth from the ground surface.
- The location and the depth at which a shallow foundation is placed should have adequate bearing capacity. The foundation should not be affected by volume changes of the soil caused by weathering, expulsion of soil from beneath or due to the foundations of adjoining structures.
- The settlement, especially any differential settlement, tends to cause damages to the structure. Hence, both the settlement and differential settlement should be within limits.

The above requirements of location, adequate bearing capacity and permissible settlement, should be satisfied individually for each footing.

Deep Foundations

- Piles and well foundations are classified as deep foundations. In piles, the load from the structure is supported by frictional resistance around the piles and/or by end bearing when the piles rest on rock or a hard stratum.
- Unlike a shallow foundation constructed after excavating the soil up to the foundation bed, in-situ construction of a deep foundation makes it impossible to visually inspect the quality.
- For buildings on expansive clay such as black cotton soil, under-reamed piles are most suitable. A pile with a bulb at the bottom is taken to a suitable depth where seasonal moisture variation is less.

12.3 DEFICIENCIES OF FOUNDATION

It is essential to diagnose the deficiencies in the foundation before undertaking retrofit. Also, the highlighting of deficiencies is expected to create awareness for future construction. The causes of deficiencies are listed.

1. The foundation may not have adequate strength as per the requirements of the current seismic code IS 1893: 2002. Based on the increased seismic forces, the forces on the foundation will increase. Very often, old buildings are found to have been designed for gravity loads only.
2. The foundation can deteriorate due to the ground water and/or soil containing aggressive chemicals. Industrial pollutants such as sulphates and chlorides are responsible for deterioration of concrete. In such situations, chemical analyses of the soil and water must be undertaken.
3. On many occasions, settlement of the soil causes damage to the building.

12.4 CONDITION ASSESSMENT OF FOUNDATION

In a retrofit project, the condition of the foundation can be a determining factor as to whether a building can be retrofitted. It is necessary to check the foundation for the following.

1. Adequacy of soil strata against resistance to liquefaction.
2. Stability of the area, particularly for buildings on retained earth or on hill slopes.
3. Maximum bearing pressure on the foundation including under the design earthquake.
4. Structural adequacy of the foundation elements such as footing slabs, pile caps, piles, etc.

The condition assessment of buildings is covered in Chapter 4. Here, some information is provided pertinent to foundations. The condition assessment includes some of the following depending upon the nature of building.

- Investigation of construction records and archives for soil conditions.
- Fresh soil investigation including sampling and testing. Measurements of groundwater level and pore water pressure.
- Survey of the foundation and foundation walls. If required, a few typical footings may be exposed. The survey examines any deterioration of the material.
- Report of settlement, formation of cracks and tilting of the walls and vertical/horizontal alignment of the foundation.

12.5 ANALYSIS

12.5.1 Analysis of the Building

In the analysis of a building, traditionally the soil-structure interaction is neglected. The bottom of the ground storey walls or columns are considered to be fixed or pinned depending upon the type of foundation. Recommendations on the modelling of the base conditions are given in Chapter 8, Structural Analysis for Seismic Retrofit. Once the forces at the base are obtained from the analysis, the foundation can be analysed separately.

> For a refined analysis for important buildings, the soil-structure interaction can be incorporated. The interaction refers to the effect of the soil deformation on the forces in the building.

The following approaches are usually adopted for modelling the soil-foundation system.

(a) Shallow foundations: The soil beneath the shallow foundations can be modelled using spring supports (referred to as Winkler's spring supports) with stiffness calculated based on the modulus of subgrade reaction (simply, subgrade modulus). Typical ranges of subgrade modulus are provided by Bowles (2001).

(b) Pile foundations: A pile group can be modelled as a column. The following concepts are used in the model.

- Depth of fixity: The piles are assumed to be fixed at certain depth below the pile cap. The depth of fixity depends on the stiffness of the soil (represented by the subgrade modulus) and the fixity condition of the piles at the pile cap. IS 2911 provides recommendation for the depth of fixity.
- Soil springs: Soil springs are assumed at the interface of each pile and the soil. The stiffness of a spring is derived form the subgrade modulus.

The forces on the foundation are subsequently obtained from the analysis of the building model.

12.5.2 Analysis of the Foundation

(a) Shallow foundations: The foundation may be considered to be either rigid or flexible depending on the relative stiffness of the foundation with respect to the soil. For a rigid foundation, the soil pressure is linear and the computations of the bending moment and shear force in the footing slab or raft are simpler. For a flexible foundation, the analysis is done using Winkler's spring supports. Although the analysis needs more effort, the bending moment is usually less compared to that obtained from the analysis based on rigid foundation.

(b) Pile foundations: In the case of a pile group, the forces on individual piles can be evaluated using the finite element method. In this method, a pile is modelled using beam elements and the pile cap is modelled as a thick plate. The forces from the column are applied on top of the pile cap. From the analysis, a pile is checked for the maximum compression force, uplift force, bending moment and shear force.

12.6 TYPES OF INTERVENTIONS

If the existing foundation does not meet the demand, retrofitting needs to be carried out. It is of course difficult to carry out the operation while the building is in service.

The following difficulties are to be envisaged during retrofitting of foundations.

1. Evacuation of the entire area or part of the area in phases.
2. Breaking the ground floor slab inside the building and paved areas outside the building.
3. Restricted movement during intervention due to temporary supports.
4. Restricted headroom for mobilizing large construction equipments such as piling rig.
5. Noise / vibrations in the building.

In spite of the above difficulties, retrofitting has to be carried out in view of the safety of the building for future use. To improve the inadequate bearing capacity, reduce excessive settlement or potential for liquefaction, the zone below the existing foundation can be improved by injection of grout or chemical into the ground. This is described in Chapter 11, Mitigation of Geotechnical Seismic Hazards. The following interventions are related with the foundation alone.

12.6.1 Strengthening Rubble Masonry Foundation

The method of underpinning can be used only if the masonry has sufficient strength for intervention. For underpinning, both sides of the wall are excavated. The material underneath the foundation is removed in segments (say, for about a length of 1.5 m). Reinforcing bars are introduced below the wall and concrete is cast. A layer of expansive mortar can be provided between the masonry and the new concrete (Figure 12.1).

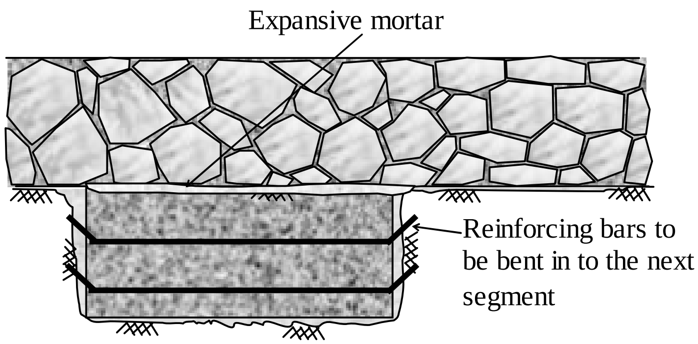


Figure 12.1 Underpinning of rubble masonry foundation

A masonry foundation can be strengthened by additional reinforcement and concrete (Figure 12.2). First, both sides of the wall are exposed by excavating the soil. Holes are drilled at intervals of 0.5 to 1.0 m. Reinforcing bars are placed in the holes. After cleaning all loose material, the surface of the wall is covered with concrete. Instead of conventional concrete, shotcrete can be applied. The bars must be protected from corrosion by a concrete cover of 40 mm. The wall can be strenghtened by grout injection as described in Chapters 5 and 6.

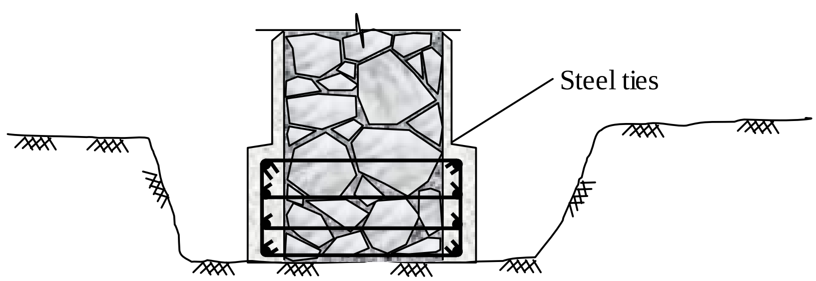


Figure 12.2 Strengthening of rubble masonry foundation by reinforcement

Another method of strengthening is to provide reinforced concrete (RC) beams on either side of the wall. The beams can be tied at regular intervals (Figure 12.3). If necessary, the beams can also be supported on piles.

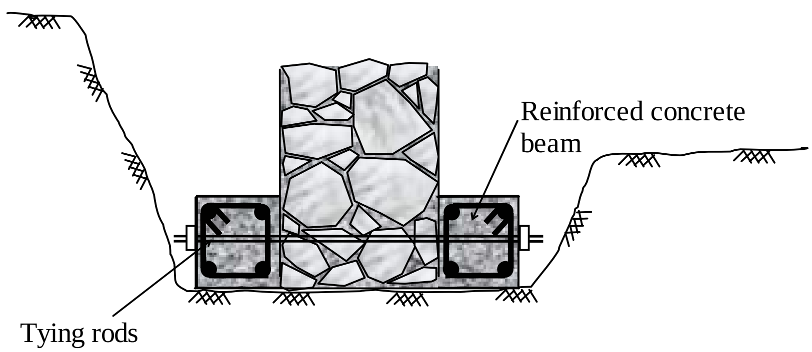


Figure 12.3 Strengthening of rubble masonry foundation by beams

12.6.2 Enlarging the Area of Footing

In some cases, the foundation can be strengthened by enlarging the size of the RC footing (Figure 12.4). The footing can be a wall footing or a column footing. The following procedure can be adopted.

i. Excavate around the foundation up to the bottom of the plain concrete (PC) levelling course over the required width.

ii. Expose the cover of the footing along the vertical face and top surface.

iii. Drill horizontally along the perimeter to attach dowel bars.

iv. Cast additional PC levelling course for the extended area of the footing.

v. Attach dowel bars with epoxy grout into the holes drilled.

vi. Apply a suitable polymer bonding agent over the exposed surface of the footing.

vii. Arrange the rebar for the additional area as per the design.

viii. Cast the additional concrete against formwork with proper compaction.

ix. Remove the formwork after initial set.

x. Cure the foundation by covering the concrete with wet gunny hags. Do not flood the area with water, as this may weaken the founding strata.

xi. Strengthen the column, if required, by external jacketing.

xii. Backfill the foundation with selected cohesionless soil in layers, well compacted up to the ground level.

xiii. Complete the plinth filling and flooring or paving.

Along with the footing, the column can be strengthened by concrete jacket. This is covered in Chapter 9, Retrofit of Reinforced Concrete Buildings.

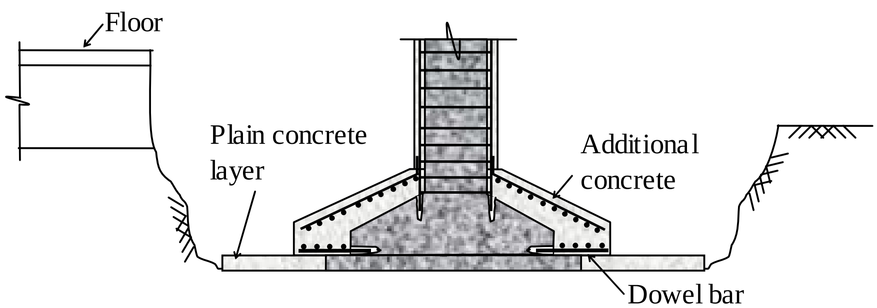


Figure 12.4 Increasing the area of footing

12.6.3 Drilling Micro-piles (Root Piles)

Micro-piles are small diameter (100 to 200 mm) piles that can be drilled through the existing foundations, either vertical or inclined. A schematic sketch of these piles is shown in Figure 12.5. The piles extend much beyond the foundation level and transfer the loads to the deeper level over a wider width. After a hole is drilled, it is grouted with cement slurry and reinforcement bars (rebar) are inserted before the initial setting of the grout.

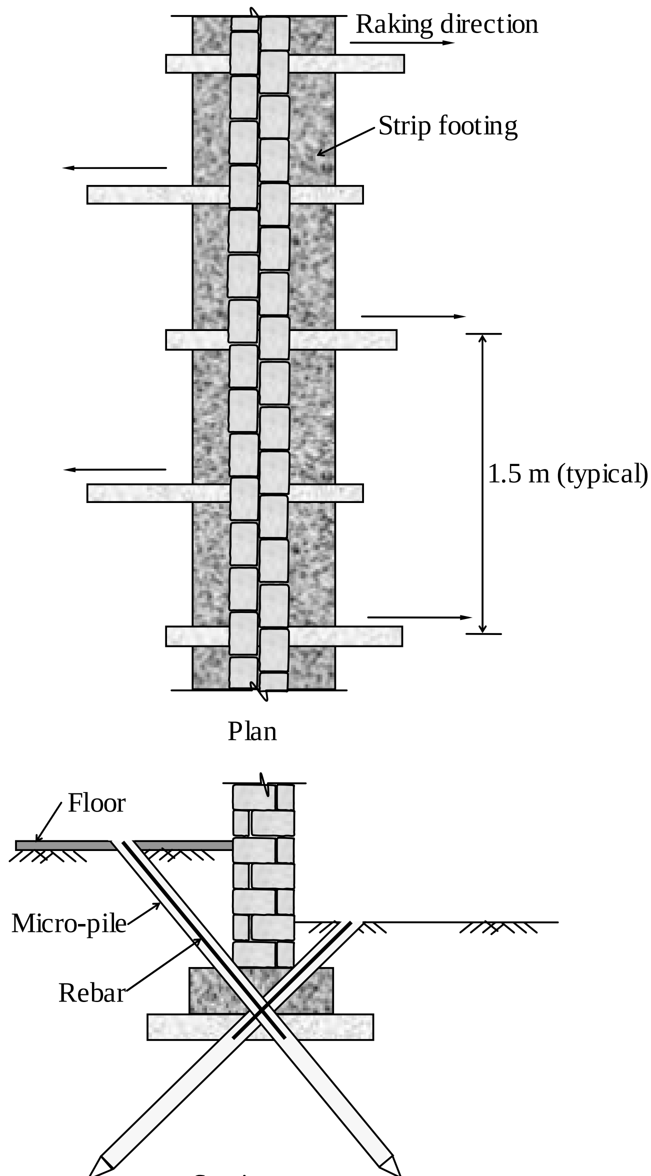


Figure 12.5 Strengthening by micro-piles

12.6.4 Underpinning with Piles

When the strata below the foundation are found to be very weak and increasing the area of a footing is not sufficient, it becomes necessary to underpin the foundation. This is achieved by transferring the load of the existing structure to a deeper level after installing additional piles. The piles are connected to the existing footing. The procedure is separately explained for foundations for masonry walls and RC columns.

(a) Continuous Strip Foundation under Masonry Walls

Figure 12.6 shows that the piles can be provided in pairs at intervals. If the wall is close to the property line, the pile cap can be provided form one side as shown in Figure 12.7. In this case, the external pile may be subjected to tension and hence, should be designed accordingly.

In absence of existing RC footing, it is necessary to introduce a longitudinal plinth beam to transfer the concentrated reactions from the pile caps uniformly over the length of the wall. Introduction of the plinth beam is complicated and needs careful planning. After temporarily shoring the roof, intermediate floor and wall, 50 percent of the width of the wall over the height of the designed plinth beam is removed in alternate stretches of about 3m length each. The plinth beam is cast in such a way that the stirrup can be extended when the other half of the wall is removed. A schematic sketch of the procedure is shown in the case study at the end of the chapter.

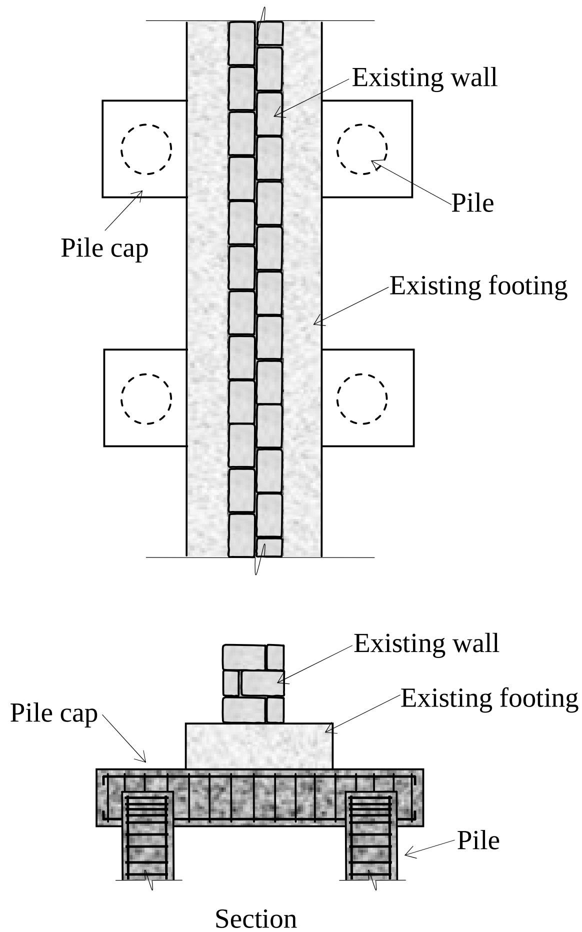


Figure 12.6 Arrangement of piles for strip foundation

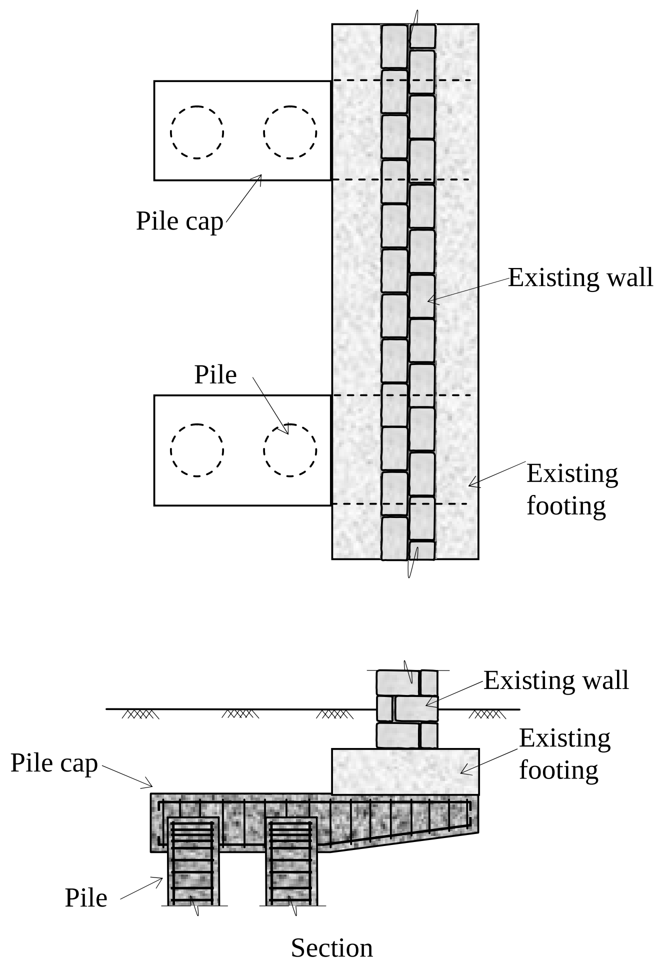


Figure 12.7 Alternative arrangement of piles for strip foundation

Depending on the rigidity of the plinth beam, needle beams are cast under the plinth beam with centre-to-centre spacing of 2 to 4 m. Each needle beam has a length equal to the width of the strip footing plus 1 m. The needle beams are supported on pile foundation. Based on the load, the piles can be small under-reamed piles. The following construction sequence can be adopted.

i. Prop the roof, intermediate floor and walls.

ii. Excavate the floor inside the building and the ground outside the building up to the bottom of the proposed plinth beam.

iii. Break the wall over the required height of the plinth beam and for half the thickness of the wall.

iv. Introduce the reinforcement cage.

v. Cast half of the plinth beam, preferably with self-compacting concrete. Use steel formwork (preferably) along the plinth wall and compact the concrete adequately.

vi. Within 3 to 7 days, remove the other half of the wall. Extend the reinforcement from the previous cage, provide additional longitudinal reinforcement as required, and cast the remaining half of the plinth beam.

vii. At the selected locations for needle beams, excavate both inside and outside of the building to the required dimensions. The depth of the trench should be up to bottom of the needle beam plus 50 mm for the PC levelling course.

viii. Construct the pile foundation for each end of the needle beam. The cut-off level of these piles should be 50mm above the PC level.

ix. Break the wall directly below the plinth beam for the required cross-sectional area of the needle beam.

x. After completion of the piles, provide PC over the required area for receiving the needle beams.

xi. Arrange the reinforcement of the needle beams integrating with the reinforcement of the piles.

xii. Provide formwork for the vertical faces of the needle beams.

xiii. Cast the needle beams to a level such that top surfaces are in contact with the bottom of the plinth beam.

xiv. Cure the needle beams and plinth beams.

xv. Backfill the excavation on both sides of the wall with well compacted cohesionless soil.

xvi. Complete the flooring and paving.

(b) Foundation for Reinforced Concrete Building

In the case of a reinforced concrete framed structure supported on individual footings, the number and size of additional piles required under a column is calculated based on the analysis of the structure. The additional piles are symmetrically placed around the edge of the existing foundation to ensure effective transfer of load. After construction of the piles, the size of the footing is increased both in plan and in thickness with additional reinforcement. Dowel bars are provided between the existing and new concrete. A schematic sketch of the arrangement is shown in Figure 12.8.

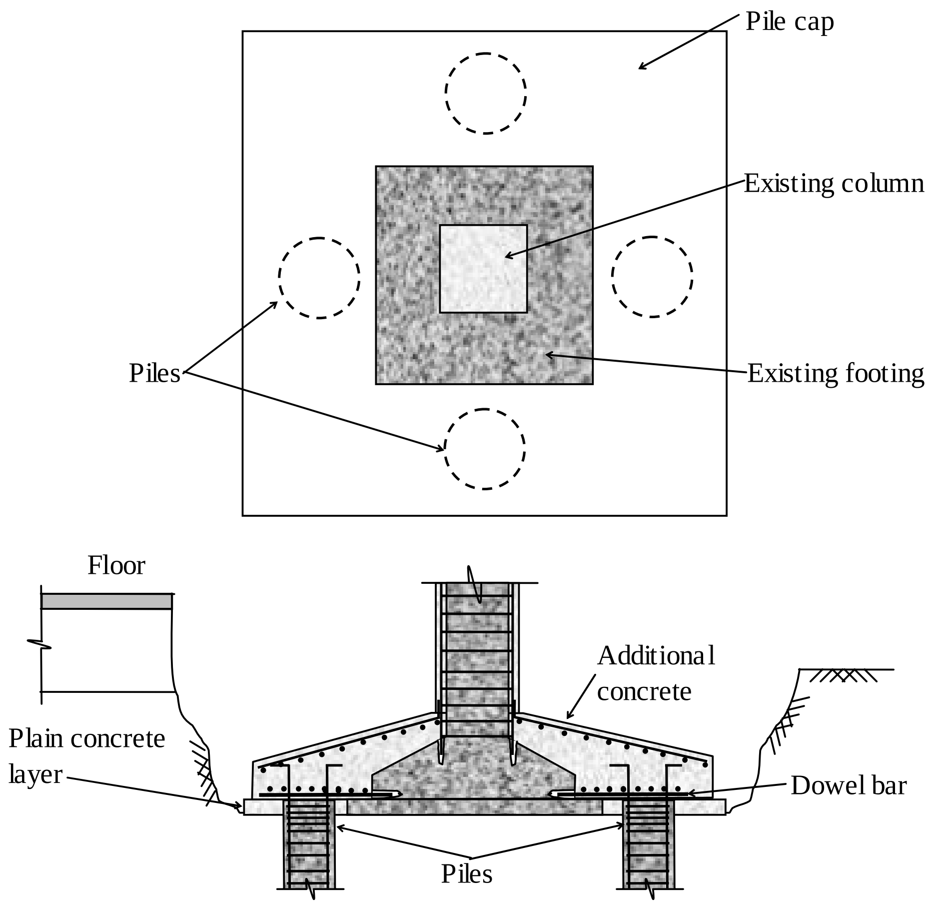


Figure 12.8 Underpinning of reinforced concrete footing with piles

The following steps can to be adopted for construction.

i. Prop the portion of the building around the column.

ii. Expose the existing footing or pile cap up to the PC level. The excavated area should be adequate to accommodate the additional piles and the pile cap.

iii. Chip-off the cover of the footing/pile cap along the vertical face and top surface.

iv. Drill the holes through the vertical face to provide necessary dowel bars.

v. Construct piles at required locations. The cut-off level of the piles should be 50mm above the PC level.

vi. Extend the PC course over the required area of the additional pile cap.

vii. Insert dowel bars in the holes drilled and bond them with epoxy.

viii. Provide additional reinforcement for the footing as required.

ix. Erect formwork.

x. Cast the concrete and compact the same.

xi. Remove the formwork after 24 hours and cure the concrete surface.

xii. Backfill the foundation with well compacted cohesionless soils.

xiii. Complete the flooring.

12.6.5 Strengthening of Piles

For buildings on piles, the piles may be inadequate to resist the design lateral loads calculated as per the current seismic code. Also, the piles may be deficient just below the pile-cap, where deterioration and damage to piles are common. The strengthening of piles can be as follows.

i. Dig down in segments to expose the piles to a level where the defects are visible (Figure 12.9 a).

ii. Expose the cover to an extent such that the damages and corrosion are removed. Provide a base cap at the bottom level to ensure continuity of the additional reinforcement for the piles (Figure 12.9 b).

iii. Provide additional reinforcement and concrete jacket around the piles based on the forces from the analysis of the building (Figure 12.9c).

iv. Backfill the foundation with well compacted cohesionless soils (Figure 12.9d).

v. Complete the flooring.

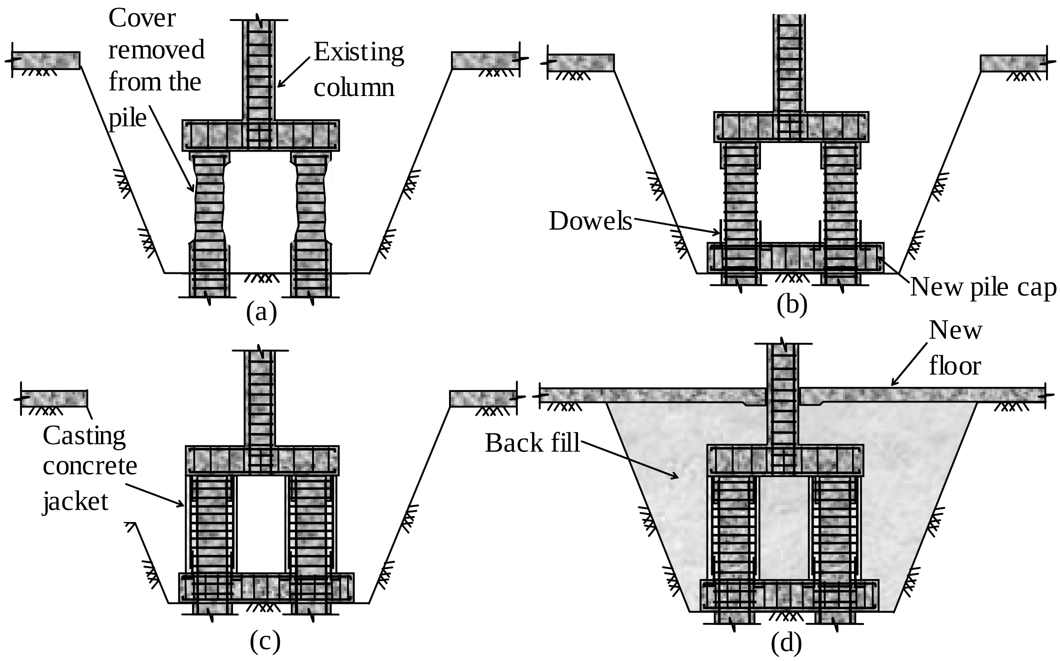


Figure 12.9 Strengthening of piles

12.6.6 Strengthening of Base Plates

Base plates and anchor bolts designed for gravity loads can fail under tension due to lateral loads (Figure 12.10). The strengthening involves increase in the size of the base plate and anchoring it to the new foundation (Figure 12.11). The new foundation can consist of piles and a pile cap. If there is an existing pile foundation, it can be strengthened as described in the earlier section.

Additional anchor bolts are installed in the new concrete. These anchor bolts are bolted to strap-ons to keep the old and new plates together. Since the lever arm between the new bolts is more, the lateral load resistance of the base plate significantly improves. Figure 12.12 shows a retrofitted tower foundation with a strap-on.

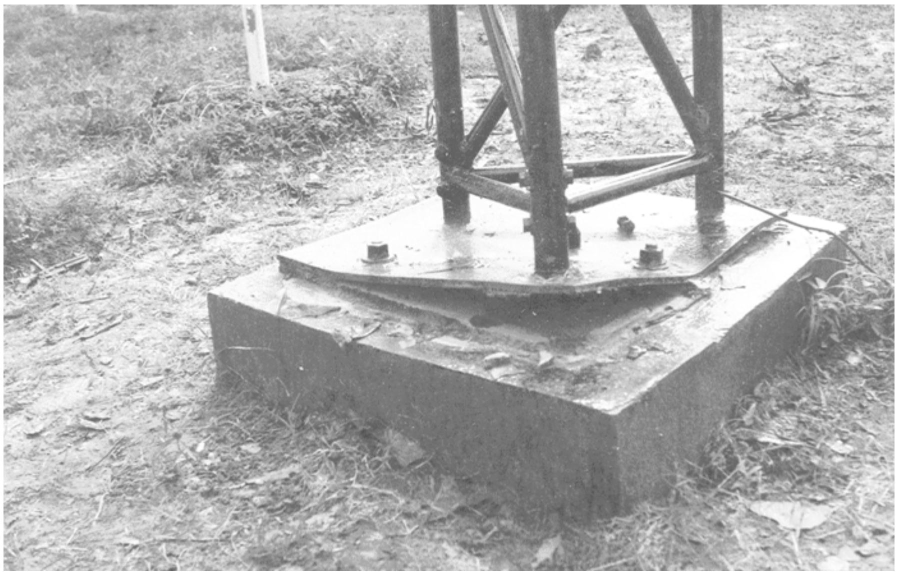


Figure 12.10 Damage in a base plate

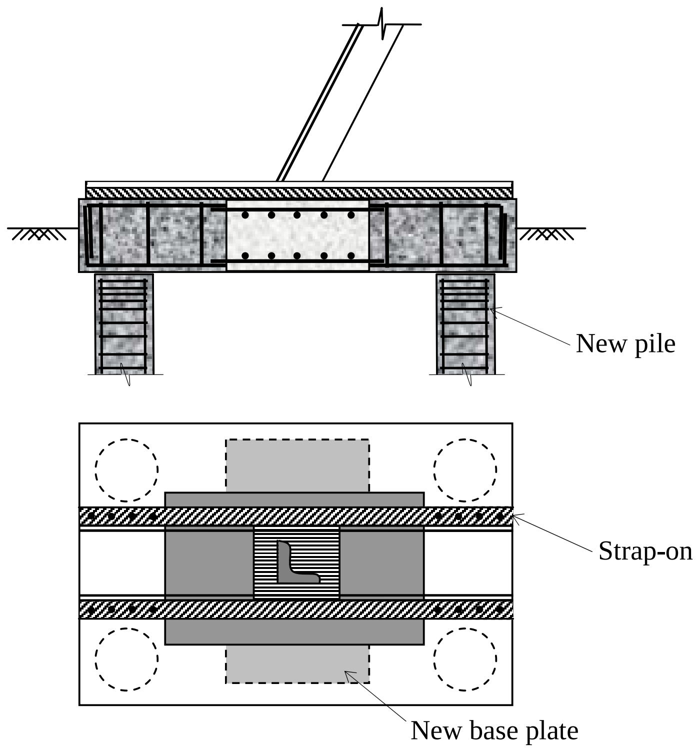


Figure 12.11 Anchoring base plate on to new pile cap

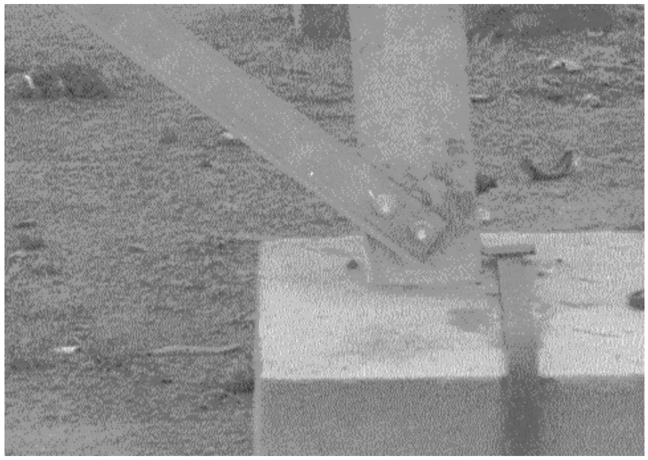


Figure 12.12 Retrofitted tower foundation with strap-on anchor

12.7 METHODS OF EXECUTION

Before any underpinning operation, the foundation should be preferably relieved of the load. Shoring is employed to temporarily support the structure against possible settlement or even collapse.

> Both shoring and underpinning require specialised expertise. Hence, these should be undertaken by firms with knowledgeable and experienced persons. A great deal of shoring work can be minimised by carefully studying the inherent strength and redistribution capabilities of the structure. Since a reinforced concrete framed building can redistribute the vertical loads, they do not need extensive shoring. On the other hand, buildings with load bearing walls need extensive shoring.

When the load bearing walls are in good condition, they can be secured by temporary horizontal ties and vertical braces. The ties are generally installed at all the floor levels including the roof.

12.7.1 Shoring

The shoring depends on the specific project and type of underpinning envisaged. However, a few common techniques of shoring are discussed below.

(a) Raking Shore

Raking shores are used to support external walls from going out-of-plumb during the underpinning operation. A typical raking shore for a three-storey building is shown in Figure 12.13. It should be ensured that the soil underneath the timber supporting system does not settle. Otherwise a larger area for the timber base should be considered.

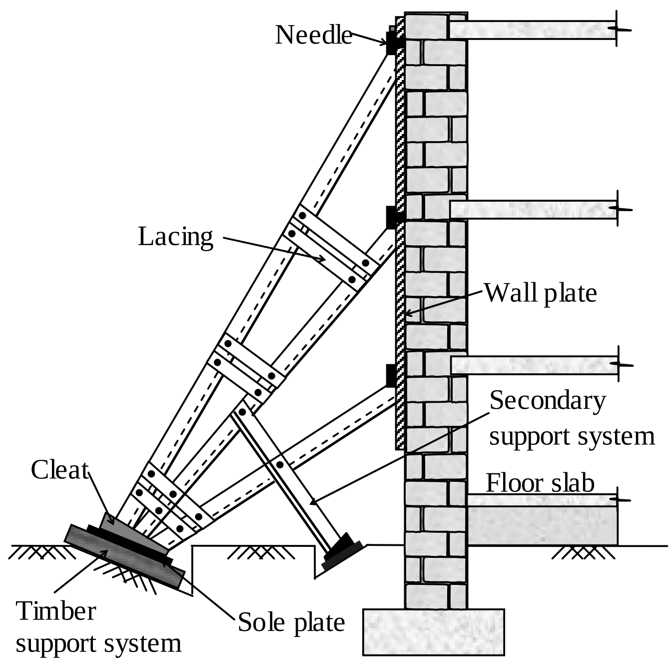


Figure 12.13 Typical arrangement for raking shores

(b) Flying Shore

The wall of one building can be supported by the wall of the adjacent building by providing 'flying' shores (Figure 12.14). The distance between the two buildings should be small for effective shoring. Flying shores cannot support the weight of the wall. However, they prevent bulging and out-of-plane movement of the wall.

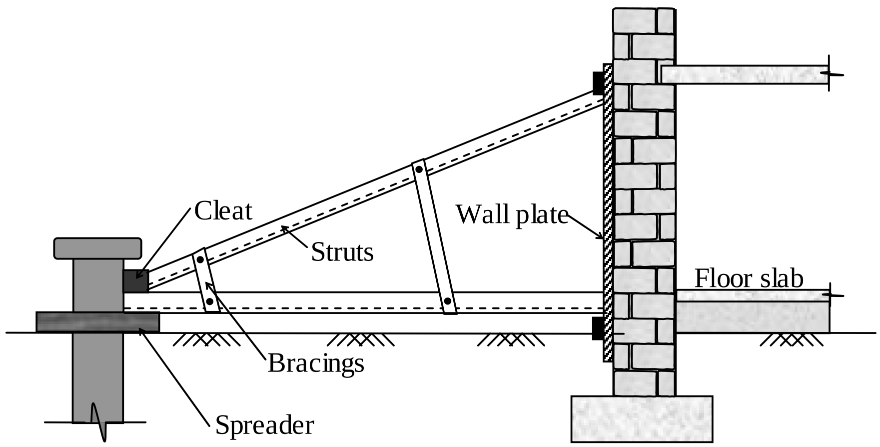


Figure 12.14 Typical arrangement for flying shores

(c) Dead Shores

Dead shores are vertical struts that relieve the load on the foundation by providing alternate load path to the ground. Dead shores enable to clear the area for underpinning operations. The needle beams are passed through holes cut in the wall. They transfer the superimposed load to temporary foundations (Figure 12.15).

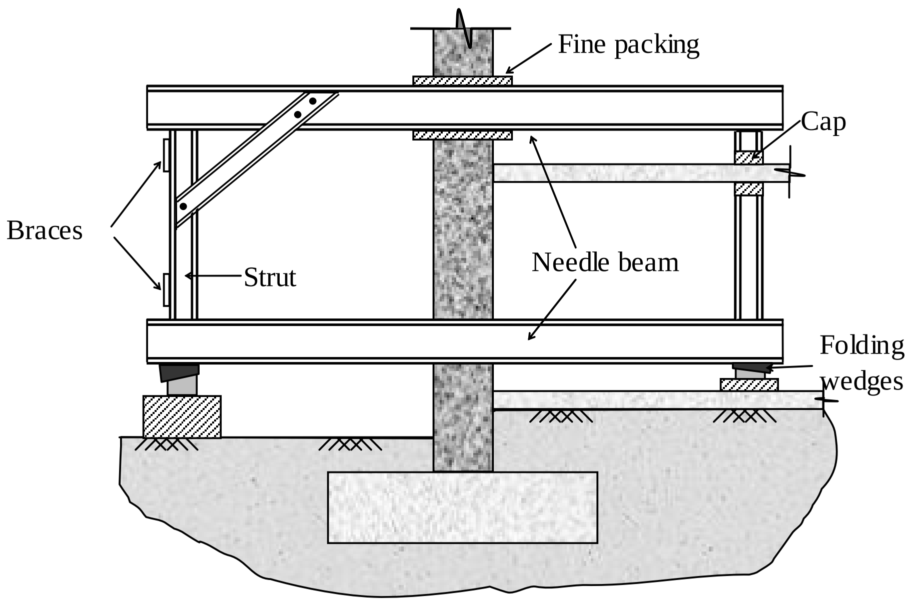


Figure 12.15 Typical arrangement for dead shores

12.7.2 Other Supporting Schemes

A column in a framed building can be supported by needle beams (Figures 12.16 a and b). The loads can be transferred to the needle beams through a steel collar by wedge action or channel embedded in the column. The needle beams are supported away from the area to be excavated.

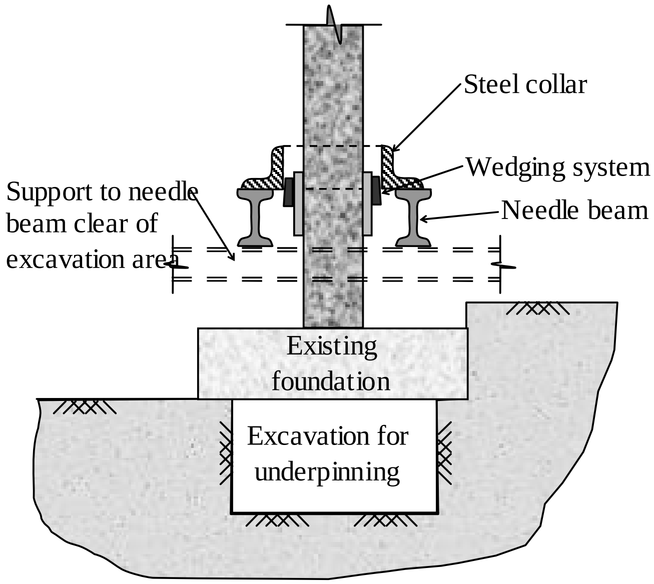


Figure 12.16a Supporting a column by collar and needle beams

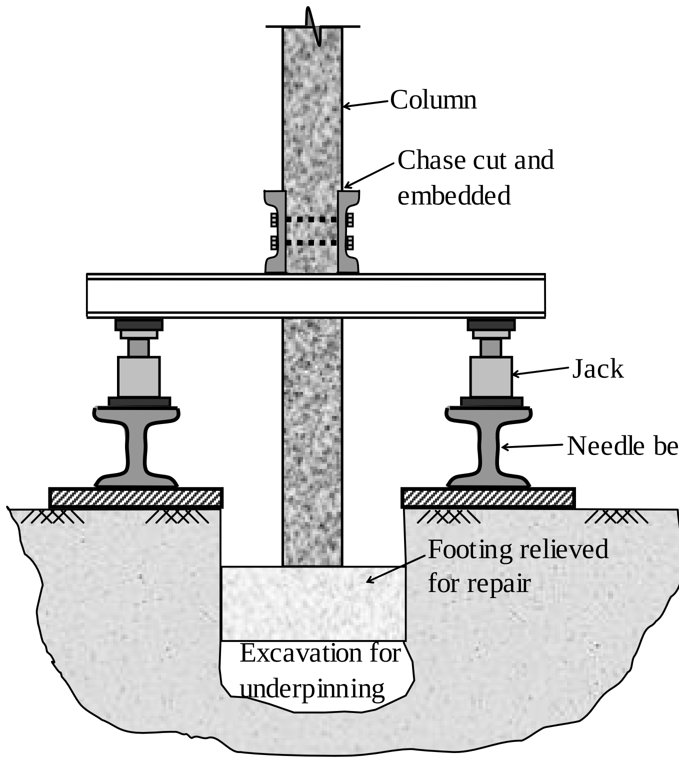


Figure 12.16b Supporting a column by needle beams

Instead of the above schemes, the first floor beams can be supported on all the four sides and the column can be left hanging as shown in Figure 12.17. Once the column load is removed, the excavation for underpinning can commence.

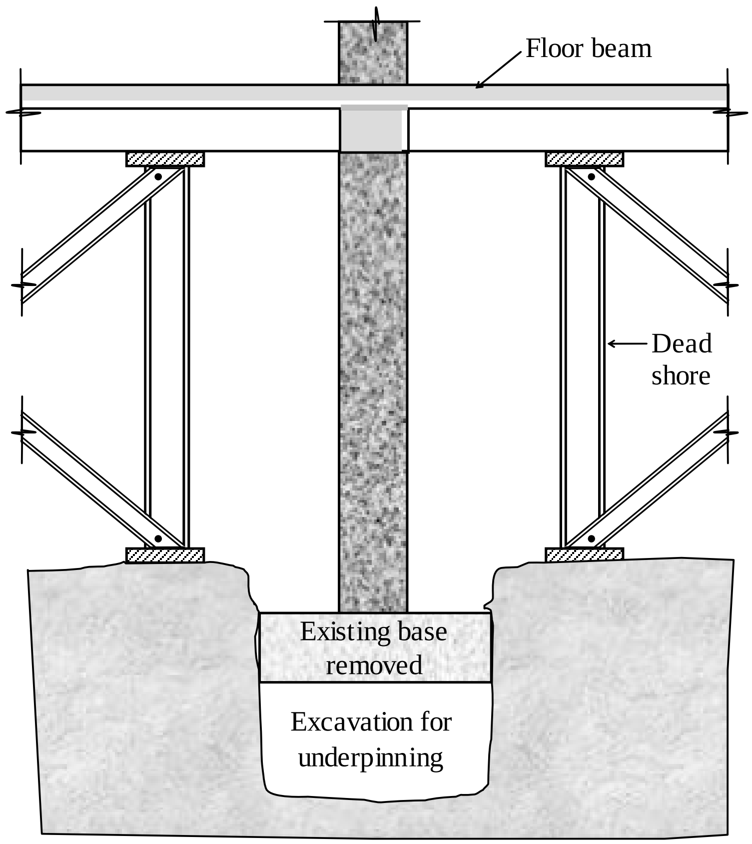


Figure 12.17 Supporting floor beams by dead shores

12.7.3 Methods of Underpinning

Once underpinning is selected as the retrofit strategy, the method of underpinning and temporary support should be worked out. The methods of underpinning are described here.

a) Underpinning Continuous Strip Foundation

Generally, a masonry wall can be unsupported over a length of 1 to 1.5 m (Figure 12.18). Figure 12.19 shows the sequence of the pits for underpinning. The segments are numbered 1 to 6 based on the sequence. The work can simultaneously proceed on segments having the same number. After the excavation, concrete is placed in the pit up to a depth of 50 to100 mm from the underside of the existing foundation. Any gap between the new concrete and the existing foundation can be filled with expanding mortar. Horizontal reinforcing bars of short lengths can be provided longitudinally between the segments. Polymeric agents, provided between adjacent segments, improve the bond. After the last pit is completed, it is worthwhile to undertake pressure grouting to fill up any void in between the segments.

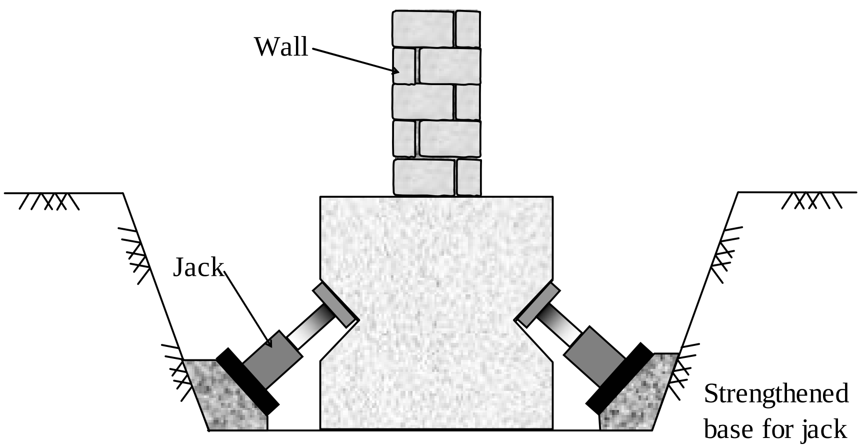


Figure 12.18 Method of supporting the wall

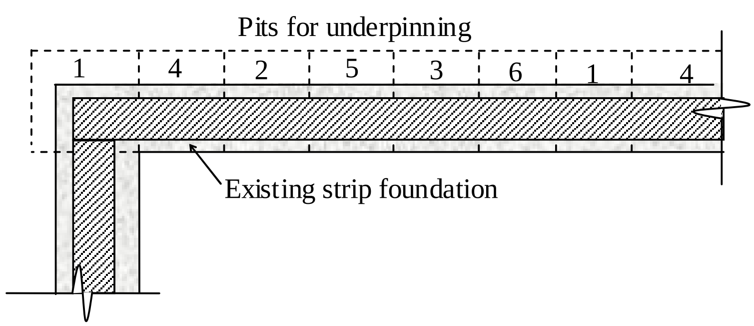


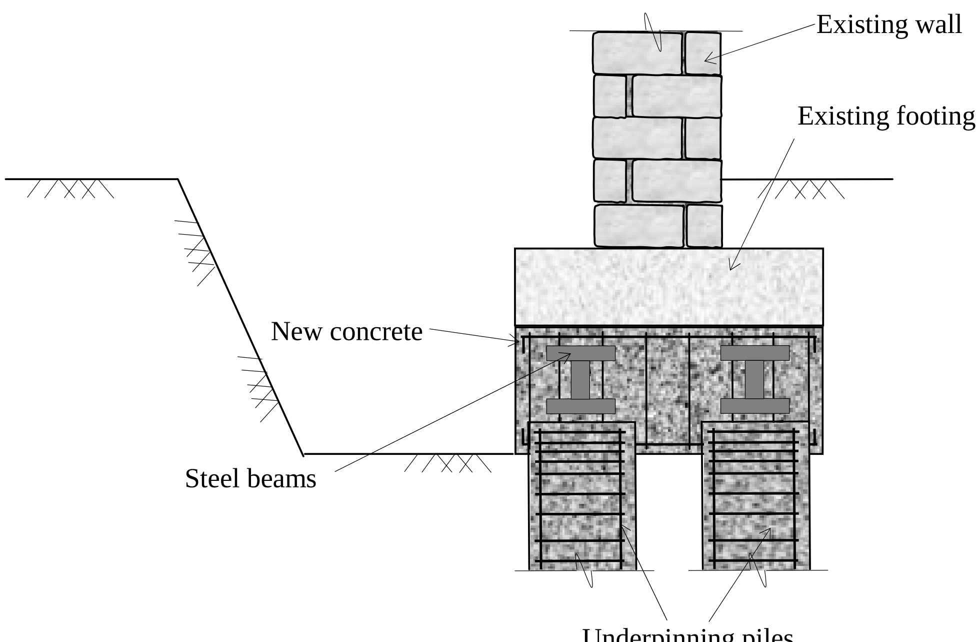
Figure 12.19 Sequence of pits for underpinning strip foundation

(b) Underpinning with Piers or Pile Groups

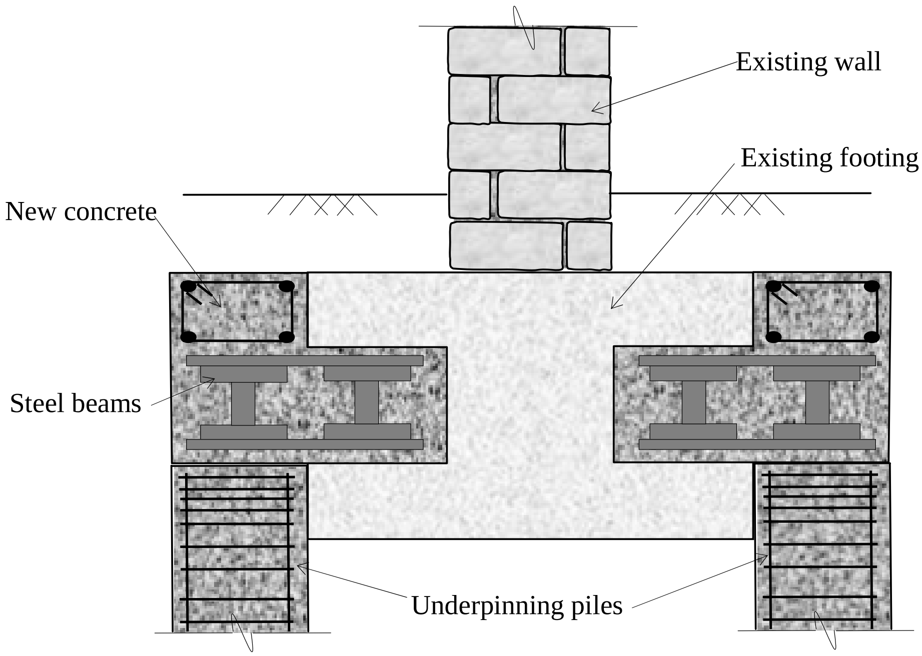
Pier or pile group foundation (including well foundation) may be a desirable method of underpinning at deep excavation sites. First, the load on the existing foundation is relieved by inserting needle beams as explained in the previous section. Next, additional beams are provided to transfer the load from the existing foundation to the piers or pile groups. The beams can be inserted in one of the following ways.

1. Support the wall for the full length between the piers (Figure 12.20a).
2. Insert the beams into openings made in the foundation and supporting them on piers (Figure 12.20b).
3. Insert precast concrete blocks into the openings made in the foundation and make it continuous between the piers (Figure 12.20c).

After connecting the piers to the inserted beams, the needle beams can be carefully removed to transfer the load to the piers.



(a)


Existing wall
Existing footing
New concrete
Steel beams
Underpinning piles

(b)

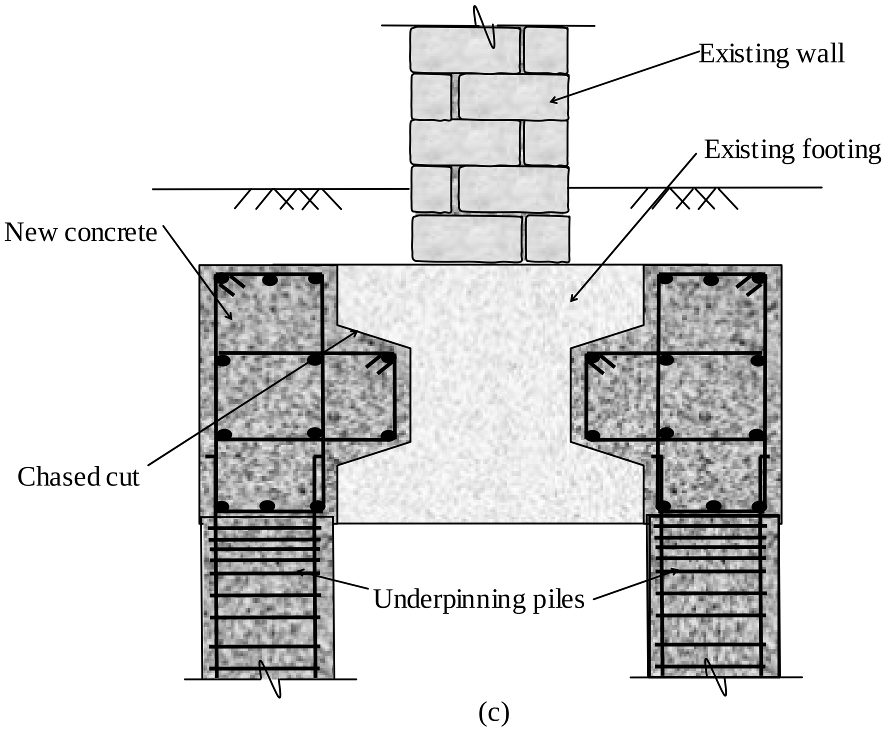


Figure 12.18 Underpinning with pier / piles and beams

12.8 A CASE STUDY

A laboratory building of a Polytechnic College had undergone excessive settlement, with tilt and wide cracks in the walls due to inadequacy of the foundation. The structure was considered to be unsafe and alternative arrangement was made to run the laboratory classes. The site had 4 to 6m thick filled-up soil, which was of very low shear strength and high compressibility. The foundation was strengthened by introducing a new plinth beam and under-reamed piles. After strengthening, the building has been in operation without any sign of further settlement or cracks. Figure 12.19 shows elevations of the existing and strengthened foundations. Figure 12.20 shows the sequence of underpinning.

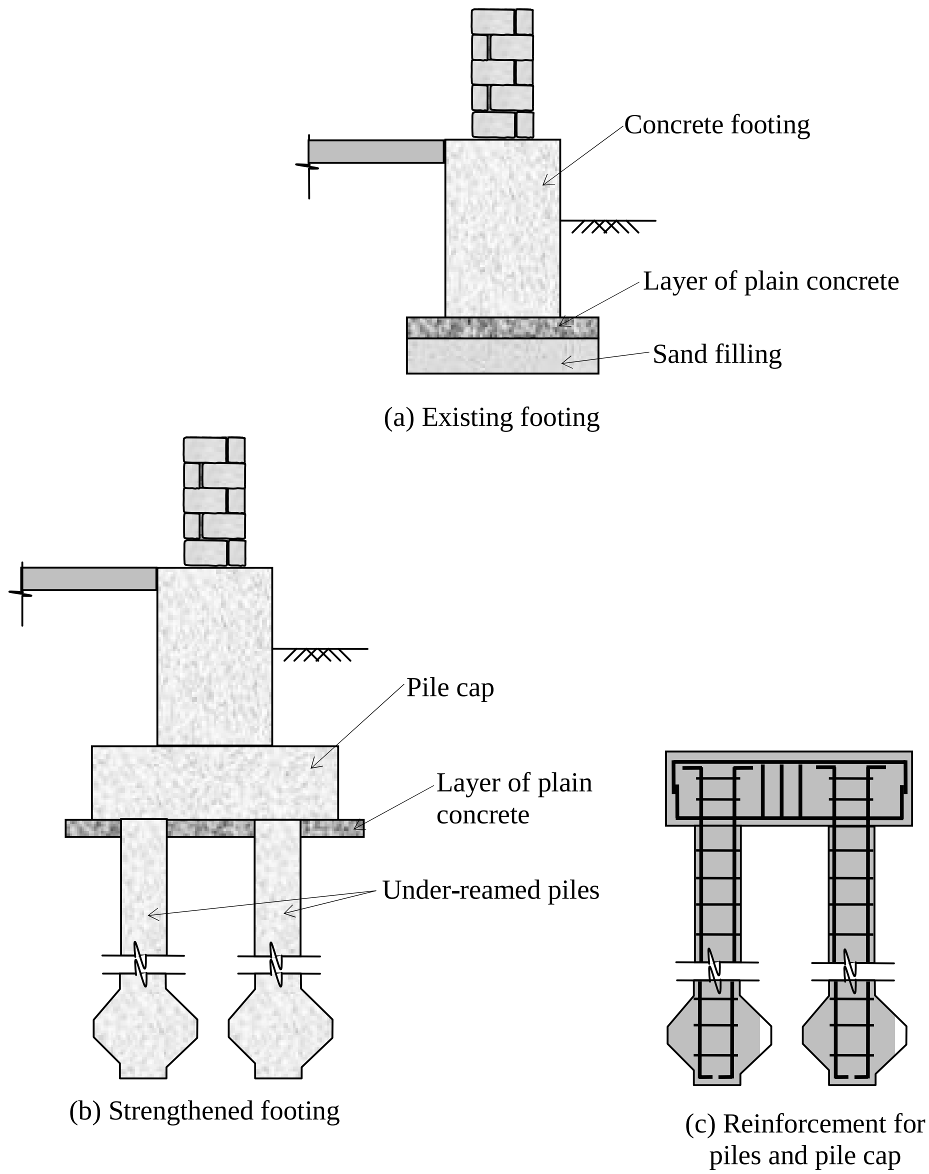
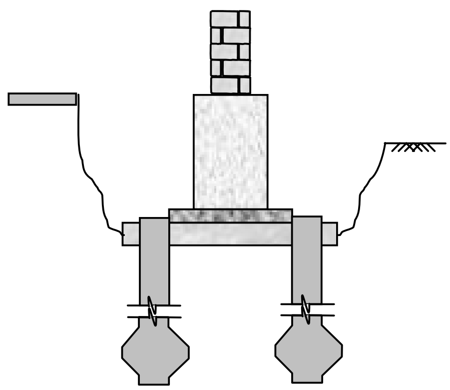
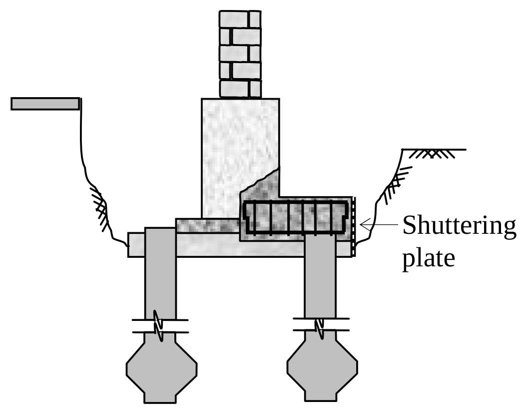


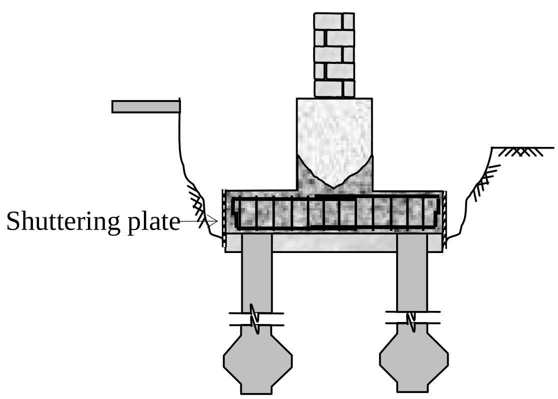
Figure 12.19 Foundation before and after underpinning



(a) Stage I: Pile casted



(b) Stage II: Half beam casted



(c) Stage III: Full beam casted

Figure 12.20 Sequence of underpinning operation

12.9 SUMMARY

This chapter covers the important aspects of deficiencies of foundation, analysis and assessment of foundation, the types of intervention to strengthen the foundation and the methods of execution. The types of intervention include strengthening rubble masonry foundation, enlarging the area of reinforced concrete footing, underpinning the foundation, drilling micro-piles, strengthening of piles and base plates. The methods of execution briefly describe the types of shoring, temporary supports and underpinning.

12.10 REFERENCES

1. IS 1893: 2002, “Criteria for Earthquake Resistant Design of Structures”, Bureau of Indian Standards.
2. IS 2911, “Indian Standard Code of Practice for Design and Construction of Pile Foundations”, Parts 1 to 4, Bureau of Indian Standards.
3. Bowles, J.E. (2001), “Foundation Analysis and Design”, Fifth Edition, McGraw-Hill, Inc.
4. Tomlinson, M.J. (1981), “Foundation Design and Construction”, Pitman Books Ltd. London.
5. Santhakumar, A.R. (2006), “Concrete Technology”, Oxford IBH Publication, New Delhi.

13

RETROFIT USING FIBRE REINFORCED POLYMER COMPOSITES

13.1 OVERVIEW

Fibre reinforced polymer (FRP) is a composite material consisting of polymeric resin reinforced with high strength fibres. The composite material is available in the form of sheets (or fabrics), pre-formed shapes and bars. In this chapter, the application of sheets is highlighted. The FRP sheets are thin, light and flexible enough to be inserted behind pipes, electrical and mechanical ducts, thus facilitating installation. The chapter highlights the advantages of using FRP, the constituent materials, the types of FRP composites and the technique of bonding the laminates on a substrate.

For strengthening of masonry walls, sketches of the configurations of the FRP laminates are provided. The procedures for analysis of a retrofitted wall for out-of-plane bending and in-plane shear are elucidated. The use of FRP in retrofitting reinforced concrete beams is explained under strengthening for flexure and shear. The use of FRP in increasing the shear strength and confinement of concrete in a column are briefly covered. Equations relevant in the analysis of a retrofitted section are provided.

13.2 INTRODUCTION

The fibre reinforced polymer (FRP) composites are useful for repair, rehabilitation and retrofit of structures for the following reasons.

- The FRP sheets are light and flexible, which facilitates installation. It does not need drilling of concrete or masonry. There is less disruption during strengthening.
- The curing time is less. This leads to reduced down time to the users of a building.
- The sheets are thin and hence there is marginal increase in the size of a retrofitted member.
- The sheets have high strength-to-weight ratio and superior creep properties.
- The material is chemically inert and has resistance against electro-chemical corrosion.
- There is good fatigue strength, which is suitable for fluctuating loads.

The fibre reinforced polymer (FRP) composites are used in the repair of buildings, water tanks, bridges, marine structures etc. This chapter covers the use of FRP in the repair and retrofit of masonry and reinforced concrete buildings.

Even though the materials used in FRP are relatively expensive as compared to the traditional strengthening materials such as concrete and steel, the labour, equipment and construction costs are often lower. Strengthening by FRP can be used in areas with limited access, where traditional techniques such as concrete and steel jacketing would be impractical.

13.2.1 Constituent Materials

FRP consists of primarily two components, polymeric resin and high strength non-metallic fibres. The resin acts as the matrix (the bulk component) and the fibres as the reinforcement. There can also be additives and fillers. Resins such as epoxy and polyesters have been used in different techniques of repair and retrofit. The availability of high strength resins has increased the use of FRP. The resins must have the following characteristics.

1. Compatibility and adhesion with the substrate, such as concrete or masonry
2. Compatibility and viscous enough for adhesion with the fibres
3. Not too viscous to retain the filling ability so that voids are not present
4. Development of appropriate mechanical properties for the composite.
5. Resistance to environmental effects, like moisture, salt water, temperature and other chemicals normally associated with corrosion

Typical values of the properties of epoxy resin are provided in the following table. However, a particular resin has to be tested before application.

Table 13.1 Typical values of the properties of epoxy resin

Specific gravity	1.2 to 1.3
Tensile strength	55 to 130 N/mm^2
Tensile modulus of elasticity	2800 to 4200 N/mm^2
Flexural strength	125 N/mm^2
Flexural modulus	2000 to 4200 N/mm^2
Impact strength	0.1 to 1.0 J/m of notch
Poisson's ratio	0.2 to 0.33

The fibres can be of glass, carbon or aramid. Among them, glass and carbon fibres are common. The glass and carbon fibres are made into fabric form like unidirectional cloth (Figure 13.1) and woven roving mat (Figure 13.2). Glass fibres have lower stiffness and cost as compared to carbon fibres. They are suitable in low cost seismic retrofit applications. Typical values of the properties of glass fibre are given in Table 13.2.

Table 13.2 Typical values of the properties of glass fibre

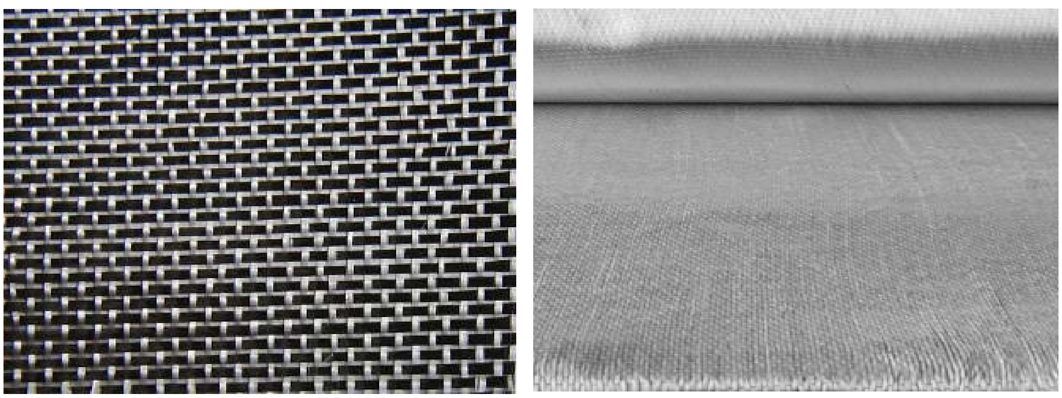
Density	0.9 kg/m^3
Tensile strength	1700 N/mm^2
Tensile modulus of elasticity	75,000 N/mm^2

The fibres in the sheet or fabric can be oriented as follows.

i. Unidirectional fibre sheets, where the fibres run predominantly in one direction

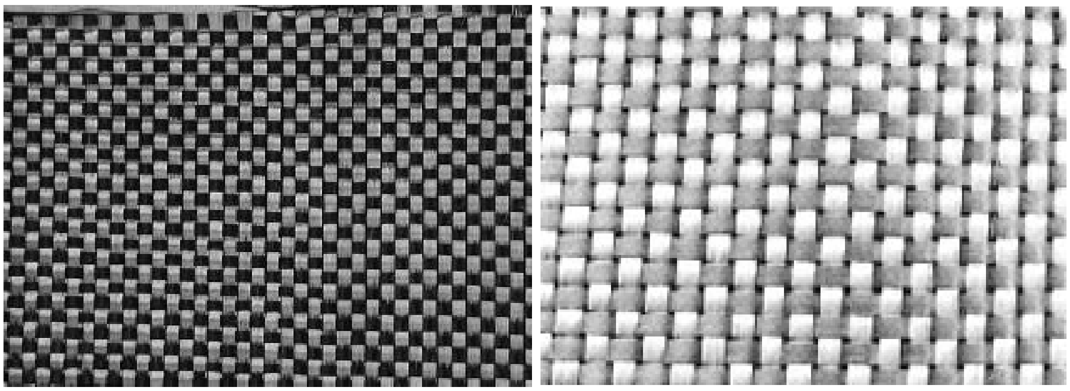
ii. Multidirectional fibre sheets or fabrics, where the fibres are oriented in at least two directions.

The final composites are elastic up to failure and do not exhibit plasticity. The ultimate strain is high which is desirable in seismic retrofit applications. The axial stress versus strain curves of concrete cylinders wrapped with several layers of FRP laminates show increased strength and ductility due to confinement (Mukherjee and Joshi, 2002).



(a) Carbon fibres (b) Glass fibres

Figure 13.1 Unidirectional cloth



(a) Carbon fibres (b) Glass fibres

Figure 13.2 Woven roving mat

13.2.2 Types of FRP Composites based on Installation

The FRP composites can be categorized based on how they are delivered to the site and installed.

Dry lay-up FRP composites

In this method, first the substrate is coated with resin. Next, the dry FRP sheets are bonded on the substrate and a fresh coating of resin may be applied. The composite is subsequently cured.

Wet lay-up FRP composites

First, the FRP sheets are saturated with resin and then bonded on the substrate. It is then cured as per the specifications.

Pre-impregnated FRP composites

These consist of fibre sheets that are pre-impregnated (pre-preg) with a saturating resin off-site in the supplier's facility. The pre-preg composites are bonded to the substrate with or without additional resin application, depending upon the specific system requirements. The fibre tow varieties are wound or mechanically applied on the surface. The composites are subsequently cured. The pre-preg composites are widely used in aero-space, automobile and ship building applications.

Pre-cured FRP composites

These consist of a wide variety of shapes manufactured off-site in the supplier's facility and shipped to the site. They are bonded to the substrate with resin. Pultruded and pre-cured FRP sheets are frequently used for repair and retrofit. The pultruded sheets are manufactured by the pultrusion process. In this process, the fibres are pulled through a bath of resin and passed through pre-forming fixtures.

13.2.3 Selection of FRP Composites

For a particular project, the suitability of using FRP composites for seismic retrofit should be assessed prior to selecting the product. To assess the suitability of FRP, the designer should evaluate the existing building for lateral strength, identify the deficiencies and their causes, and determine the condition of the substrate. A thorough field investigation of the dimensions of the members, compressive strength and soundness of the concrete or masonry, location, size and cause of cracks, spalling of plaster or concrete cover, location and extent of corrosion of reinforcing bars etc., should be undertaken. The topics are covered under preliminary evaluation, condition assessment and seismic analysis in Chapters 3, 4 and 8, respectively.

13.2.4 Installation of FRP Composites

The concrete or masonry surfaces to which the FRP laminates are to be applied should be freshly exposed and loose unsound materials have to be removed. Surface preparation can be done using sand or water blasting techniques. All dust, dirt, oil, curing compound, existing coatings and any other matter that could interfere with the bond of the FRP composites to the substrate should be removed. Bug holes and other small surface voids should be completely filled with putty materials. The surface of application should be smooth and convex.

The following points are important.

- Concavity in the surface should be eliminated. A stretched FRP sheet will lose contact with the surface around depressions.
- Sharp edges and corners should be avoided. The corners should be rounded to a minimum radius of 25 mm. The sheets are susceptible to tearing at these locations due to stress concentration.
- Since the installation procedure depends on the product, the instructions attached with the product should be followed carefully.

The typical lay-up of an FRP composite on the substrate is shown in Figure 13.3.

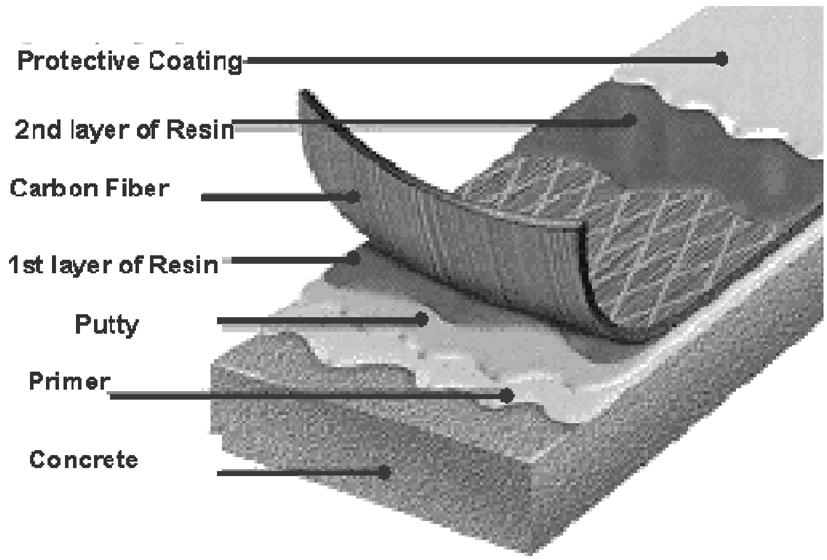
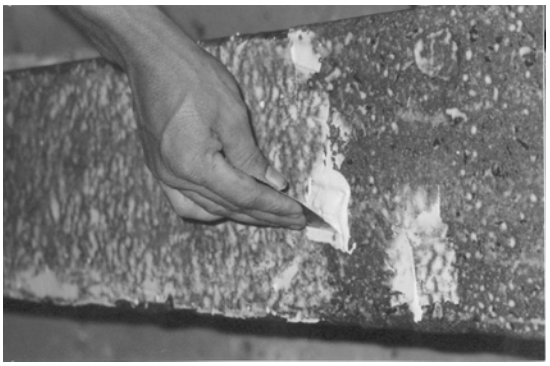


Figure 13.3 Typical lay-up of FRP composite (extruded view)

The FRP systems are commonly installed using dry fibre fabrics/sheets and a saturating resin. The saturating resin is applied uniformly to the whole prepared surface. The fibres should be gently pressed into the uncured saturating resin in a manner recommended by the composite manufacturer. Entrapped air between layers should be released or rolled out before the resin sets. Sufficient resin should be applied to achieve full saturation of the fibres. Successive layers of resin should be placed before the complete curing of the previous layer. The sequence of steps for installation of FRP sheets on a concrete substrate are shown in Figures 13.4 (a) to (d). After the resin is completely cured, a top coating is provided.

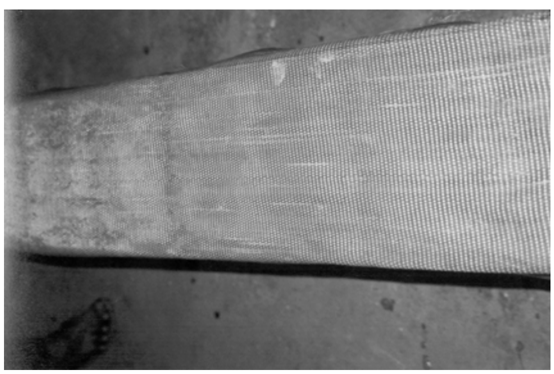
Machine applied systems utilize pre-impregnated (pre-preg) tows or dry fibre tows. The pre-preg tows are impregnated with saturating resin off site and delivered to the work site as spools. The dry fibres are impregnated at the job site during the winding process. The wrapping machines are used for automated wrapping on existing concrete columns.

Pre-cured systems like shells of FRP composites, strips, and open grid forms are typically installed with an adhesive. The adhesive should be applied uniformly to the prepared surface and at a rate recommended by the FRP manufacturer to ensure full bonding of the laminate. The laminate surface should be cleaned and prepared in accordance with the manufacturer's recommendation.

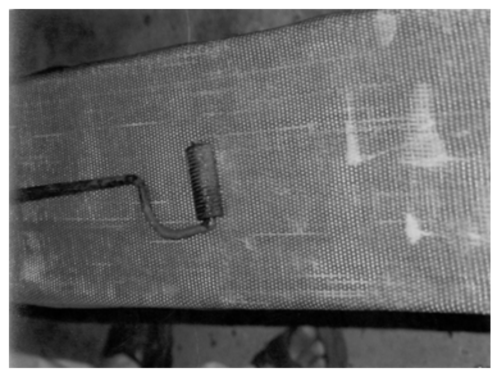


(a) Primer and putty application

(b) Levelling with putty



(c) Laying of FRP sheet on surface coated with resin



(d) Removal of entrapped air

Figure 13.4 Installation of FRP sheet on concrete substrate

13.3 STRENGTHENING OF MASONRY WALLS

The failure of masonry load bearing walls and the infill walls in framed buildings during earthquakes is due to the low tensile and shear resistance of the walls. For masonry load bearing walls, the modes of failure under in-plane forces are sliding shear cracking along a bed joint, diagonal cracking through several bed joints and flexural crushing (for tall walls). The walls can also fail due to out-of-plane bending. The different failure modes of masonry infill walls are local crushing at the corners and shear cracking along the bed joints. These are shown in Figure 13.5. For slender infill walls, there can be diagonal compression failure due to the effect of buckling.

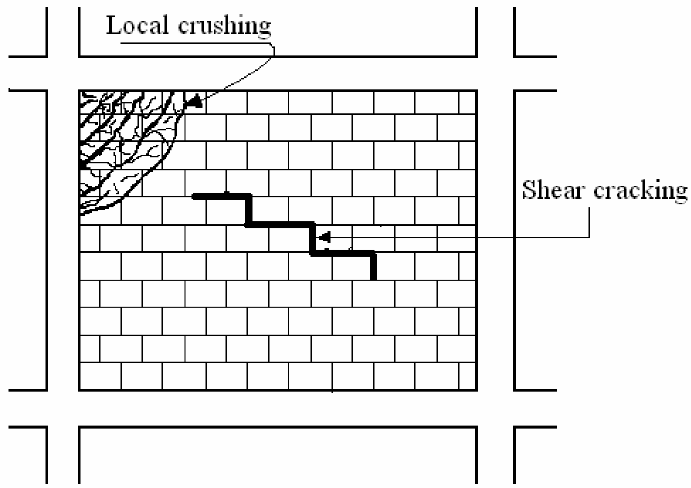


Figure 13.5 Failure modes of masonry infill walls

Retrofitting existing masonry walls using FRP composites is a cost effective means to prevent the loss of life and damage to property during a future earthquake. The possible schemes of layout of FRP wraps are shown in Figure 13.6.

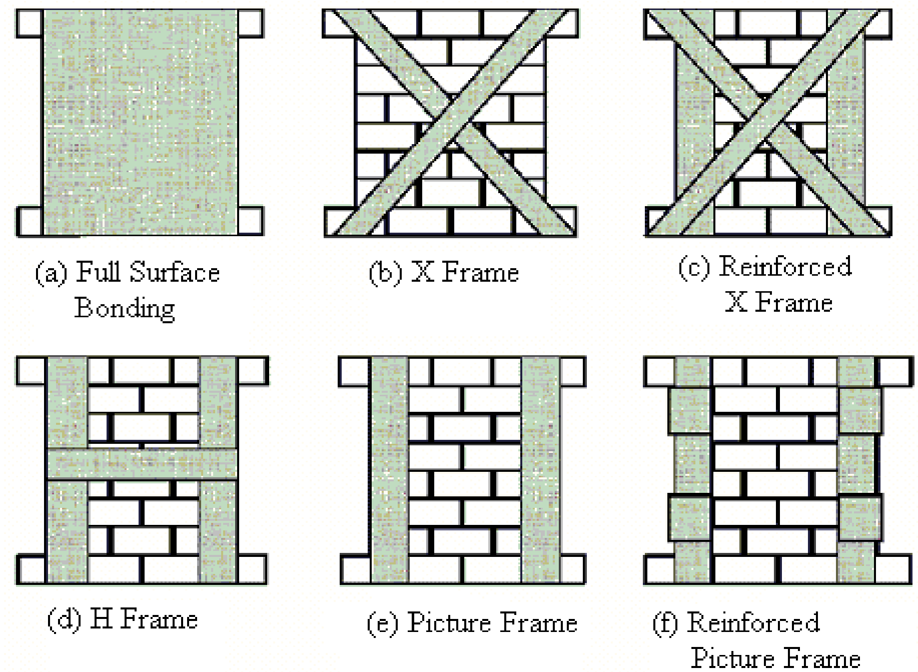


Figure 13.6 Configurations of FRP laminates for retrofitting of masonry walls

13.3.1 Strengthening for In-plane Shear

The shear strength of a masonry wall (V_{uR}) is calculated by superposing the strengths of unreinforced masonry (V_m) and the vertical FRP laminates (V_f). The detailed aspects of bond characteristics, anchoring and stress concentration effects are neglected in the design provisions. A simple procedure for the design for strengthening a masonry wall is given below.

$$V_{uR} = V_m + V_f \quad \textbf{(13.1)}$$

The shear strength of masonry is calculated from the permissible shear stress. As per IS 1905: 1987, the permissible shear stress (f_s) is the least of a) 0.5 MPa, b) 0.1+0.2 f_d and c) 0.125 $\sqrt{f_m}$. Here, f_d is the direct compressive stress in the wall segment and f_m is the crushing strength of the bricks.

$$V_m = f_s\, b\, t \quad \textbf{(13.2)}$$

Here,

b = width of the wall

t = thickness of the wall.

The shear contribution of the FRP laminate is calculated as follows.

$$V_f = E_f\, \varepsilon_{fe}\, A_{fv} \tag{13.3}$$

Here,

A_{fv} = area of vertical FRP laminate.
ε_{fe} = effective usuable strain in the FRP laminate
= $0.5\, \varepsilon_{fu}\, C_E$.

The effective usable strain in FRP can be taken as a factor of the half of the ultimate tensile strain (ε_{fu}). C_E is an environmental reduction factor.

13.3.2 Strengthening for Out-of-plane Bending

FRP laminates are bonded to masonry walls to check the failure due to out-of-plane bending. They also reduce the crack width and the lateral deflection of the walls. The design guidelines ensure compression failure by designing over-reinforced sections. This mode of failure gives warning due to plastic deformation of the masonry. Based on the working stress method for a cracked section, the strain and stress diagrams across a horizontal section of a wall are shown in Figure 13.7. The thickness and width of the masonry wall are represented as t and b, respectively. The width b can be taken as 1 m. A_f is the area of FRP laminate.

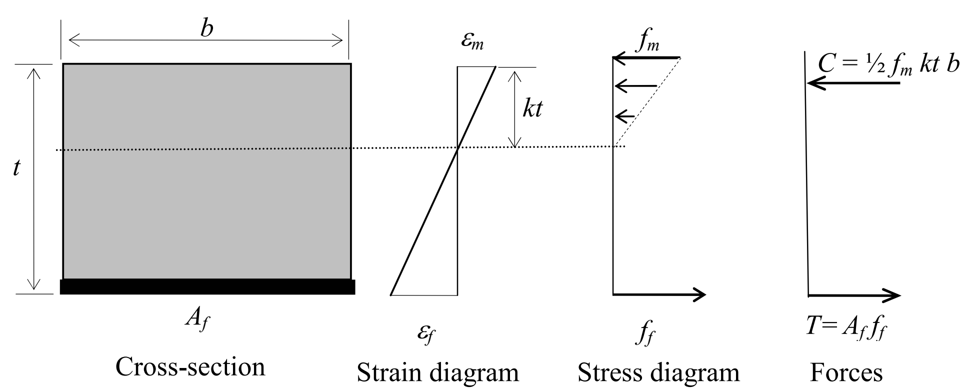


Figure 13.7 Analysis of a wall section bonded with FRP laminate

In the strain diagram, ε_m and ε_f are the extreme compressive strain in masonry and tensile strain in the FRP, respectively. The strain ε_f is limited to the effective usuable value ε_{fe}. In the stress diagram, the compressive stress at the extreme compressive fibre can be determined as $f_m = \varepsilon_m \times E_m$, where the modulus E_m is estimated from the characteristic compressive strength of the masonry $f_m^/$. For clay masonry, E_m= 700 $f_m^/$ and for concrete masonry, E_m= 900 $f_m^/$. The value of f_m should be limited to the permissible compressive stress calculated from $f_m^/$ (IS 1905: 1987, Clause 5.4.1). The tensile stress in the FRP laminate is calculated as $f_f = \varepsilon_f \times E_f$.

The depth of neutral axis '*kt*' can be estimated from the equilibrium of internal forces as follows.

$$\tfrac{1}{2} f_m\, kt\, b = A_f f_f \qquad \textbf{(13.4)}$$

When the FRP reinforcement ratio ρ_f is defined as A_f/bt, the coefficient '*k*' can be obtained in terms of ρ_f by the following equation.

$$k = \sqrt{(m\rho_f)^2 + 2(m\rho_f)} - m\rho_f \qquad \textbf{(13.5)}$$

Here, the modular ratio $m = E_f / E_m$.

The flexural capacity of the retrofitted section is calculated as follows.

$$M_{uR} = \rho_f\, b\, f_f\, t^2\left(1 - \frac{k}{3}\right) \qquad \textbf{(13.6)}$$

13.4 STRENGTHENING OF REINFORCED CONCRETE BEAMS

The flexural capacity of a concrete beam can be increased by bonding unidirectional cloth FRP fabrics/sheets to the tension face, with the fibres oriented along the span of the member. The shear capacity of a beam can be enhanced by bonding fabrics at angles ± 45º to the side faces. The capacities can be increased up to about 40 percent.

13.4.1 Strengthening for Flexural Capacity

The flexural capacity of a concrete beam strengthened with FRP laminate is calculated based on the equilibrium of forces, compatibility of strains and the constitutive relationships. Based on the limit states method of analysis of IS 456: 2000, Figure 13.8 shows the ultimate limit state of a singly reinforced rectangular section with the strain and stress diagrams across the depth and the internal forces. It is assumed that the beam is propped during strengthening, so that under ultimate load the full section is effective. The section should remain under-reinforced, that is the steel should yield before the crushing of the concrete.

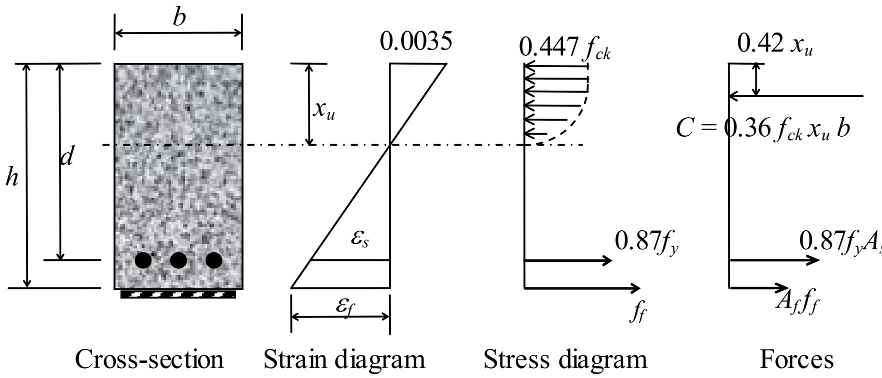


Figure 13.8 Analysis of a beam section bonded with FRP laminate

The ultimate flexural capacity of a member must exceed the flexural demand (Equation. 13.7). The demand under factored loads is obtained from the analysis of the building.

$$M_{uR} \geq M_u \tag{13.7}$$

The ultimate flexural capacity is calculated using Equation. 13.8.

$$M_{uR} = 0.87 f_y A_s \left(d - 0.42 x_u\right) + \psi_f A_f f_{fe} \left(h - 0.42 x_u\right) \tag{13.8}$$

The notations of the variables are as follows.

A_f	= area of the FRP composite
A_s	= area of the steel reinforcement
b	= breadth of beam
C	= compression in concrete
d	= effective depth of the steel reinforcement
f_f	= stress in the FRP composite
f_y	= yield stress of the steel reinforcement
h	= depth of the beam
M_u	= ultimate moment demand
M_{uR}	= ultimate moment capacity (resistance)
x_u	= depth of the neutral axis
ε_s	= strain at the level of steel reinforcement
ε_f	= strain at the level of FRP composite

A reduction factor ψ_f = 0.85 is recommended for the strength from the FRP. The depth of the neutral axis (x_u) and the stress levels in the steel and FRP reinforcement can be obtained using force equilibrium and strain compatibility conditions. The yielding of steel under tension followed by the crushing of concrete under compression is the desired failure mode. Based on this, the thickness of FRP fabric should be calculated.

13.4.2 Strengthening for Shear

The ultimate shear strength of a concrete member strengthened with FRP composite (V_{uR}) must exceed the shear demand (V_u) as shown by Equation. 13.9.

$$V_{uR} \geq V_u \tag{13.9}$$

The shear capacity of a concrete member strengthened by FRP composite can be determined by adding the contribution of the FRP reinforcing (V_f) to the contributions from the reinforcing steel (V_s) and the concrete (V_c). As before, a reduction factor ψ_f is applied to the contribution of the FRP composite.

$$V_{uR} = V_c + V_s + \psi_f V_f \tag{13.10}$$

The reduction factor, ψ_f, should be selected based on the known characteristics of the application, but should not exceed 0.85. Typical schemes of layout of FRP wraps for strengthening for shear are shown in Figure 13.9.

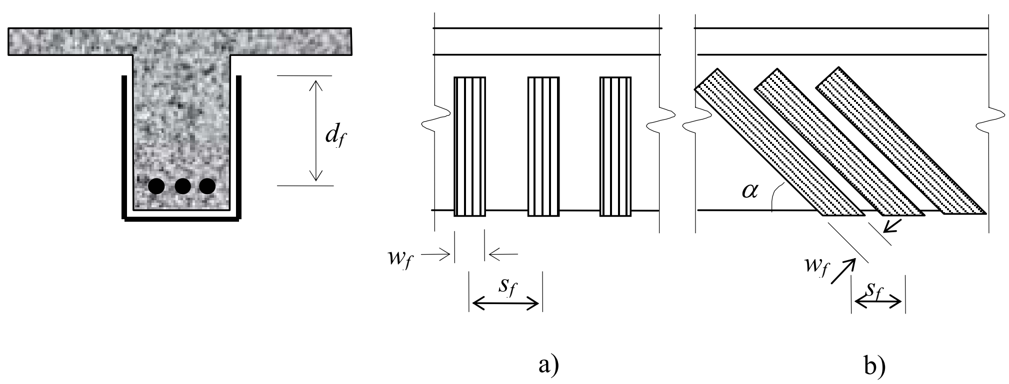


Figure 13.9 Strengthening for shear using FRP composites

The contribution of shear strength provided to a member by the FRP should be based on the fibre orientation and an assumed crack pattern. The shear strength provided by the FRP reinforcement is the force resulting from the tensile stress in the FRP along the assumed crack pattern. For discrete strips of FRP and cracking inclined at 45º, the shear strength is calculated using Equations 13.11 to 13.13.

$$V_f = \frac{A_{fv} f_f (\sin\alpha + \cos\alpha) d_f}{s_f} \tag{13.11}$$

$$A_{fv} = 2nt_f w_f \tag{13.12}$$

$$f_f = \varepsilon_{fe} E_f \tag{13.13}$$

Here,

A_{fv} = total vertical area of the FRP wrap
d_f = effective depth of the FRP wrap
E_f = elastic modulus of FRP
f_f = stress in the FRP wrap
n = number of wraps
s_f = spacing of the FRP wrap
w_f = width of the FRP wrap
α = inclination of the FRP wrap with respect to the beam axis
ε_{fe} = effective usuable strain in the FRP wrap.

For continuous FRP sheet, the expression of V_f is modified based on the length of the sheet intercepted by an inclined crack.

13.5 STRENGTHENING OF REINFORCED CONCRETE COLUMNS

The failure of a column may be due to a single or a combination of the following deficiencies.

1. Inadequate shear capacity
2. Inadequate confinement of column core
3. Inadequate splicing of rebar
4. Inadequate capacity for sustaining combined load effects such as biaxial bending.

The first three inadequacies can be improved by FRP wrapping. For increasing the shear strength or the confinement of concrete or the clamping force at a splice location, the fibre should be oriented horizontally perpendicular to the axis of a column. The wrapping of a column not only increases the strength of the concrete by confinement but also improves the ductility (energy absorption capacity) that is an essential requirement for resisting seismic forces. The increase in thickness of the wrapping improves the strength and ductility, but they are limited by the ultimate strain of the FRP. A typical wrapping of columns using FRP composite is shown in Figure 13.10.

Figure 13.10 Wrapping of column using FRP composites

13.5.1 Strengthening for Shear

The shear failure is the most prevalent failure mode in columns under seismic forces. Assuming 45º cracking, the thickness of the FRP jacket (t_j) for shear retrofit can be determined using the following equation.

$$t_j = \frac{V_u - (V_c + V_s)}{2\varepsilon_{fe} E_j \cdot D} \qquad \textbf{(13.14)}$$

Here,

D	= the column dimension in the direction of loading
E_j	= elastic modulus of composite
V_u	= shear demand based on the flexural capacity in the potential hinge locations
$V_{c,}$	= shear capacity of concrete (IS 456: 2000, Clause 40.2)
V_s	= shear capacity from steel ties
ε_{fe}	= effective usuable strain in the FRP wrap.

13.5.2 Strengthening for Confinement

The thickness of the FRP wrap can be calculated based on the capacity of the confined concrete. For non-circular columns, the confinement is not as effective as a circular column. There are empirical expressions for thickness of the FRP wrap for different column cross-sections.

13.6 STRENGTHENING OF BEAM-COLUMN JOINTS

A beam-column joint is a critical element in a reinforced concrete frame. A joint should maintain its integrity and must be designed stronger than the members framing into it. Quite often, the detailing at the joint is not taken care of. The weakness at the joints is due to the lack of confinement in absence of ties and the lack of adequate anchorage of the longitudinal bars of the beams. The joints which are suitable for retrofitting, such as those in exposed frames in industrial buildings, water tanks, bridges and other structures, can be strengthened using FRP composites. The wrapping of the FRP laminates enhances the confinement of the joint. Typical schemes of bonding the FRP laminates is shown in the following figure (Mukherjee and Joshi, 2002)

13.7 SUMMARY

This chapter explains the retrofitting of masonry and concrete buildings using fibre reinforced polymer (FRP) composites. Brief descriptions on the constituent materials of FRP, the types and installation of FRP composite are provided. Guidelines for calculating the thickness of the FRP laminates for strengthening masonry walls, reinforced concrete beams and columns are given.

13.8 REFERENCES

1. ACI 440R-96, "Report on Fibre Reinforced Plastic Reinforcement for Concrete Structures", American Concrete Institute.
2. IS 456: 2000, "Plain and Reinforced Concrete – Code of Practice", Bureau of Indian Standards.
3. IS 1905: 1987, "Code of Practice for Structural Use of Unreinforced Masonry", Bureau of Indian Standards.
4. Mukherjee, A. and Joshi, M. V. (2002), "Seismic Retrofitting Techniques using Fibre Composites", The Indian Concrete Journal, The Associated Cement Companies Ltd., August, Vol. 76, No. 8, pp. 496 – 502.
5. Mukherjee, A., Kalyani, A. R. and Joshi, M. V. (2004), "Upgradation of RCC Frames with FRC − I, Design of Elements", The Indian Concrete Journal, The Associated Cement Companies Ltd., October, Vol. 78, No. 10, pp. 15 – 19.
6. Mukherjee, A., Kalyani, A. R. and Joshi, M. V. (2004), "Upgradation of RCC Frames with FRC − II, Design of Structures", The Indian Concrete Journal, The Associated Cement Companies Ltd., October, Vol. 78, No. 10, pp. 22 – 25.

14

BASE ISOLATION AND ENERGY DISSIPATION

14.1 INTRODUCTION

Conventional seismic design practice permits the reduction of forces for design below the elastic level on the premise that inelastic action in a suitably designed structure will provide that structure with significant energy dissipation potential and enable it to survive a severe earthquake without collapse. This inelastic action is typically intended to occur in specially detailed critical regions of the structure, usually in the beams near or adjacent to the beam-column joints. Inelastic behavior in these regions, while capable of dissipating substantial energy, also often results in significant damage to the structural member, and although the regions may be well detailed, their hysteretic behavior will degrade with repeated inelastic cycling. The interstory drifts required to achieve significant hysteretic energy dissipation in critical regions are generally large and usually result in substantial damage to non-structural elements such as in-fill walls, partitions, doorways, and ceilings. As a response to the shortcomings, in recent years, considerable attention has been paid to research and development of structural control devices, with particular emphasis on reduction of seismic response of buildings and bridges. In this field, serious efforts have been undertaken to transform the structural control concept into a workable technology, and many such devices have been installed in a wide variety of structures throughout the world. By and large, structural control devices can be grouped into four broad areas:

1. Base isolation
2. Passive energy dissipation
3. Active control, and
4. Semiactive control

In this chapter, an overview of various base isolation systems and passive energy dissipation devices that have been proposed and developed for reduction of seismic response of buildings is presented. However, active control devices and semi-active control devices are not discussed because it is unlikely that these devices would be used for seismic retrofitting of buildings in our country in near future.

14.1.1 Fundamentals of Passive Control Systems

Base Isolation

The principle of seismic isolation is to introduce flexibility at the base of a structure in the horizontal plane, while at the same time introducing damping elements to restrict the amplitude of the motion caused by the earthquake. The concept of isolating structures from the damaging effects of earthquakes is not new. However, a new impetus was given to the concept of seismic isolation by the successful development of mechanical-energy dissipaters and elastomers with high damping properties. Mechanical energy dissipation devices, when used in combination with a flexible base isolation device, can control the response of the structure by limiting displacements and forces, thereby significantly improving seismic performance.

Energy Dissipating Devices

The role of a passive energy dissipation device is to increase the hysteretic damping in the structure. The basic energy relationship of the structure is represented in the following equation:

$$E_I = E_K + E_S + E_\xi + E_H$$

where

E_I = earthquake input energy
E_K = kinetic energy in structure
E_S = strain energy in structure
E_ξ = viscous damping energy
E_H = hysteretic damping energy

The role of this equation in the design process has been developed by a number of researchers. The goal is to increase E_H so that, for a given E_I, the elastic strain energy E_S in the structure is minimized. This means that the structure will undergo smaller deformations for a given level of input energy than if it did not include energy dissipaters. Alternatively, increasing E_H permits E_S to be reduced for a higher level of E_I.

Tuned Systems

Tuned systems are supplemental devices attached to structures to reduce vibrations due to wind, earthquakes, or other dynamic loading conditions. Because the natural frequencies of these devices are equal or close to those of the structures to which they are attached, they are called tuned systems.

14.2 BASE ISOLATION SYSTEMS

The objective of seismic isolation systems is to decouple the building structure from the damaging components of the earthquake input motion, i.e., to prevent the superstructure of the building from absorbing the earthquake energy. The entire superstructure must be supported on discrete isolators whose dynamic characteristics are chosen to uncouple the ground motion. Some isolators are also designed to add substantial damping. Displacement and yielding are concentrated at the level of the isolation devices, and the superstructure behaves very much like a rigid body.

14.2.1 Important Base Isolation Systems

Some of the commonly used isolation systems are laminated rubber (or elastomeric) bearings and sliding isolation systems. Laminated rubber bearings are used with passive dampers for control of excessive base displacement. Laminated rubber bearings with inherent energy dissipation capacities have also been developed. Lead rubber bearings and high damping rubber bearings are examples of this category of isolation system. Sliding bearings mainly utilizes Teflon-stainless steel, flat or spherical, interface. Sometimes, separate elements are provided for recentering of the isolated system. Two of such systems are Friction Pendulum System (FPS) and TASS system. Performance of base isolated buildings in different parts of the world during earthquakes in the recent past established that the base isolation technology is a viable alternative to conventional earthquake resistant design of medium-rise buildings. Four important base isolation systems (Naeim and Kelly, 1999) are shown in Figure 14.1.

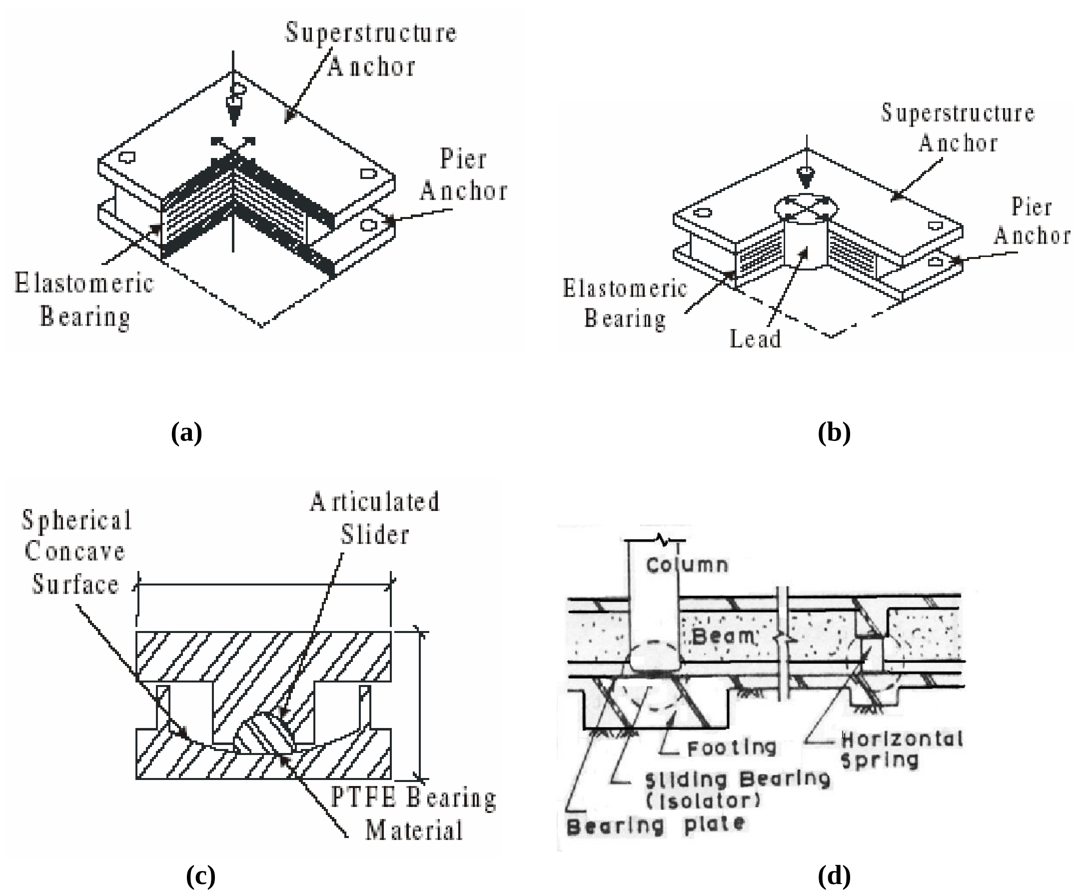


Figure 14.1 Important Base Isolation Systems: (a) laminated rubber bearing, (b) Lead rubber bearing, (c) FPS System and (d) TASS System

14.2.2 Suitability of Base Isolation Systems

Earthquake protection of structures using base isolation technique is generally suitable if following conditions are fulfilled:

- The subsoil does not produce a predominance of long period ground motion.
- The structure is fairly squat with sufficiently high column load.
- The site permits horizontal displacements at the base of the order of 200 mm or more.
- Lateral loads due to wind are less than approximately 10% of the weight of the structure.

14.2.3 Design Parameters for Seismic Isolation Bearing

As per UBC 1997 / FEMA 356 (2000), a minimum level for design displacements and forces is to be obtained from static analysis for all seismic isolation designs. The static analysis is also useful both for preliminary design of isolation system and the structure. Dynamic analysis may be used in all cases and must be used if the requirements mentioned for adequacy of static analysis are not satisfied.

Minimum design displacements

Four distinct displacements calculated using simple formulas and used for static analysis. These values also serve as the UBC 1997 / FEMA 356, 2000 permitted lower bound values for dynamic analysis results. These are:

- D_D : the design displacement, being the displacement at centre of rigidity of isolation system at design basis earthquake (DBE);
- D_M: the design displacement, being the displacement at centre of rigidity of isolation system at maximum considered earthquake (MCE);
- D_{TD}: the total design displacement, being the displacement of a bearing at a corner of the building and includes the component of the torsional displacement in the direction of D_D
- D_{TM}: same as D_{TD} but calculated for MCE.

D_D and D_M are given as:

$$D_D = \frac{(g/4\pi^2)S_{D1}T_D}{\beta_D} \tag{14.1}$$

$$D_M = \frac{(g/4\pi^2)S_{M1}T_M}{\beta_M} \tag{14.2}$$

where g is the acceleration due to gravity, S_{D1} and S_{M1} are spectral coefficients, T_D and T_M are isolated periods, and β_D and β_M are damping coefficients corresponding to the DBE and MCE level responses, respectively.

Effective isolated system periods

The effective isolated periods T_D and T_M corresponding to the DBE and MCE level response are:

$$T_D = 2\pi\sqrt{\frac{W}{K_{D,\min}g}} \tag{14.3}$$

$$T_M = 2\pi\sqrt{\frac{W}{K_{M,\min}g}} \tag{14.4}$$

where W = the weight of the building

g = acceleration due to gravity

K_{Dmin} = minimum effective horizontal stiffness of the isolation system at the design displacement (DBE).

K_{Mmin} = minimum effective horizontal stiffness of the isolation system at the maximum displacement (MCE).

The values of K_{Dmin} and K_{Mmin} are not known during preliminary design phase and hence the design process will begin with an assumed value, which is obtained from previous tests on similar components. After moulding of prototype bearing, the actual values of K_{Dmin} , K_{Dmax} , K_{Mmin} and K_{Mmax} will be obtained from the results of shear test on bearings. K_{Dmax} *and* K_{Mmax} are maximum effective stiffness at displacement corresponding to DBE and MCE, respectively.

The total design displacements, D_{TD} and D_{TM} are given as:

$$D_{TD} = D_D\left(1 + y\frac{12e}{b^2 + d^2}\right) \tag{14.5}$$

$$D_{TM} = D_M\left(1 + y\frac{12e}{b^2 + d^2}\right) \tag{14.6}$$

where b and d are plan dimensions at the isolation plane, e is the actual eccentricity plus 5% accidental eccentricity, and y is the distance to a corner perpendicular to the direction of seismic loading.

<u>Design forces</u>

The superstructure and elements below the isolation interface are designed for forces based on DBE design displacement, D_D. The isolation system, the foundation and structural elements below the isolation system must be designated to withstand the following minimum lateral seismic force:

$$V_b = K_{Dmax}\, D_D \tag{14.7}$$

If other displacements rather than D_D generate larger forces, then those forces should be used in design rather than the force obtained from Eq.2.7.

The structure above the isolation plane should withstand a minimum shear force, V_s, as if it is fixed base where:

$$V_s = \frac{K_{D\max} D_D}{R_I} \qquad \textbf{(14.8)}$$

In IBC2000, R_I is defined as

$$1.0 \le R_I = 3/8\, R \le 2.0 \qquad \textbf{(14.9)}$$

where R is reduction factor defined in the code for superstructure.

14.2.4 Testing Requirements for Isolation Systems

The UBC 1997 code requires that at least two full-sized specimens of specimens of each type of isolator be tested. The tests required are a specified sequence of horizontal cycles under DL + 0.5LL from small horizontal displacements up to D_{TM} . In addition, tests are also carried out for the maximum vertical load $1.2DL + 0.5LL + E_{max}$ and for the minimum load $0.8DL - E_{min}$ where E_{max} and E_{min} are the maximum downward and upward load on the isolator that can be generated by an earthquake.

14.2.5 Typical Set-up for Shear Test of Isolation Systems

Aiken et al. (1989) carried out extensive testing of high damping laminated rubber bearings (LRB) using a typical set-up for shear test shown in Figure 14.2. On the basis of the test results, a number of comparisons have been made for bearings with different characteristics. The influences of axial load and shear strain on bearing characteristics, e.g., shear stiffness, vertical stiffness, and damping behaviour have been investigated. Figure 14.3 shows a typical force-displacement hysteresis loop of high damping LRB obtained during shear test. The shear tests demonstrated that the LRBs possess stable stiffness and damping properties.

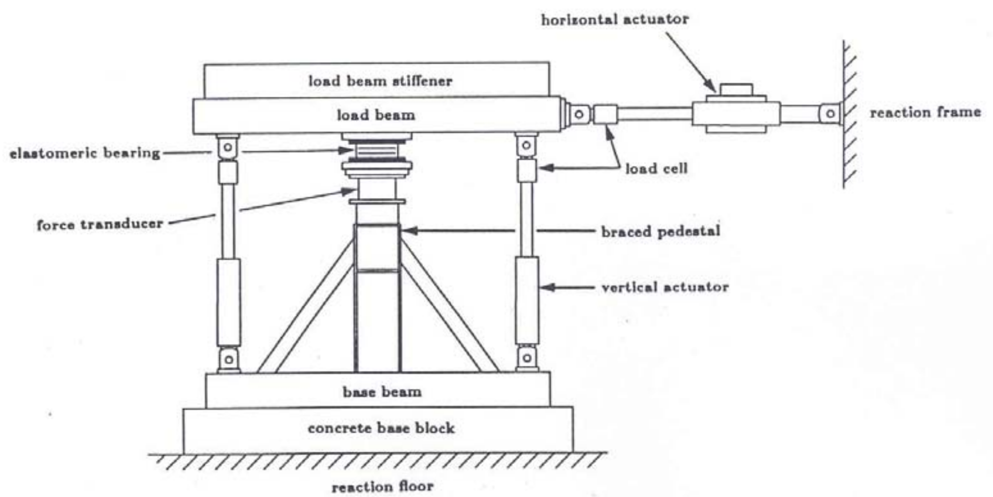


Figure 14.2 Typical set up for Shear Test of Laminated Rubber Bearing (Aiken et al., 1989)

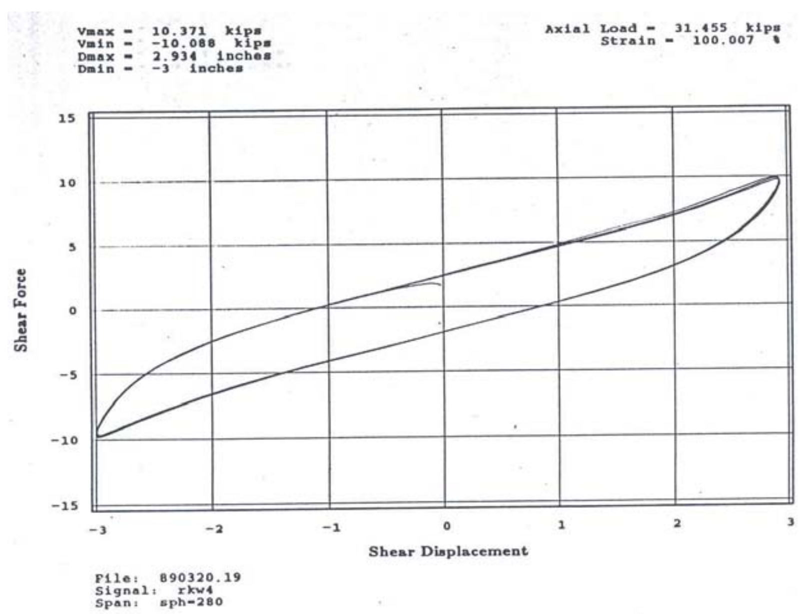


Figure 14.3 A typical force-displacement hysteresis loop of laminated rubber bearing (Aiken et al., 1989)

14.2.6 Details of Connections

Base isolation systems are placed at the interface of superstructure and substructure. A typical connection detail of a laminated rubber bearing with superstructure and substructure is shown in Figure 14.4.

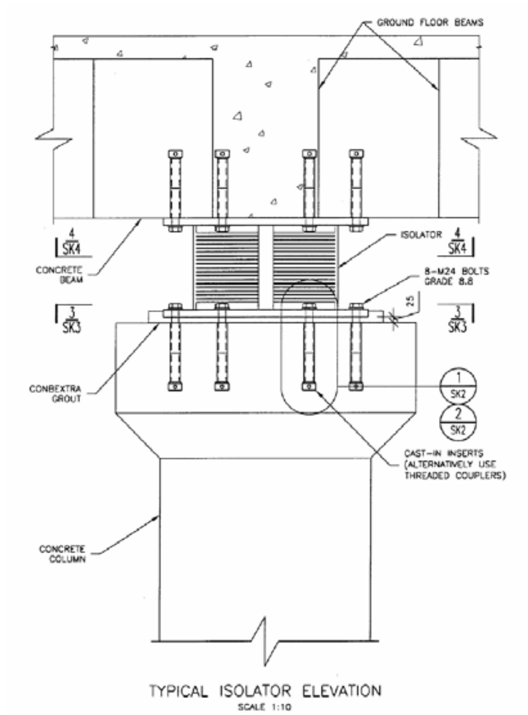


Figure 14.4 Connection Details

14.2.7 Example on Finalisation of Design Parameters as per UBC 1997 / FEMA 356

The steps involved are explained with sample calculations for design parameters of seismic isolation bearings for the proposed three storeyed reinforced concrete base isolated building at IIT Guwahati. The project is sponsored by BRNS, GOI.

Steps:

1. Seismic Zone Factor: $Z = 0.36$

   Near Source factors: $N_a = 1$; $N_v = 1$

2. For $Z.N_v$ = 0.36 × 1 = 0.36, maximum capable earthquake response coefficient M_M = 1.50

3. For Z = 0.36 and soil type S_D , seismic coefficients C_{VD} = 0.6, C_{AD} = 0.4

4. For $M_M Z N_v$ = 1.5 × 0.36 × 1 =0.54 and $M_M Z Na$ = 0.54, seismic coefficients (M_M is MCE response coefficient)
   C_{VM} = 0.864, C_{AM} = 0.594

5. R_I = 2.0

6. For 20 % damping ratio, damping coefficients $\beta_D = \beta_M$ = 1.5

7. T_D and T_M are assumed as 1.8 sec and 2.1 sec respectively corresponding to DBE & MCE

8. $$T_D = 2\pi\sqrt{\frac{W}{K_{D,min}\,g}}$$

   where, W is the weight of the building in kN
   $K_{D,min}$ is minimum design stiffness of the bearing
   g is the acceleration due to gravity in m/s^2

   $K_{D,min}$ = 1422.52 kN/m for T_D = 1.8 sec
   $K_{D,min}$ = 1045.11 kN/m for T_M = 2.1 sec

   Assuming a +10 % variation about mean stiffness value

   $K_{D,max}$ = 1738.64 kN/m
   $K_{M,max}$ = 1277.36 kN/m

9. Displacement in bearing corresponding to DBE and MCE

   $$D_D = \frac{(g/4\pi^2)C_{VD}T_D}{\beta_D} = 0.18 \text{ m}$$

   $$D_M = \frac{(g/4\pi^2)C_{VM}T_M}{\beta_M} = 0.30 \text{ m}$$

10. $D_{TD} = D_D\left(1 + y\dfrac{12e}{b^2 + d^2}\right)$ = 0.22 m

$D_{TM} = D_M\left(1 + y\dfrac{12e}{b^2 + d^2}\right)$ = 0.34 m

Details of design procedure / criteria of various base isolation systems are presented in FEMA 356, 2000.

14.3 PASSIVE ENERGY DISSIPATING DEVICES

14.3.1 Friction Dampers

A variety of friction devices has been proposed and developed for energy dissipation in buildings. Most of these devices generate rectangular hysteresis loops. Figure 14.5 indicates that the behaviour of friction dampers is similar to that of Coulomb friction. Generally, these devices have good performance characteristics, and their behaviour is relatively less affected by the load frequency, number of load cycles, or variations in temperature. Furthermore, these devices have high resistance to fatigue. The devices differ in their mechanical complexity and in the materials used for the sliding surfaces. An example of friction dampers proposed by Pall and Marsh (1982) is a device that can be located at the intersection of cross bracings in frames as shown in Figure 14.6. When loaded, the tension brace induces slippage at friction joint. Consequently, the four links force the compression brace to slip. In this manner, energy is dissipated in both braces even though they are designed to prevent slippage under normal service loads. Effectiveness of these devices in providing a substantial increase in energy dissipation capacity and reducing inter-story drifts in comparison to moment resisting frames without such devices has been witnessed in several practical implementation. Filiatrault and Cherry (1990) have developed a design method to estimate the optimum slip load distribution for the Pall friction dampers.

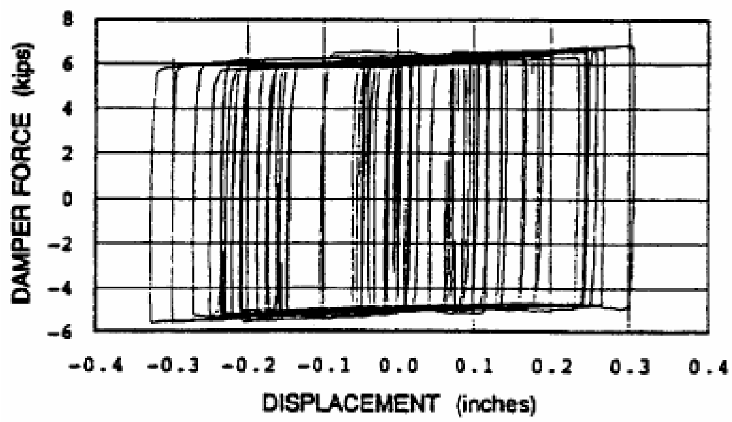


Figure 14.5 Hysteresis loops for Sumitomo friction dampers (Aiken et al. 1992)

Sumitomo Metal Industries of Japan introduced another device that utilizes friction to dissipate energy. The device was originally used as shock absorbers in railway cars and recently its application was extended to structures. Figure 14.7 shows the construction of a typical Sumitomo friction damper. The device consists of copper pads impregnated with graphite in contact with steel casing of the device. The load on the contact surface is developed by a series of wedges, which act under the compressive force of the Belleville washer springs (cup springs). The graphite serves as a lubricant between the contact surfaces and ensures a stable coefficient of friction and silent operation. The dampers can be placed parallel to the floor beams with one end attached to the floor beam above and the other end connected to a stiff chevron brace arrangement attached to the floor beam below as shown in Figure 14.8. Similar to Pall friction dampers, reductions in displacements were observed using the Sumitomo friction damping devices. The reductions, however, depend on the input ground motion because friction dampers are not activated by small excitations. These dampers have been used for seismic protection of few buildings in Japan. There are many other types of friction dampers developed in different parts of the world. Slotted Bolt Connection proposed by Fitzgerald et al. (1989) and Energy Dissipating Restraint (EDR) developed by Richter et al. (1990) are two more important dampers in the category of friction dampers.

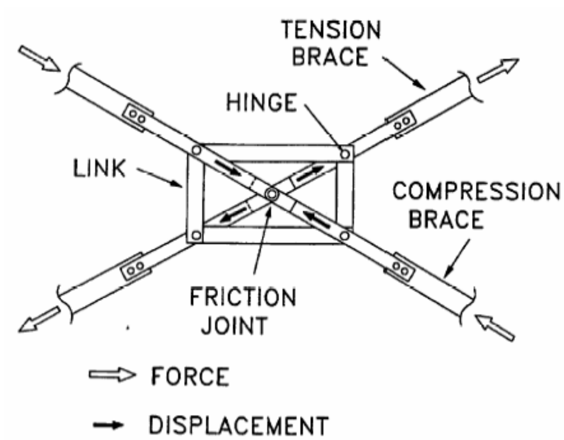


Figure 14.6 Pall friction damper (Pall and marsh, 1982)

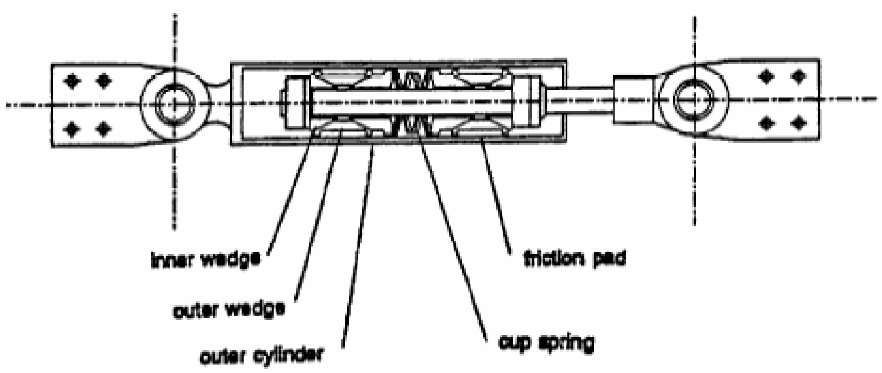


Figure 14.7 Sumitomo friction damper (Aiken et al. 1992)

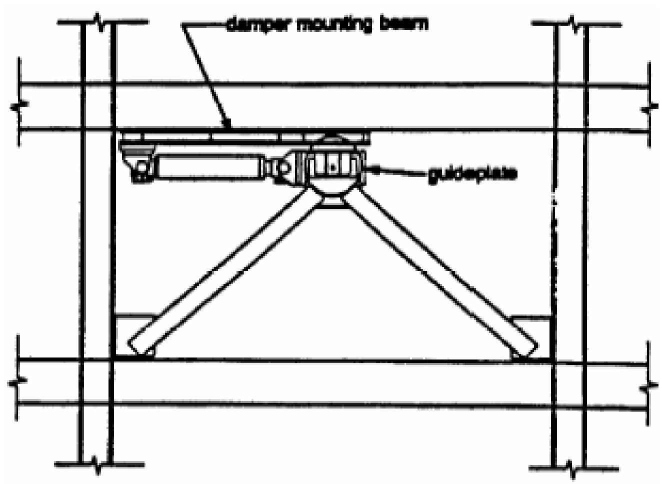


Figure 14.8 Installation details of Sumitomo friction dampers in the experimental frame (Aiken et al. 1992)

14.3.2 Metallic Dampers

This type of energy dissipation devices utilizes the hysteretic behaviour of metals in the inelastic range. The resisting force of the dampers, therefore, depends on the nonlinear stress-strain characteristics of the material. Different devices that utilize flexure, shear, or extensional deformation in the plastic range have been developed. The advantages of this type of dampers are their stable behaviour, long-term reliability, and good resistance to environmental and thermal conditions. Moreover, metallic dampers are capable of providing buildings with increased stiffness, strength, and energy dissipation capacity. The following describes several types of metallic dampers.

<u>Yielding Steel Dampers</u>

The yield properties of mild steel have long been recognized and used to improve the seismic performance of buildings. The eccentrically-braced frame represents a widely accepted concept where energy dissipation can be concentrated primarily at shear links. These links represent part of the structural system, which is likely to suffer damage in severe earthquakes. The ability of braced frames to dissipate energy over extended periods is questionable because the repeated buckling and yielding of the braces may cause degradation of their stiffness and strength.

Figure 14.9 shows an energy dissipater fabricated from round steel bars for cross-braced buildings by Tyler (1995). In this device, the compression brace disconnects from the rectangular steel frame to prevent buckling and pinched hysteretic behaviour. Energy is dissipated by inelastic deformation of the rectangular steel frame in the diagonal direction of the tension brace. This device has been used in New Zealand and a variation of this device has been used for seismic response control of a 29 storeyed building in Italy.

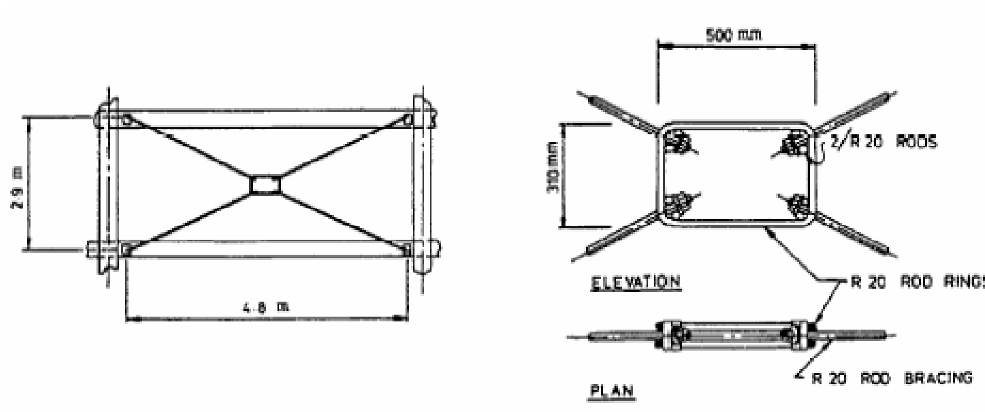
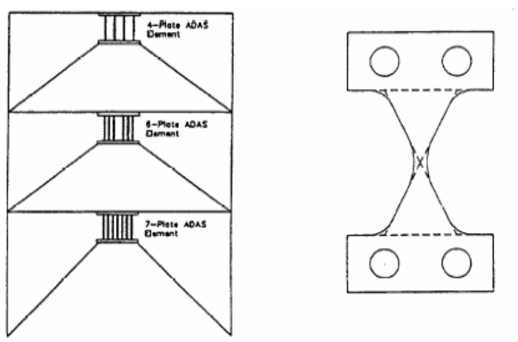
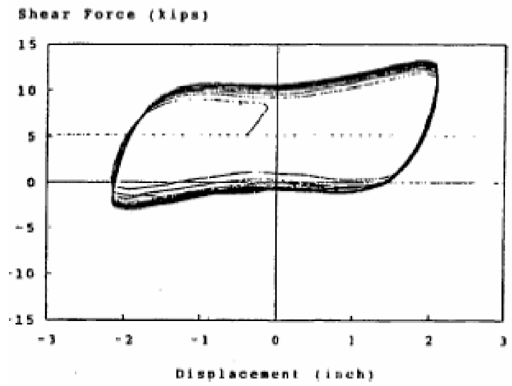


Figure 14.9 Yielding steel bracing system (Tyler, 1985)

Another device, referred to as *added damping and stiffness* (ADAS) consisting of multiple X-shaped steel plates (Figure 14.10) was introduced by Bechtel Power Corporation. By using rigid boundary members, the plates deform in double curvature, and yielding takes place over the entire plate surface. The device can sustain repeated inelastic deformation by avoiding concentrations of yielding and premature failure. Extensive experimental studies have been carried out to investigate the behaviour of ADAS elements in dissipating energy (Whittaker et al. 1991). The tests showed stable hysteretic behaviour without any sign of pinching or stiffness degradation for displacements up to 13.6 times yield displacements of the device (Figure 14.11). It has been observed that the inter-storey drifts in the frame have reduced by 30 to 70 percent with the addition of the ADAS elements. The ratios of base shears in the structures with ADAS elements to those without ADAS elements, however, ranged from 0.6 to 1.25. The ADAS elements yield in a predetermined manner and relieve the frame from excessive ductility demands.





(b)

Figure 14.10 ADAS elements and installation **Figure 14.11** Hysteresis loop(Whittaker et.al.1991)

Triangular plate energy dissipaters were developed in New Zealand and used as damping elements in several base isolated applications. Later, they were used in buildings in the form of Triangular ADAS (T-ADAS) elements (Tsai and Hong, 1992). Typical hysteresis loops for the T-ADAS elements are shown in Figure 14.12. Large numbers of similar dampers based on yielding of steel have developed in New Zealand and Japan.

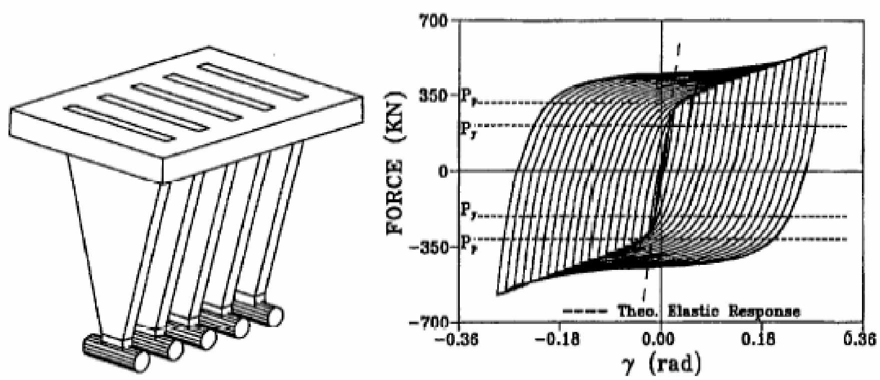


Figure 14.12 Hysteresis loops for T-ADAS devices (Tsai and Hong, 1992)

Friction and yielding devices share the following characteristics: they are force limited, highly nonlinear, and their response is velocity independent. Scholl (1993) has suggested a procedure for their design, which considers a SDOF frame with stiffness k_S and a damping device with stiffness k_d attached to a brace with stiffness k_b. The combined stiffness of the device and the brace k_{bd} is equal to $k_b/(1+k_b/k_d)$. The stiffness ratio *SR* is defined as the ratio of the combined stiffness of the device and brace to that of the structure; thus,

$$SR = \frac{k_{bd}}{k_S} = \frac{k_b/k_S}{1+k_b/k_d} \tag{14.10}$$

and the force ratio *FR* is defined as the ratio of the force in the damper to the elastic force in the structure at the maximum displacement. It should be noted that for friction dampers, there is no initial flexibility; i.e., $k_d = \infty$ and thus, $SR = k_b/k_S$. Scholl has shown that the equivalent damping ratio ξ of the device should take the form:

$$\xi = \frac{2FR}{\pi}\left(\frac{SR-FR}{SR+FR^2}\right) \tag{14.11}$$

The effect of the parameters *SR* and *FR* on the damping ratio is presented in Figure 14.13. For the design of the dampers, the following limitations should be imposed on the stiffnesses of the different components: 1) the ratio k_b/k_d should be kept as large as possible to maximize the energy dissipated by the dampers. A value of $k_b/k_d \geq 2$ has been found to be practical for design; 2) according to Figure 14.13, increasing *SR* results in a higher damping ratio. Therefore, the largest feasible value of *SR* is desirable. For practical applications, it is difficult to achieve *SR* values of 3 or 4. The results of several nonlinear computer simulations indicate that $SR \geq 2$ is a recommended value for design.

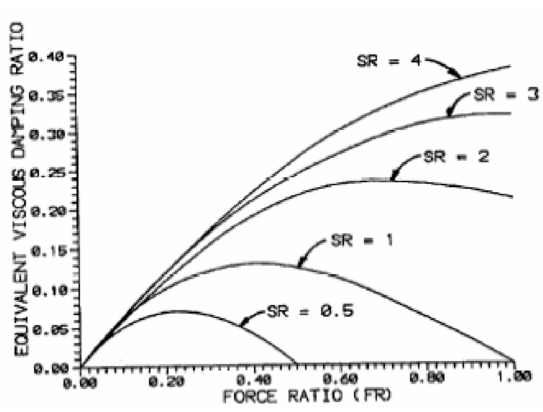


Figure 14.13 Equivalent damping ratio for friction and yielding devices (Scholl, 1993)

Based on the analysis and limitations outlined above, the design of friction and yielding dampers may proceed as follows: 1) perform a dynamic or a response spectrum analysis for the structure without dampers to determine the target damping ratio ξ for which the building response remains elastic; 2) determine the stiffnesses of the braces and those of the dampers (for yielding devices only) from Eq. (14.10) and the limitations mentioned above; 3) estimate the value of *FR* from Figure 14.13 or from Eq. (14.11) using the target damping ratio ξ; 4) compute the yield load for yielding devices or the slip load for friction devices based on the computed *FR;* and 5) perform a nonlinear response analysis after selecting the parameters to ensure the safety and stability of all components.

The above discussion indicates that the yielding steel dampers may be effective as passive energy dissipation devices in reducing the response of structures to earthquake loading. The post-yield deformation range of the devices is a major concern, which should be addressed to insure that the device can sustain a sufficient number of cycles of deformation without premature fatigue. Another concern is the stable hysteretic behavior of the dampers under repeated inelastic deformation.

<u>Lead Extrusion Devices</u>

Another type of damper which utilizes the hysteretic energy dissipation properties of metals is the lead extrusion damper (LED). The process of extrusion consists of forcing a material through a hole or an orifice, thereby altering its shape. The extrusion of lead was identified as an effective means of energy dissipation. LEDs were first suggested by Robinson as a passive energy dissipation device for base isolated structures in New Zealand. Two devices introduced by Robinson are shown in Figure 14.14 (Robinson and Cousins, 1987). The first device (Figure 14.14a) consists of a thick-walled tube and a co-axial shaft with a piston. There is a constriction on the tube between the piston heads and the space between the piston heads is filled with lead. The lead is separated from the tube by a thin layer of lubricant kept in place by hydraulic seals around the piston heads. The central shaft extends beyond one end of the tube. When external excitation is applied, the piston moves along the tube and the lead is forced to extrude back and forth through the orifice formed by the constriction of the tube. The second device (Figure 14.14b) is similar to the first except that the extrusion orifice is formed by a bulge on the central shaft rather than by a constriction in the tube. The shaft is supported by bearings, which also serve to hold the lead in place. As the shaft moves, the lead must extrude through the orifice formed by the bulge and tube. Similar to most friction devices, the hysteretic behavior of LEDs is essentially rectangular (Figure 14.15). Examples of the application of LEDs in New Zealand include a ten-story base-isolated cross-braced concrete building with sleeved piles and LED damping elements used as a police station in Wellington, and several seismically isolated bridges.

Lead extrusion devices have the following advantages: 1) their load deformation relationship is stable and unaffected by the number of loading cycles; 2) they are insensitive to environmental conditions and aging effects; and 3) they have a long life and do not have to be replaced or repaired after an earthquake since the lead in the damper returns to its undeformed state after excitation.

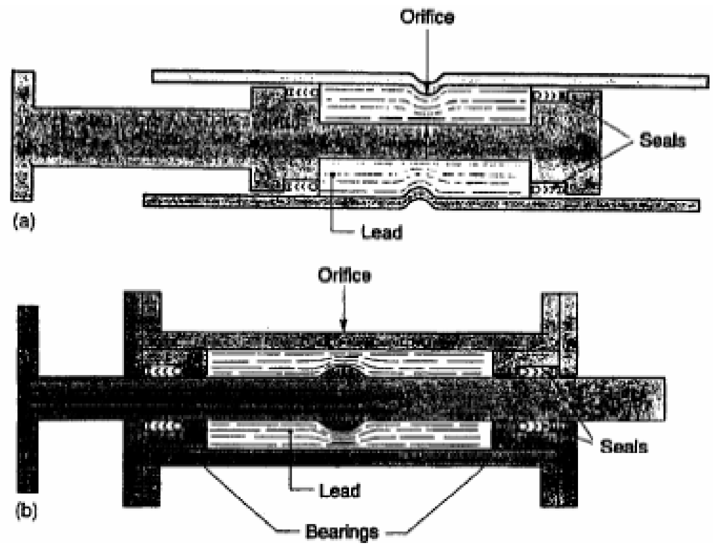


Figure 14.14 Longitudinal section of lead extrusion dampers: (a) constricted-tube type and (b) buldges-shaft type (Skinner et al., 1993)

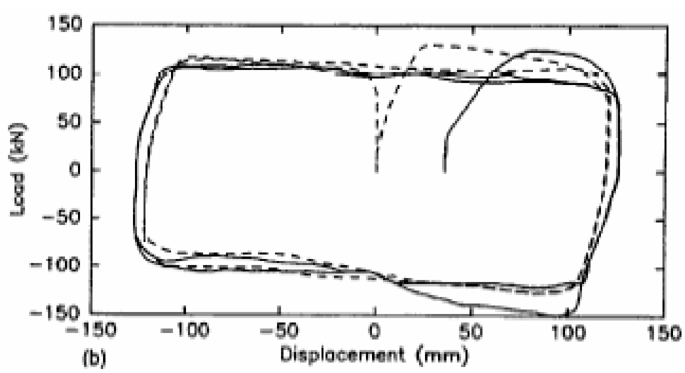


Figure 14.15 Hysteresis loops of LEDs (Robinson and Cousins, 1987)

Shape Memory Alloys

In contrast with most materials such as steel, which experience intergranular dislocation when loaded, shape memory alloys (SMA) undergo a reversible phase transformation as they deform and therefore have the ability to yield repeatedly without sustaining any permanent deformation. The yielding mechanism is such that the applied load induces a crystal phase transformation, which is reversed when the load is removed (Figure 14.16). The figure shows that at low stresses, the material behaves elastically. At high stresses, phase transformation takes place; thereby reducing the modulus of elasticity. During unloading, the material undergoes a reverse transformation at a stress lower than that for loading. Once the reverse phase transformation is complete, the material behaves elastically again. This behavior can be utilized in devices, which have self-centering capabilities and undergo repeated hysteretic cycles. SMA devices are relatively insensitive to temperature changes and can sustain large loads, which make them suitable for base isolation systems. Some members of the SMA family exhibit excellent fatigue resistance. Among the SMA family, Nitinol (nickel-titanium) has corrosion resistance superior to other corrosion-resistant materials such as stainless steel. Shape memory alloys, however, are extremely expensive.

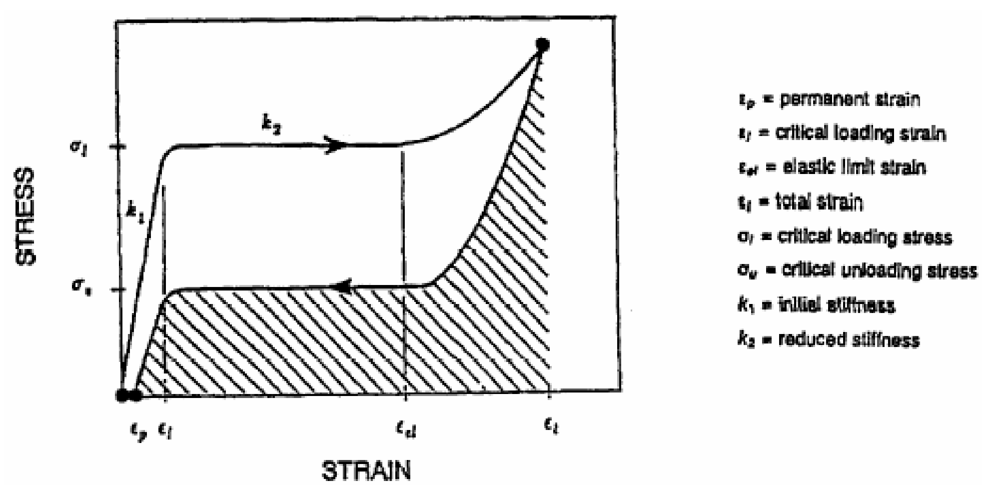


Figure 14.16 Superelastic behaviour of shape memory alloys (Aiken et al., 1992)

SMA devices have been tested under earthquake loadings. Aiken et al. (1992) tested a three story steel frame with Nitinol tension devices as part of a cross-bracing system. Results of this study reveal that SMAs are effective in reducing the seismic response of the structures. Typical hysteresis loop is shown in Figure 14.17. Whittaker et al. (1995) suggested upgrading an existing 3-story non-ductile reinforced concrete building using SMA dampers to meet the current seismic code criteria. Analytical studies indicated that the use of SMA dampers in the bracing system significantly reduces the seismic response of the building.

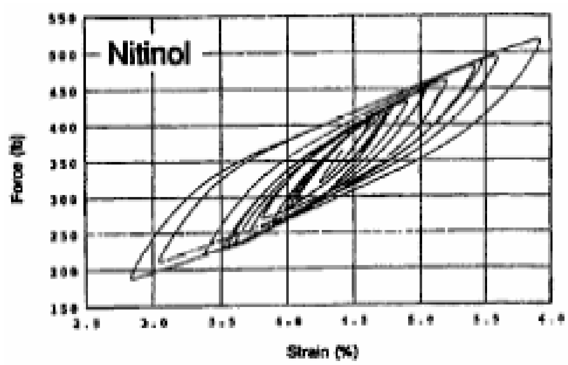


Figure 14.17 NiTi hysteresis loops (Aiken et al., 1992)

14.3.3 Viscoelastic Dampers

Viscoelastic (VE) dampers have been used as energy dissipation devices in structures where the damper undergoes shear deformations. As their name implies, viscoelastic materials exhibit combined features of elastic solid and viscous liquid when deformed, i.e., they return to their original shape after each cycle of deformation and dissipate a certain amount of energy as heat. These dampers, made of bonded viscoelastic layers (acrylic polymers), have been developed by 3M company and used to control wind-induced vibrations in buildings. The 3M dampers are known to have a stable behavior with good aging properties and resistance to environmental pollutants. The extension of VE shear dampers to seismic applications is more recent. For seismic applications, more effective use of VE materials is required since larger damping ratios than those for wind are usually required.

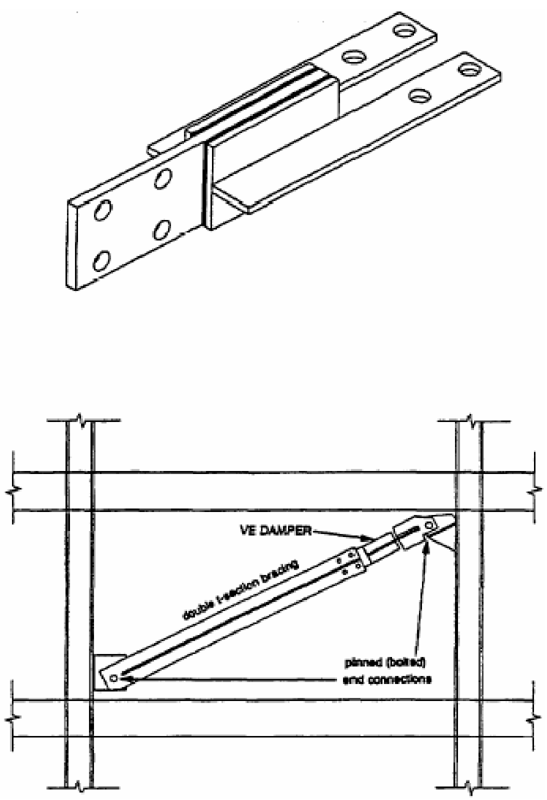


Figure 14.18 Viscoelastic damper and installation ((Aiken and Kelly, 1990)

A typical VE shear damper consists of viscoelastic layers bonded to steel plates (Figure 14.18). When mounted to a structure, shear deformations and consequently energy dissipations take place when relative motions occur between the center plate and the outer steel flanges. To understand their behavior under a sinusoidal load with a frequency $\bar{\omega}$, the shear stress $\tau(t)$ can be expressed in terms of the peak shear strain γ_0 and peak shear stress τ_0 as (Zhang et al., 1989)

$$\tau(t) = \gamma_0[G'(\bar{\omega})\sin \bar{\omega}t + G''(\bar{\omega})\cos \bar{\omega}t \quad \textbf{(14.12)}$$

where $G'(\bar{\omega}) = \tau_0 \cos\delta/\gamma_0, G''(\bar{\omega}) = \tau_0 \sin\delta/\gamma_0$ and δ is the lag (phase) angle between the shear stress and shear strain. Equation (14.12) can be written as

$$\tau(t) = G'(\bar{\omega})\gamma(t) \pm G''(\bar{\omega})\sqrt{\gamma_0^2 - \gamma(t)} \quad \textbf{(14.13)}$$

which defines an elliptical stress-strain relationship similar to that shown in Figure 14.19. The area of the ellipse indicates the energy dissipated by the viscoelastic material per unit volume and per cycle of oscillation. From Eq (14.13), it can be seen that the in-phase term $G'(\bar{\omega})$ represents the elastic stiffness component, and the out-of-phase term $G''(\bar{\omega})$the damping component. Rewriting Eq (14.13) as

$$\tau(t) = G'(\bar{\omega})\gamma(t) + \frac{G''(\bar{\omega})}{\bar{\omega}}\dot{\gamma}(t) \tag{14.14}$$

and comparing it with the equation of a single-degree-of-freedom system, the equivalent damping ratio of the VE material is obtained as

$$\xi = \frac{G''(\bar{\omega})}{\bar{\omega}}\left(\frac{\bar{\omega}}{2G'(\bar{\omega})}\right) = \frac{G''(\bar{\omega})}{2G'(\bar{\omega})} \tag{14.15}$$

Accordingly, $G'(\bar{\omega})$is defined as the shear storage modulus and $G''(\bar{\omega})$ is the shear loss modulus. The loss factor η is given as

$$\eta = \frac{G''(\bar{\omega})}{G'(\bar{\omega})} = \tan\delta \tag{14.16}$$

The loss factor is used as a measure of energy dissipation capacity of the VE damper.

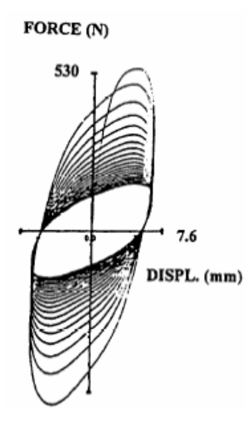


Figure 14.19 Eliptical force-displacement loops for VE dampers under cyclic loading

For a VE damper with a total shear area A and thickness h, the force-displacement relationship from equation (14.14) is given as

$$F(t) = k'(\bar{\omega})x(t) + c'(\bar{\omega})\dot{x}(t) \qquad \textbf{(14.17)}$$

where $k'(\bar{\omega}) = AG'(\bar{\omega})/h$ and $c'(\bar{\omega}) = AG''(\bar{\omega})/\bar{\omega}h$. Equation (14.17) indicates that unlike friction or yielding dampers, the VE dampers behave partly as an elastic material, which stores energy, and partly as a viscous material, which dissipates energy.

The shear storage and shear loss moduli are generally functions of the excitation frequency, shear strain, ambient temperature, and material temperature. The dependence of the VE dampers on these parameters has been studied analytically and experimentally. For the material temperature, one is interested in the temperature rise within the material over the loading history. Field observations and laboratory experiments have shown that, for wind and seismic excitations, the temperature increase in VE dampers is usually less than $10^\circ C$, and this has a minor effect on the performance of the dampers. For VE dampers undergoing moderate strains (less than 20%), the shear storage and shear loss moduli depend on the excitation frequency and the ambient temperature. In general, as the excitation frequency increases, $G'(\bar{\omega})$ and $G''(\bar{\omega})$ become larger. It was also found that VE materials soften and the effectiveness of the dampers decreases as ambient temperature increases. The loss factor η, however, remains relatively insensitive to moderate changes in frequencies and temperatures.

Several shake table tests of large-scale steel frame and reinforced concrete models with added VE dampers have been carried out by different investigators. In each study, VE dampers were found to significantly improve the response of the frame and reduce inter-story drifts and story shears. The experimental results, together with analytical models, have led to the development of design procedures for structures with supplemental VE dampers.

A design procedure for VE dampers has been outlined by Zhang and Soong (1992) and Chang et al.(1993) as follows

1. Analyze the structure without the dampers to determine the force demand/capacity ratios for the structural members;
2. Determine the target damping ratio ξ for which the building response remains elastic;
3. Using the modified strain energy method determine the added stiffness for the case of horizontal dampers $k_d = 2\xi k_s/(\eta - 2\xi)$ where k_s is the story stiffness without added dampers. If the dampers are located in the diagonal braces with an angle θ with the floor, the required stiffness is obtained by dividing k_d by $\cos^2\theta$;
4. Compute the required damper area $A = k'h/G'$ where h is determined such that the shear strain in the VE damper is lower than the ultimate value;
5. Determine the number, size, and location of the dampers;

6. Check the strength of structural members that are part of the damper bay assembly using a damper force of 1.2 times the maximum;
7. Perform a response spectrum analysis to determine the demand/capacity ratios of the structural members. If the ratios are greater than one, include more dampers;
8. Check interstory drifts to ensure that they are within the allowable limits; and
9. Perform a non-linear dynamic analysis of the damped structure. Check overall structural stability and strains in the dampers.

14.3.4 Viscous Dampers

Dampers, which utilize the viscous properties of fluids, have been developed and used in structural applications. A viscous-damping (VD) wall system was developed by Sumitomo Construction Company, Japan. The device consists of an outer steel casing attached to the lower floor and filled with a highly viscous fluid. An inner moving steel plate hanging from the upper floor is contained within the steel casing, Figure 14.20. The viscous damping force is induced by the relative velocity between the two floors. Earthquake simulator tests of a full scale 4-story steel frame with and without VD walls indicate response reductions of 66 to 80 percent with the walls. A 4-story reinforced concrete building with VD walls was constructed in 1987 in Tsukuba, Japan and has since been monitored for earthquake response. Reductions in acceleration responses between 33 to 75 percent were observed when using the VD walls. The 170 VD walls installed in the 78 m high SUT steel building in Shizouka City, Japan, provided 20 to 35 percent damping for the building and reduced the response up to 70 to 80 percent (Miyazaki and Mitsusaka, 1992).

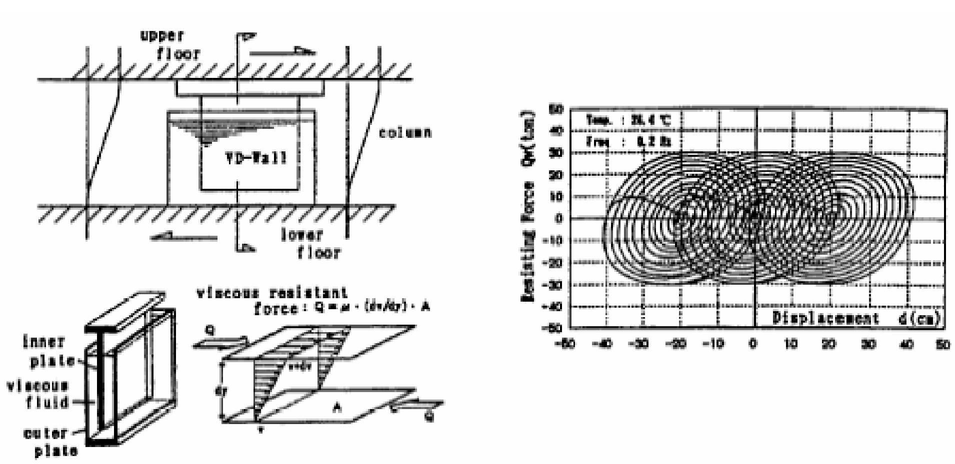


Figure 14.20 Viscous damping wall and hysteresis loops (Miyazaki and Mitsusaka, 1992)

Fluid viscous dampers which operate on the principle of fluid flow through orifices have been used for many years in automotive, aerospace, and defense industries. They are beginning to emerge in structural applications. These dampers possess linear viscous behavior and are relatively insensitive to temperature changes. Experimental and analytical studies of buildings and bridges with fluid dampers manufactured by Taylor Devices, Inc. have been carried out by Constantinou and Symans (1992). The Taylor device, which is filled with silicone oil, consists of a stainless steel piston with a bronze orifice head and an accumulator, Figure 14.21. The flow through the orifice is compensated by a passive bi-metallic thermostat that allows the operation of the device over a temperature range of $40^{\circ}C$ to $70^{\circ}C$.. The force in the damper is generated by a pressure differential across the piston head. The fluid volume is reduced by the product of travel distance and piston rod area. Since the fluid is compressible, the reduction in volume causes a restoring force, which is prevented by the accumulator.

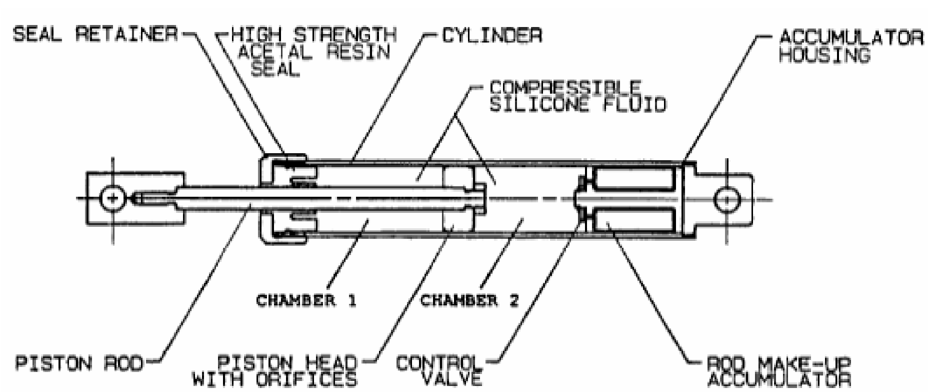


Figure 14.21 Construction of fluid viscous damper (Constantinou and Symans, 1992)

The force P in the fluid damper may be expressed as

$$P = \overline{C}|\dot{u}|^{\alpha}\,\frac{\dot{u}}{|\dot{u}|} \qquad \textbf{(14.18)}$$

where $\dot{u}$ is the velocity of the piston rod, $\overline{C}$ is a damping constant, and α is a coefficient ranging from 0.5 to 2.0. An $\alpha = 0.5$ is effective in attenuating high velocity pulses similar to those encountered in the near fault earthquake excitations. An $\alpha = 2.0$, achieved with cylindrical orifices, is unacceptable in applications involving high velocity excitations. An $\alpha = 1$ results in dampers with linear viscous behavior which is desirable in applications of seismic energy dissipation. Damping forces with α less than 2.0 require specially shaped orifices to alter the flow characteristics with the fluid speed.

The suitability of fluid dampers for seismic applications was studied by Constantinou and Symans (1992). Fluid dampers with an orifice coefficient $\alpha = 1$ were tested over the temperature range $0^{\circ}C$ to $50^{\circ}C$. Reducing the temperature from $24^{\circ}C$ to $0^{\circ}C$ increased the damping coefficient by 44% whereas increasing it from $24^{\circ}C$ to $50^{\circ}C$ decreased the damping coefficient by 25%. The change in damping properties over a wide temperature range indicates that, unlike VE dampers, fluid dampers are less sensitive to temperature changes and show stable behavior over a wide temperature range. Shake table tests of structures with fluid dampers have shown reductions in story drifts of 30% to 70%, which are comparable to those achieved by other energy dissipating systems such as friction, metallic and VE dampers. The use of fluid dampers, however, reduced the story and base shears by 40% to 70% because of their pure viscous behavior while other passive systems were not as effective in reducing base shears. Another desirable feature of fluid dampers is that the damping force is out-of-phase with the displacement. If dampers are included in a structure in such a way that they have an inclined force; for example, along a diagonal brace, the horizontal component of the damper force is out-of-phase with the displacement and therefore, the peak induced column moments are less than those when the peak damper force occurs at peak displacement. On the other hand, fluid dampers have the following disadvantages: a) maintaining seals over a long time; and b) small motions in the structure may cause seals to wear and fluid to leak out.

14.3.5 Retrofit Design using Dampers

A simplified design procedure (SDP) has been developed by Lee et al. (2005) for seismic response control and / or retrofitting of frame buildings with visco-elastic or elastomeric structural dampers. The SDP uses elastic-static analysis and is applicable to structural dampers. In SDP visco-elastic / high damping materials are idealized as linear visco-elastic material although behaviour of these materials are actually nonlinear. This simplified analysis method involving suitable practical design implementation, have been validated by comparing the computed responses with that obtained from nonlinear dynamic time history analysis.

<u>Simplified Design Procedure</u>

The SDP for multi-degree-of-freedom (MDOF) frame systems with VE or high-damping elastomeric dampers uses elastic-static analysis results to determine damper properties so that the damped frame satisfies specified seismic performance objectives, and has the following steps:

Step (1): Establish target seismic performance and associated design criteria. Seismic performance levels, such as *operational* or *immediate occupancy* under the design basis earthquake (DBE) are often the objective of using dampers. To achieve the target performance level, detailed design criteria, such as storey drift limits and limits on the internal forces of members under the DBE, are established in Step (1).

Step (2): Idealize nonlinear damping material as linear VE material. In general, VE or elastomeric damping materials have frequency, temperature, and strain amplitude-dependent behaviour. To keep the SDP simple, these non-linear damping materials are idealized as a linear VE material. Constant elastic shear modulus G' and loss shear modulus $G'' = \eta_d G'$, where η_d is the loss factor define the linear VE model. Usually, G' and η_d are estimated from harmonic loading test results using two criteria: (1) the maximum stress and strain are similar and (2) the hysteresis loop area is similar. The tests are conducted at constant frequency, temperature, and strain amplitude, and G' and η_d are tabulated as a function of these parameters. For the SDP, the value of G' at the first-mode natural frequency of the damped frame, the design temperature, and the expected strain amplitude is used in the linear VE model. The dependence of η_d on these parameters is typically small, and using the *minimum value* of η_d over a frequency range near the first-mode natural frequency and within a design temperature range will provide sufficiently accurate and conservative results. Figure 14.22 compares the experimental UHDNR hysteresis loops for 50% strain amplitude at 20°C and 0.5 Hz with the equivalent linear VE hysteresis loop. The agreement between loops is acceptable for the SDP. The validity of using this idealization in the SDP will be demonstrated later in the paper.

Step (3): *Select design temperature range.* Since damping materials have temperature dependent behaviour, upper and lower bound ambient temperatures are selected to define the design temperature range.

Step (4): Select appropriate a (ratio of the brace stiffness per storey *in the global direction to the* storey *stiffness without dampers and* braces) value, one or more β (ratio of the *damper stiffness per storey in the global direction to the storey stiffness without dampers and braces) values, and damper locations.* The stiffness ratios α and β are often used to represent VE or elastomeric dampers in a preliminary design process. When the actual damper material properties are frequency dependent, the damper stiffness and the period of the damped structure, which are unknown in preliminary design, are coupled. Using the stiffness ratio β, instead of the actual damper stiffness based on the shear modulus and damper geometry (area and thickness), makes the design procedure nearly insensitive to the frequency dependence of the material. After selecting a β value (as discussed later), the damper geometry is determined at the end of the

SDP. A range of β values from 0.5 to 5 is recommended. A value of α between 10 and 30, which results in stiff braces to maximize the damper energy dissipation, is recommended.

Step (5): Perform elastic-static analysis. With a range of β values defined in Step (4), a series of elastic-static analyses are performed using the elastic-static analysis procedure (ESAP) discussed later.

Step (6): Compare structural response obtained in Step (5) with design criteria from Step (1).

Step (7): Select minimum β that satisfies design criteria and provides target seismic performance.

Step (8): Determine structural response at low-end temperature of design temperature range and compare structural response with design criteria. Given the typical temperature dependent behaviour of VE and high-damping elastomeric materials, the minimum β selected in Step *(7)* is assumed to represent the damper stiffness at the high-end temperature of the design temperature range where the damping material is most flexible. When the temperature decreases, the damped system behavior changes due to increased damper stiffness, so the structural response at the low-end temperature is investigated. The value of β_{low} (β at the low-end temperature) can be obtained from the value of β_{high} (β at the high-end temperature from Step (7)) by solving iteratively the following equation:

$$\beta_{low} = \frac{G'(low\text{ - end temperature, }\omega_{\mathrm{low}}(\beta_{low}))}{G'(high - end\text{ temperature, }\omega_{\mathrm{high}}(\beta_{high}))}\beta_{high} \tag{14.19}$$

where G' (temperature, ω) is the elastic shear modulus of the damping material as a function of temperature and frequency, $\omega_{low}(\beta_{low})$ is the damper frequency at the low-end temperature as a function of β_{low}, and $\omega_{high}(\beta_{high})$ is the damper frequency at the high-end temperature as a function of β_{high}. To apply Equation (14.19), we assume that $\omega_{low}(\beta_{low})$ and $\omega_{high}(\beta_{high})$ are the first-mode natural frequencies of the damped system at the low-end and high-end temperatures, respectively. When β_{low} is determined & structural response at the low-end temperature is obtained from Step (5) results and compared with the design criteria as in Step *(6)*.

Step (9): Calculate damper area and thickness. The individual damper area, $(A_d)_i$ at each storey i equals $((K_d^e)_i \times t_d)/G'$(high-end temperature, $\omega_{high}(\beta_{high})$), where $(K_d^e)_i$ is the individual damper stiffness in the local direction and equals $\beta_{high}(K_0)_i/n_i \cos^2(\phi_d)_i$, i is the storey number, $(\phi_d)_i$ is the angle between the damper local direction and the global horizontal direction, n_i is the number of dampers at storey *i,* $(K_0)_i$ is the storey stiffness of the frame system without dampers and braces, and td is the thickness of the damper.

Elastic-static Analysis Procedure for Damped MDOF Frame Systems

Figure 19.23(a) shows a typical frame system with dampers and braces. The damper is connected with a brace in series, hereafter referred to as the damper-brace component. The analytical model for the ESAP uses elastic beam-column elements for the beams and columns of the frame system, and elastic truss elements for the damper-brace component. Figure 14.23(b) shows two models for the damper-brace component. The first is the linear VE damper-brace model, which has a complex (elastic and loss) stiffness K^*_{d+b}. The second is the simplified elastic-viscous damper-brace model with an elastic stiffness $K'_{d+b,sim}$ which is used in the ESAP, and a dashpot coefficient $C'_{d+b,sim}$. In the linear VE damper-brace mode K^*_{d+b} equals $K'_{d+b}(1+i\eta_{d+b})$, where K'_{d+b} is the elastic stiffness of the damper-brace component and η_{d+b} is the loss factor, as follows:

$$K'_{d+b} = \frac{1}{(1/K_b)+(1/\mu K'_d)} \qquad \eta_{d+b} = \frac{\eta_d}{\mu(1+(K'_d/K_b))} \qquad \mu = 1+\frac{\eta_d^2}{1+(K_b/K'_d)} \tag{14.20}$$

K'_d is the elastic stiffness of the damper in the global direction, η_d is the damper loss factor, and K_b is the brace stiffness in the global direction. K'_{d+b} in Eq. (3.12) is a function of μ, which is a function of η_d, and therefore K'_{d+b} is not purely elastic. A purely elastic stiffness for the damper-brace component, based on the simplified elastic-viscous model of Figure 14.23(b), $K'_{d+b,sim}$ is obtained by setting μ equal to 1, whereby

$$K'_{d+b,sim} = \frac{1}{(1/K_b)+(1/K'_d)} \tag{14.21}$$

The important differences between the results from the linear VE damper-brace model and the simplified elastic-viscous damper-brace model are corrected in Step (7) of the ESAP. The ESAP includes the following steps:

Step (1): Estimate first-mode deflected shape of damped frame system, $\underline{u}$ *by analysing the frame under a pattern of equivalent lateral forces,* $\underline{P}$.

Step (2): Estimate first-mode period, T_1*, using Rayleigh's method.*

Step (3): Estimate first-mode damping ratio, ξ_{eq}. The lateral force energy (LFE) method is used, and therefore the equivalent damping ratio, ξ_{eq}, equals $(\eta_d/2)(\underline{P}_d^T \underline{u}_d)(\underline{P}^T \underline{u})^{-1}$. $\underline{P}_d$ is the vector of forces in the dampers and $\underline{u}_d$ is the vector of damper deformations. Each damper deformation, $\underline{u}_d$, is separated from the damper-brace component deformation, u_{d+b}, and equals $(K_b/(K'_d + K_b))u_{d+b}$. $\underline{P}$ is the vector of equivalent lateral forces and $\underline{u}$ is the vector of corresponding floor lateral displacements, simulating the first-mode shape. The inherent damping of the undamped frame can be added to ξ_{eq}.

Step (4): Determine seismic coefficient from design spectrum. Tile seismic coefficient C_S, is a function of the first-mode period T_1, from Step *(2),* a response modification *R*, and other factors. The damping ratio from Step (3) is higher than that of the design spectrum (5% damping); thus, C_S, obtained from the design spectrum is divided by a damping reduction factor, such as B_S and B_1, from FEMA 356, 2000.

Step (5): Compute equivalent lateral forces. The equivalent lateral forces (ELF) are calculated as $\underline{Mu}(\Gamma.C_S/B_S \text{ or } \mathrm{B}_1).g)$ where $\underline{M}$ is the mass matrix, $\underline{u}$ is the vector of the first-mode floor lateral displacements, Γ is the participation factor, and *g* is the acceleration due to gravity.

Step (6): Perform static analysis under ELF to estimate displacements, internal forces, and deformations of damped frame system.

Step (7): Correct calculated response considering simplifications of analytical model. The simplified elastic-viscous model of the damper-brace component has only an elastic stiffness. As shown in Figure 14.22, the maximum stress for a linear VE model is the product of the magnitude of the complex modulus $|G^*|$ and the maximum strain. The hysteresis loop of the linear VE damper-brace model is similar to that shown in Figure 14.22, so the elastic damper force from the simplified elastic-viscous model (Figure 14.23(b)) underestimates the actual damper force. The magnitude of the complex stiffness $|K^*_{d+b}|$ of the linear VE damper-brace model equals $K'_{d+b}\sqrt{1+\eta^2_{d+b}}$. The damper forces from the elastic-static analysis should be amplified by *AF* x *MF* to account for the differences between $|K^*_{d+b}|$ and $K'_{d+b,sim}$, where *AF* is an amplification factor and *MF* is a modification factor. *AF* accounts for the difference between $|K^*_{d+b}|$ and K'_{d+b}, and equals $\sqrt{1+\eta^2_{d+b}}$. *MF* accounts for the difference between K'_{d+b} and $K'_{d+b,sim}$, and equals $\mu(\alpha+\beta)/(\alpha+\mu\beta)$. The column axial forces in the bays of the frame with the dampers are amplified by the same factors because they are in equilibrium with the damper forces. Member forces more closely related to lateral drift than to damper forces, such as the moment and shear in the beams and columns, are not amplified.

Steps (1) to (7) of the ESAP are repeated for each β value selected in Step (4) of the SDP. Note that Steps (1) and (2) of the ESAP could be replaced by an eigenvalue analysis, and Steps (4) to (6) of the ESAP could be replaced by a response spectrum analysis. However, the ESAP as presented above, uses only static analysis based on the widely used equivalent lateral force procedure. As a result, the ESAP is quite simple and easily applied in practice. The main advantage of the SDP and ESAP is that they verify seismic performance by comparing design criteria at both the global and member levels, without requiring an nonlinear dynamic time history analysi. The SDP and ESAP are illustrated below using the retrofit of a non-ductile reinforced concrete frame building as in example.

Retrofit Design of Non-ductile RC Frame Building

A retrofit design for a prototype non-ductile RC frame building with elastomeric dampers is used to illustrate the SDP. The three-storey, five-bay by five-bay non-ductile RC frame building (Figure 14.24(a) and (b)) is assumed to be built in the U.S. in the 1960s, before seismic loads were adequately considered in design. The target seismic performance of the RC frame building retrofit with elastomeric dampers is *operational* under the DBE. Therefore, the following retrofit design criteria are imposed. (1) the columns in the frame building should remain elastic when the building is subjected to the DBE, and (2) the storey drifts under the DBE are limited to 0.4%. Attention is focused on the columns rather than the beams to prevent a storey failure mechanism. The 0.4% drift limit is determined from inelastic analyses, where extensive inelastic behaviour in the first storey columns occurs after 0.4% drift. Figure 14.24(c) shows the damper and brace configuration in the damped RC frame building. An MDOF frame model was developed for the building. From symmetry of the building, only three frames are included in the model, as shown in Figure 14.24(c). Three dampers are located in each storey of one of the interior frames, referred to as the interior damped frame. The flexural stiffness and strength of the beams and columns were evaluated from moment-curvature analysis results. The SAP-2000 computer program was used for the ESAP.

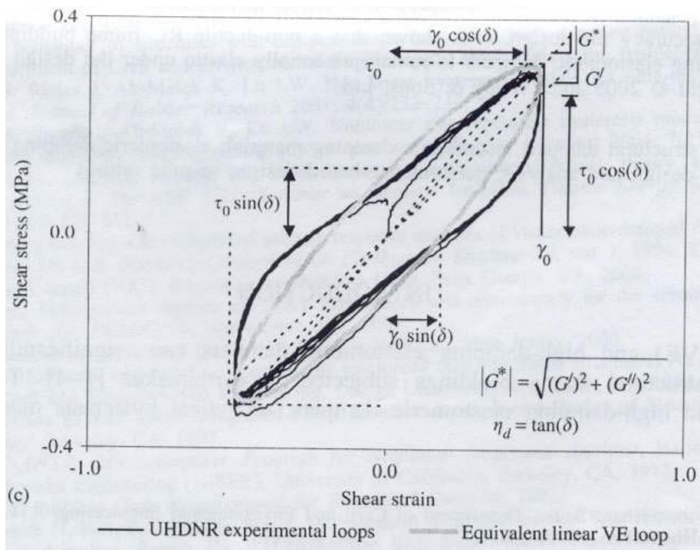


Figure 14.22 Ultra-high-damping natural rubber and equivalent linear VE hysteresis loops

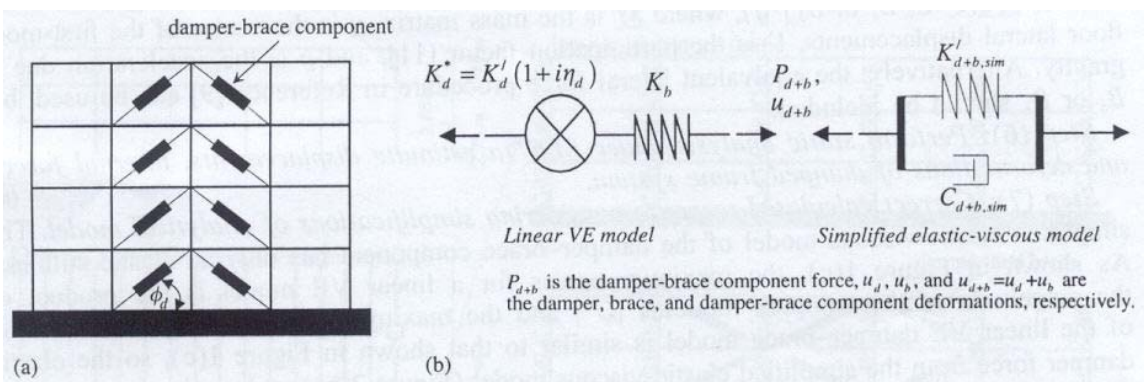


Figure 14.23 Structure with dampers and model for damper-brace component: (a) typical damper and brace configuration; and (b) models for damper-brace component

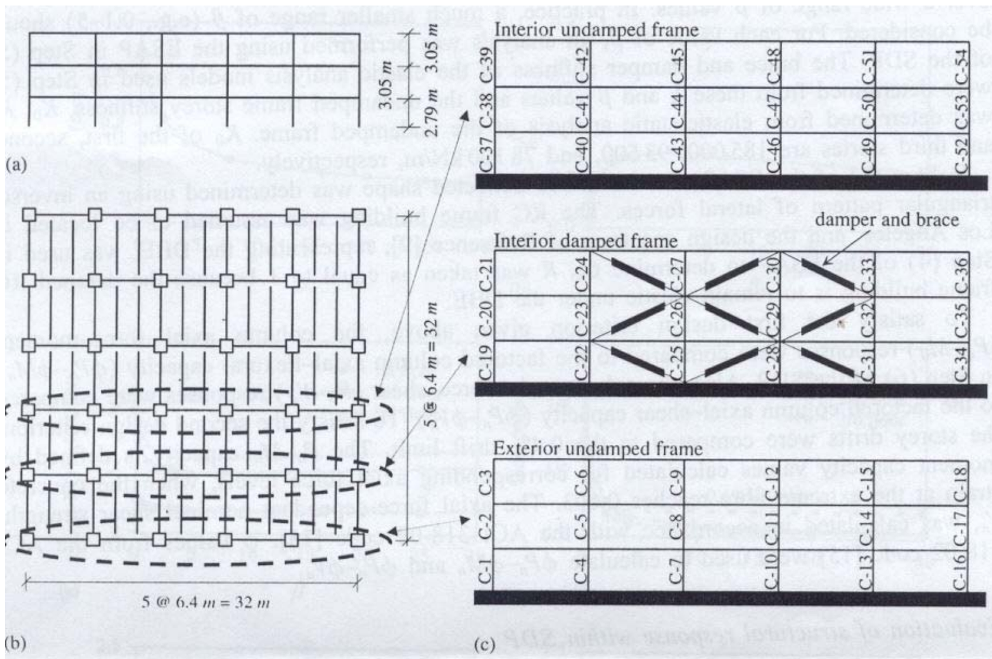


Figure 14.24 Prototype non-ductile RC frame building and analytical: (a) elevation view; (b) plan view (c) damper configuration and analytical model

14.4 TUNED SYSTEMS

This category of passive devices includes tuned mass dampers (TMD) and tuned liquid dampers (TLD). Tuned devices are relatively easy to implement in new buildings and in the retrofit of existing ones. They do not require an external power source to operate and do not interfere with vertical and horizontal load paths. Tuned systems may also be combined with active control mechanisms to function as hybrid systems, with the passive device serving as the back-up in case of failure of the active device. The following is a brief discussion of each device.

14.4.1 Tuned Mass Dampers

A typical tuned mass damper (TMD) consists of a mass, which moves relative to the structures and is attached to it by a spring and a viscous damper in parallel (Figure 14.25). When the structure vibrates, it excites the TMD and the kinetic energy is transferred from the structures to the TMD and is absorbed by the damping component of the device. The mass of the TMD usually experiences large displacements (stroke lengths).

A tuned mass damper is characterized by its mass, tuning and damping ratios. The mass ratio is defined as the TMD mass to that of the structure and the tuning ratio is defined as the ratio of the fundamental frequency of the TMD to that of structure. The optimum tuning and damping ratios that result in the maximum absorbed energy have been studied by several investigators. TMDs have been found effective in reducing response of structures to wind and harmonic loads and have been installed in several buildings. For seismic applications, there has not been a general agreement about the effectiveness of TMDs in reducing the response. Sadek et al. (1996) have studied the optimum parameters of tuned mass dampers for maximum reductions in response to earthquakes. For a given mass ratio, they determined the tuning and damping ratios of the TMDs that would result in approximately equal damping in the first two modes of vibrations. They found that that the equal damping ratios in the first two modes are greater than the average of the damping ratios of the lightly damped structure and the heavily damped TMD insuring that the fundamental modes of vibrations are more heavily damped. Sadek et al. (1996) used the method to select the parameters of TMDS for several SDOF and MDOF structures subjected to a number of earthquake excitations. The results indicate that using the optimum parameters reduces the displacement and acceleration responses significantly. They also showed that in order for TMDs to be effective, large mass ratios must be used, especially for structures with higher damping ratios. Thus, large mass of TMDs necessitates larger spaces to achieve the required damping effect. Further, performance of TMDs depends on frequency content of the earthquake ground motion.

14.4.2 Tuned Liquid Dampers

Tuned Liquid Dampers (TLD), which have been used extensively in space satellites and marine vessels, are being implemented in structures for vibration control. TLDs (Figure 14.26) consist of rigid tanks filled with shallow liquid, where the sloshing motion absorbs the energy and dissipates it through viscous action of the liquid, wave breaking, and auxiliary damping appurtenances such as nets or floating beads. The principle of absorbing the kinetic energy of the structure is similar to TMDs where the fluid functions as the moving mass and the restoring force is generated by gravity. TLDs have several advantages over TMDs such as reducing the motion in two directions simultaneously and not requiring large stroke lengths. On the other hand, the relatively small mass of water or other fluids compared to the large mass TMDs (usually steel, concrete, or lead) to achieve the same damping effect.

According to Sun et al. (1989), the natural frequency of TLDs can be computed from

$$\omega = \sqrt{\frac{\pi g}{2a}\tanh\left(\frac{\pi h}{2a}\right)} \qquad \textbf{(14.22)}$$

where *g, 2a*, and *h* are acceleration of gravity, tank length, and liquid height, respectively. The natural frequencies of TLDs are therefore easily adjusted by the dimensions of the tank. The governing equations of motion for a structure equipped with a TLD as well as their solutions can be found in Sun and Fujino (1994). TLDs are effective in reducing the response of structures to harmonic and wind excitations (Fujino et al., 1992). An example of the application of TLDs is the 149.4m high Shin Yokohama Prince Hotel in Japan with 30 TLD units attached to the top floor to suppress wind-induced vibrations. Shaking table tests of TLDs were carried out at Kyoto University to investigate their performance for seismic applications. A TLD was attached to the top floor of a 3-story steel frame and the frame was subjected to the NS component of the El Centro accelerogram from the 1940 Imperial Valley Earthquake scaled to a peak ground acceleration of $0.25\,m/s^2$. The results indicate that the TLD somewhat reduced the first mode response only, but it was not effective in reducing the total response.

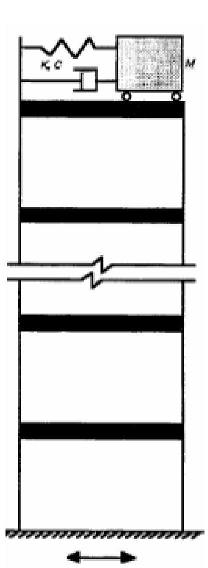


Figure 14.25 A building with a tuned mass damper

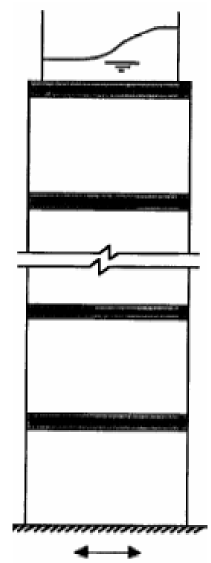


Figure 14.26 A building with a tuned column damper

14.5 SUMMARY

In this era of technological revolution, the world of seismic engineering is in need of creative thinking and advanced technology beyond conventional solutions. Seismic base isolation and energy dissipating devices are suitable technologies for earthquake resistant design / seismic retrofitting of a variety of buildings that have the requisite dynamic characteristics. Passive control technologies have matured in recent years to highly dependable and reliable level. Academic research on the subject is well advanced, and practical application are now being introduced throughout the world.

14.6 REFERENCES

1. UBC (1997), "Earthquake Regulations for Seismic Isolated Structures," Uniform Building Code, Appendix, Chapter –16, Whittier, CA.
2. FEMA 356 (2000), "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", American Society of Civil Engineers, USA.
3. Aiken, I.D., Kelly J.M. and Tajrican F.F., "Mechnics of low shape factor elastomeric seismic isolation bearings," Report No. UCB/EERC-89/13, University of California at Berkeley, 1989.
4. Aiken, I. D. and Kelly, J. M., (1990), "Earthquake simulator testing and analytical studies of two energy-absorbing systems for multistory structures," *Report No. UCB/EERC-90-03,* EERC, University of California, Berkeley, CA.
5. Aiken, I. D., Nims, D. K., and Kelly, J. M., (1992), "Comparative study of four passive energy dissipation systems," Bull. *N.Z Nat. Soc. for earthquake Engrg.,* Vol. 25(3), pp. 175-192.
6. Chang, K. C., Lai, M. L., Soong, T. T., Hao, D. D., and Yeh, Y. C., (1993), "Seismic behavior and design guidelines for steel frame structures with added viscoelastic dampers," *NCEER 93-0009,* National Center for Earthquake Engineering Research, Buffalo, NY.
7. Constantinou, M. C. and Symans, M. D., (1992a), 'Experimental and analytical investigation of seismic response of structures with supplemental fluid viscous dampers, *NCEER* 92-0032, National Center for Earthquake Engineering Research, Buffalo, NY.
8. Filiatrault, A. and Cherry, S., (1990), "A simplified design procedure for friction damped structures," *Proc. 4th U.S. Nat. Conf. Earthquake Engrg.,* palm Springs, CA., Vo.3, pp.479-488.
9. Fitzgerald, T. F., Anagnos, T., Goodson, M., and Zsutty, T., (1989), "Slotted bolted connections in aseismic design of concentrically braced connections," *Earthquake Spectra,* Vol. 5(2), pp. 383-391.

10. Fujino, Y., Sun, L. M., Pacheco, B. M., and Chaiseri, P., (1992), "Tuned liquid damper (TT-D) for suppressing horizontal motion of structures," *J. Engrg. Mech.,* ASCE, Vol. 118(10), pp. 2017-2030.
11. Kelly, J.M., "Shake table tests of long period isolation system for nuclear facilities at soft soil sites," *Report No. UCB/EERC-91/03,* University of California at Berkeley, 1991.
12. Lee, K., Fan, C., Sause, R., and Ricles, J. (2005), "Simplified design procedure for frame buildings with viscoelastic or elastomeric structural dampers," *Earthquake Engng Struct. Dyn.,* Vol.34, pp.1271-1284.
13. Miyazaki, M. and Mitsusaka, Y., (1992), "Design of a building with 20% or greater damping,"*Proc., 10th World Conf. on Earthquake Engrg.,* Madrid, Spain, pp. 4143.
14. Naeim, F. and Kelly, J.M.(1999), *Design of Seismic Isolated Structures,* John Wiley & Sons, Inc.
15. Pall, A. S. and Marsh, C., (1982), "Response of friction damped braced frames," J. *Struct. Engrg.,* ASCE, Vol. 108(6), pp. 1313-1323.
16. Richter, P. J., Nims, D. K., Kelly, J. M., and Kallenbach, R. M., (1990), "Me EDR energy dissipating restraint, a new device for mitigation of seismic effects," Proc., *SEAOC 59th Annual Convention,* Lake Tohoe.
17. Robinson, W. H. and Cousins, W. J., (1987), "Recent developments in lead dampers for base isolation," *Pacific Conf. on earthquake Engrg., Vol.* 2, N.Z.
18. Sadek, F., Mohraz, B., Taylor, A. W., and Chung, R. M., (1996), "A method of estimating the parameters of tuned mass dampers for seismic applications," *Report NIS77R* 5806, National Institute of Standards and Technology, Gaithersburg, MD.
19. Scholl, R. E., (1993), "Design criteria for yielding and friction energy dissipators," *Proc. ATC- 17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control,* Applied Technology Council, Redwood City, CA., Vol. 2, pp. 485495.
20. Skinner, R. I., Robinson, W. H., and MeVerry, G. H., (1993), *"An introduction to seismic isolation,"* John Wiley and Sons, NY.
21. Sun, L. M., Fujino, Y., Pacheco, B. M., and Isobe, M., (1989), "Nonlinear waves and dynamic pressure in rectangular TLD - Simulation and experimental verification," J. *Struct. Engrg. / Earthquake Engrg.,* Vol. 6(2), pp. 251-262.
22. Sun, L. M. and Fujino, Y., (1994), "A semi-analytical model for tuned liquid damper (TLD) with wave breaking," *J. Fluids and Structures, Vol.* 8, pp. 471-488.

23. Tsai, K. C. and Hong, C. P., (1992), "Steel triangular plate energy absorber for earthquake-resistant buildings," *Proc., 1st World Congress on Constructional Steel Design,* Mexico,

24. Tyler, R. G., (1985), "Further notes on a steel energy absorbing element for braced frameworks," *Bull. N.Z Nat. Soc. for earthquake Engrg.,* Vol. 18(3), pp. 270-279.

25. Whittaker, A. S., Bertero, V. V., Thompson, C. L., and Alonso, L. J., (1991), "Seismic testing of steel plate energy dissipation devices," *Earthquake Spectra, Vol.* 7(4), pp. 563-604.

26. Whittaker, A.S., Krumme, R., Sweeney, S., and Hayes, J.R.,(1995), "Structural control of buildings response using shape memory alloys," Report No. 95/22, USACERL, US Army Construction Engineering Research Laboratories.

27. Zhang, R. H., Soong, T. T., and Mahmoodi, p., (1989), "Seismic response of steel frame structures with added viscoelastic dampers," *Earthquake Engrg. Struct. Dyn.,* Vol. 18, pp. 389-396.

28. Zhang, R. H. and Soong, T. T., (1992), "Seismic design of viscoelastic dampers for structural applications," *J. Struct. Engrg.,* ASCE, Vol. 118(5), pp. 1375-1392.

15

QUALITY ASSURANCE AND CONTROL

15.1 QUALITY CONCERNS IN RETROFIT OF BUILDINGS

The quality requirements on buildings repair and retrofitting projects are often not given the same attention as for new projects. Since many of these projects are of smaller scope and performed by contractor organizations that are small in size, formal systems of QA/QC are not followed and implemented leading to poor quality.

The primary objective of this chapter to discuss the steps to be taken to ensure that retrofit of structures are conducted in a manner that confirms to contractual and regulatory requirements. Conformance of contractor's work to the requirements is verified on the basis of objective evidence of quality. The process control procedures and systems outlined herein describe how the quality assurance and control program is designed to ensure that all quality and regulatory requirements are recognized and that a consistent and uniform control of these requirements is adequately established and maintained. The success of the QA program depends on through understanding of its aims and its full implementation by all the parties involved in the repair and retrofit process.

The quality concepts have considerably changed during the last decades. These are discussed along the definitions of various terms used. The organizational setup required and function involved in achieving a quality product are then discussed. The quality requirements for the works are conveyed by the designer to the contractor through specifications and drawings. The correct type of specification has to be used in order to ensure that quality product is delivered in the most economical way. The various types of specifications are discussed. The typical quality control plan for the retrofitting of a column is then discussed.

Documentation constitutes a very important component of any quality plan. It is found that the documentation process is very weak in many projects in India. The various types of documents and document control are discussed. A few model documents for QC purpose are presented.

In order to achieve quality repair and retrofit work, temporary structures including scaffolding and formwork need to be properly designed and erected. The issues related to scaffolding, formwork and shoring are discussed. The safety of workers involved in the repair and retrofitting works is very important to improve quality and productivity in these projects. The safety issues and protection and life saving equipment are discussed.

15.2 QUALITY CONCEPTS AND DEFINITIONS

Project quality management includes the processes required to ensure that the project will satisfy the needs for which it was undertaken. It includes "all activities of overall management function that determine the quality policy, objectives, and responsibilities and implements them by means such as quality planning, quality assurance, quality control, and quality improvement, within the quality system" (PMBOK 2000).

15.2.1 Definition of Quality

Quality can be defined in terms of conformance to the agreed requirements of the customer and a product or service free of deficiencies. In the building construction industry, quality can be defined as meeting the requirements of the owner, designer, constructor, and regulatory agencies. In terms of function, a high quality-building project can be described by such terms as ease in understanding drawings, level of agreement in drawings and specifications, economics of construction, ease of operation, ease of maintenance, and energy efficiency (Arditi and Gunaydin 1999).

15.2.2 Quality Assurance

Quality Assurance (QA) is all planned and systematic actions necessary to provide adequate confidence that a structure, system, or component will perform satisfactorily and conform to project requirements. QA manager and staff are responsible for developing this program and for monitoring the activities within the QA program.

15.2.3 Quality Control

Quality control (QC) in construction typically involves ensuring compliance with minimum standards of material and workmanship in order to ensure the performance of the facility according to the design. These minimum standards are contained in the specifications. QC activities should encompass all phases of the project including design and construction.

15.2.4 Quality Audit

Quality Audit is defined as a systematic and independent examination to determine whether quality activities and related results comply with planned arrangements and whether there arrangements are implemented effectively and are suitable to achieve objectives (Calder 1997).

15.2.5 Total Quality Management

High quality of product and service and their associated customer satisfaction are key to survival for any enterprise. The nature of the current competition generally demands from any corporation the following four types of ability characteristics:

1. To understand what the customer wants and provide it, immediately on demand, at lowest cost.
2. To provide products and services of high quality and reliability consistently.
3. To keep up with the pace of change, technological as well as political and social.
4. To be one step ahead of the customer's needs; that is, to predict what the customer will want one year or ten years from now.

The attainment of those abilities requires an organised approach to management – an approach of managing for total quality, of managing for effectiveness and competitiveness, involving each and every activity and person at all levels of organisation. Total Quality

Management (TQM) is a culture advocating total commitment to customer satisfaction through continuous improvement and innovation in all aspects of the business.

15.2.6 Elements of Quality

Quality Characteristics

The elemental building blocks out of which “quality” is constructed are the quality characteristics A physical or chemical property, a dimension, a temperature or any other requirement used to define the nature of a product or a service is a quality characteristics. For each quality characteristic there is sequence of activities to achieve it. A designer specifies the requirements of the characteristics. The construction engineer specifies the process to be used to achieve the design specification requirements. The construction craftsmen and workers execute the work to make the construction facility per the design. The product / facility created are examined/tested to judge conformance to the design specifications.

Quality of Design

It is important- that the design professional identify for the owner those tasks which are necessary to assure quality, and to insist that these be included in the project's design phase costs. While it is a truism that additional rupees spent to assure quality during the design will reduce the cost of quality control measures during and after construction, the design professional should make this clear to the owner early rather than later. Owners, particularly inexperienced ones, expect quality in the completed project, but are unaware of the effort required to deliver quality both in the design phase and the construction that follows. The design professional must be aware of the probability of loss of quality in accelerated projects, fast-track, informal procedures, rejection of recommendations by professionals, budget cutting at midstream, and other management actions on the part of the owner. All too often, the design professional hopes for the best and accepts these impositions upon the design team's ability to perform and incorporate quality. It is the design professional's responsibility to point out the probable implications of such actions.

To establish a formal, design-related QA program other than that associated with the organization's standard office procedures, the first step is to designate a QA manager with authority to develop the QA program and QC activities for the office. One advantage of establishing a formal QA program is in the actual procedures development. The following procedures will assist in performing the work in an organized fashion (O’Brien 1989).

1. Review the project procedures manual. This document, prepared under direction of the project manager, describes requirements for activities related to performance on the project.

2. Review the written project program. This is also referred to as the basis for design or criteria and is prepared by each design discipline. It describes the owner's requirements, design parameters, codes, standards, materials, and design concepts. Design concepts include the basic solution of the system to meet the owner's requirements, constructability, and suitability of materials and systems.

3. Standard office procedures should be established to clearly define work to be checked, to identify the originator and the checker, and to determine approval requirements. In a formal QA program, a discipline check should be made of design calculations, drawings, specifications, probable construction cost estimates, and reports. Checking should be performed by qualified individuals not directly involved in design or supervision of the work.

4. Many projects contain drawings .or other documents with input from several disciplines. Procedures should be established for checking the composite document by each discipline.

5. An integrated design review requires an overall review of documents that have been prepared by various disciplines and will be issued as a single construction package. The purpose of the review is to provide coordination of the work performed by various disciplines and avoid conflicts in the documents.

Finally, the following may be performed before releasing the documents.

- Review by project manager
- Review by project design professional
- Final acceptance by the owner.

Quality of Construction

With the attention to conformance as the measure of quality during the construction process, the specification of quality requirements in the design and contract documentation becomes extremely important. Quality requirements should be clear and verifiable, so that all parties in the project can understand the requirements for conformance. Quality control in construction typically involves ensuring compliance with minimum standards of material and workmanship in order to ensure the performance of the facility according to the design. These minimum standards are contained in the specifications.

For the purpose of ensuring compliance, random samples and statistical methods are commonly used as the basis for accepting or rejecting work completed and batches of materials.

Rejection of a batch is based on non-conformance or violation of the relevant design specifications.

An implicit assumption in these traditional quality control practices is the notion of an acceptable quality level which is an allowable fraction of defective items. Materials obtained from suppliers or work performed by an organization is inspected and passed as acceptable if the estimated defective percentage is within the acceptable quality level. Problems with materials or goods are corrected after delivery of the product.

15.2.6 ISO 9000

The best known formal certification for quality improvement is the International Organization for Standardization's ISO 9000 standard. ISO 9000 emphasizes good documentation, quality goals and a series of cycles of planning, implementation and review.

This International Standard specifies requirements for a quality management system where an organisation:

a) needs to demonstrate its ability to consistently provide products that meet customer and applicable regulatory requirements, and

b) aims to enhance customer satisfaction through the effective application of the system, including processes for the continual improvement of the system and the assurance of conformity to customer and applicable regulatory requirements.

All requirements of this standard are generic and are intended to be applicable to all organisations, regardless of type, size and product provided.

This International Standard promoted the adoption of a process approach when developing, implementing and improving effectiveness of a quality management system, to enhance customer satisfaction by meeting customer requirements. The application of a system of processes within an organisation, together with the identification and interaction of these processes, and their management can be referred to as the ‘process approach’. An advantage of the process approach is the ongoing control that it provides over the linkage between the individual processes within the system of processes, as well as over their combination and interaction.

When used within the quality management system, such an approach emphasizes the importance of

a) understanding and meeting requirements,
b) the need to consider the processes in terms of added value,
c) obtaining results of processes performance and effectiveness, and
d) continual improvement of processes based on objective measurement.

The model of the process based quality management system shown in Figure 15.1 illustrates the process linkages. This shows that customer play a significant role in defining requirements as inputs. Monitoring of customer satisfaction requires the evaluation of information relating to customer perception as to whether the organisation has met customer requirements.

The methodology is also known as ‘Plan-Do-Check-Act’ (PDCA). The PDCA can be briefly described as follows:

- Plan: establish the objective and necessary to deliver result in accordance with customer requirements and the organization’s policies.
- Do: implement the processes.
- Check: monitor and measure processes and product against policies, objectives and requirements foe the product and report the results.
- Act: take actions to continually improve process performance.

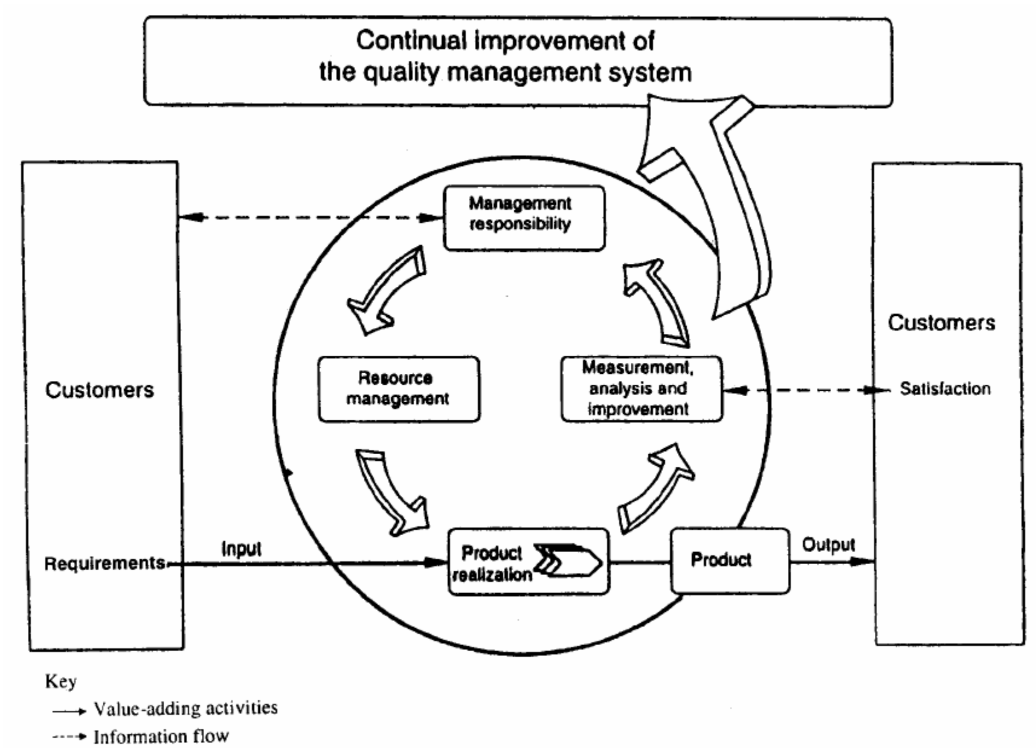


Figure 15.1 Model of a process based quality management system (IS/ISO 9001)

15.3 ORGANIZING FOR QUALITY

Quality assurance is the activity of providing the evidence needed to establish confidence, among all concerned, that the quality function is being effectively performed. Quality assurance provides protection against quality problems through early warnings of trouble ahead.

15.3.1 QA Plans

The quality assurance plan serves a number of purposes, and it must be designed so as to serve these functions:

1. Proof that policies and procedures have been thought out. “Before you can write it down you must first think it out”

2. A reference base as to policies and procedures as well as the whys and wherefores behind them. Those who use the plan will do a better job if they understand the reason behind their acts. The manual provides these reasons.
3. A textbook for training. The widest training use is for inspection and quality control personnel whose day to day work is guided by the QA plan. However, the training extends also to production supervisors, engineering personnel and others.
4. A precedent for future decisions. The plan includes a good deal of codification of agreements. Many quality standards appear in the plan, and these govern the cases often encountered in the past.
5. An aid to continuity of operations despite employee turnover. In the absence of a manual, a personnel change results in a change of practice - sometimes drastic. The plan helps to stabilize practice and to conduct operations based on “laws, not men”.
6. A reference base against which current practice can be audited.

The QA plan should address following elements:

a) Scope of the quality plan
b) Management responsibility
c) Quality system
d) Contract review
e) Design control
f) Document and data control
g) Purchasing
h) Control of customer supplied product
i) Product identification and traceability
j) Process control
k) Inspection and testing
l) Control of non conforming product
m) Corrective and preventive action
n) Control of quality records
o) Quality audits
p) Training

q) Servicing

r) Statistical technique

15.3.2 Field Administration

The basic building block of the organisation is the operation (also called “function,” “task,” “work element,” etc.). An operation is an identifiable type of activity such as assemble, transport or concreting. These operations in the context of quality management are referred as “quality management work elements”. These work elements are carried out by various departments in the organisation. The following list includes those principal work elements which have usually been assigned to such departments (Juran 1988):

1. Broad administration of the quality function

a) Draft quality policies.
b) Develop major quality objectives.
c) Develop plans for meeting quality objectives.
d) Develop the plans for meeting quality objectives.
e) Develop the organisation structure for carrying out the plans.
f) Conduct quality audits; prepare and issue summarized reports on quality.

2. Start up of new project

a) Study customer needs for quality oriented parameters.
b) Conduct design reviews for various quality oriented purposes: maintainability, etc.
c) Establish test programs to evaluate materials, processes, and products.
d) Conduct inspections and tests of trial production.
e) Estimate quality costs for processes and designs.

3. Supplier quality relations

a) Prepare supply relations manual: policies, methods, and procedures.
b) Prepare the plan for conducting supplier quality surveys.
c) Conduct surveys of prospective suppliers to judge quality capability.
d) Conduct inspection and supplier consignments.

4. Construction Procedure

a) Evaluate quality capability of processes.
b) Design plans for process control and control process surveillance.
c) Analyze out of control conditions; follow up to secure corrective action.

d) Design methods for evaluating quality performance of construction process output.

5. Inspection and Test:

a) Design the inspection and task plan.
b) Prepare supplementary standards and criteria as needed, standardize test procedures.
c) Prepare inspection manuals, systems and procedures.
d) Prepare inspection job specifications; recruit, select and train inspectors.
e) Conduct inspections and tests in accordance with plan.
f) Investigate causes of common defects, report findings, follow up for corrective action.
g) Initiate action to dispose of nonconforming product.
h) Prepare and report summaries of results of inspection in appropriate ways: by product, by component, by process, by department responsible, etc.

6. Customer Relations

a) Analyze customer quality complaints, recommend corrective action.
b) Evaluate customer experience with facility performance.
c) Identify customer needs for quality oriented assistance; develop responsive plans.

7. Training

a) Analyze costs of poor quality, identify major opportunities for improvements.
b) Stimulate companywide approaches for quality improvement.
c) Design and conduct training courses in various quality oriented skills and tools.

15.3.3 Field and laboratory testing

The contract plans and specifications should be checked for any testing requirements, sampling frequency, acceptance criteria and tolerances. Easy checklist should be developed to assist the construction inspectors in assessing conformance to all testing requirements and to ensure proper record keeping. Following are the steps required to develop QC procedures for testing and inspection:

a) Study the plans and specifications to identify all testing and inspection requirements for the project.

b) Assemble relevant contract documents needed to determine standards to be met for each test or inspection.

c) Develop any necessary checklists and train inspection staff.

d) Monitor for compliance to specified standards to be met according to the plans and specifications for the following: field tests & inspection, laboratory tests, receiving inspections and final testing & inspection.

e) Record results of required tests, inspections and observations in a timely manner on standard forms.

Steps required for maintenance of testing equipment:

a) Establish a calibration and maintenance program for all testing equipment used at work site under the control of field staff. The program may include specific contractual requirements, national standards, or manufacturer's recommendations
b) Document actions taken to calibrate and maintain testing equipment used and controlled by the work site inspection staff.

15.4 WORK AND MATERIAL SPECIFICATIONS

15.4.1 Specification Requirements

Specifications of work quality are an important feature of any new construction or retrofit designs. Specifications of required quality of components represent part of the necessary documentation to describe a facility. Typically, this documentation includes any special provisions of the facility design as well as references to generally accepted specifications to be used during construction. Construction specifications normally consist of a series of instructions or prohibitions for specific operations.

Well-written specifications are essential for the efficient construction of a successful project. Well-written specifications inform the contractor of the work to be performed, the conditions and restrictions on performance of the work, the expected quality of the work, and the manner in which the work will be measured for payment.

Well-written specifications:

- are clear, concise, and technically correct.
- do not use ambiguous words that could lead to misinterpretation.
- are written using simple words in short, easy to understand sentences.
- use technically correct terms, not slang or "field" words.
- avoid conflicting requirements.
- do not repeat requirements stated elsewhere in the contract

- state construction requirements sequentially.
- avoid the use of awkward phrases such as “and/or”,

15.4.2 Types of Specifications

Specifications are of following types:

1. Method only specification
2. End result only
3. Method and end result

Method Only Specification

A Method Specification spells out exactly the equipment, methods, materials, and techniques a contractor will be required to use. The Contractor or Producer is directed to combine specified materials in definite proportions and use specific types of equipment and methods in order to place the materials or product in a prescribed way.

The primary disadvantages of method specifications include:

- The Agency controls each step of the Contractor's operation.
- The Contractor may not be allowed to use the most economical or innovative procedures and equipment to produce the product sought.
- Materials Acceptance is based on inspection for "substantial conformance".
- Decisions based on test results of individual Field Samples can increase disputes and confrontation between the Contractor and Agency.
- Contractor payment is not linked to product quality or long-term performance.

End Result Only Specification

This type of specifications stipulates only required end result. It does not specify the method for achieving the end results. It shows that the owner is interested solely in the end result. Contractor is permitted to select his own methods which may be substantially less expensive and more effective.

Method and End Result Specification

This type of specifications specifies method to be used and stipulates the end results to be achieved. For most type of the projects this is not a satisfactory specification. It does not permit a constructor to make use of methods which he has found to be economical and effective. If the specified method could not achieve end result, the contractor should not be held responsible.

15.5 QUALITY CONTROL PLAN FOR TYPICAL RETROFITTING WORKS

Quality assurance plan need to be developed for various retrofit works including earthwork, masonry, concrete, steel construction. Typical flow diagram for QC of column repair using FRP composites is shown in Figure 15.2 for illustration purpose.

As can be seen from the diagram, the quality control activities include materials control, construction control, documentation, and adjustment of the procedures based on information collected and confidence gained on the level of quality. Similar flow diagrams can be developed for retrofitting activities.

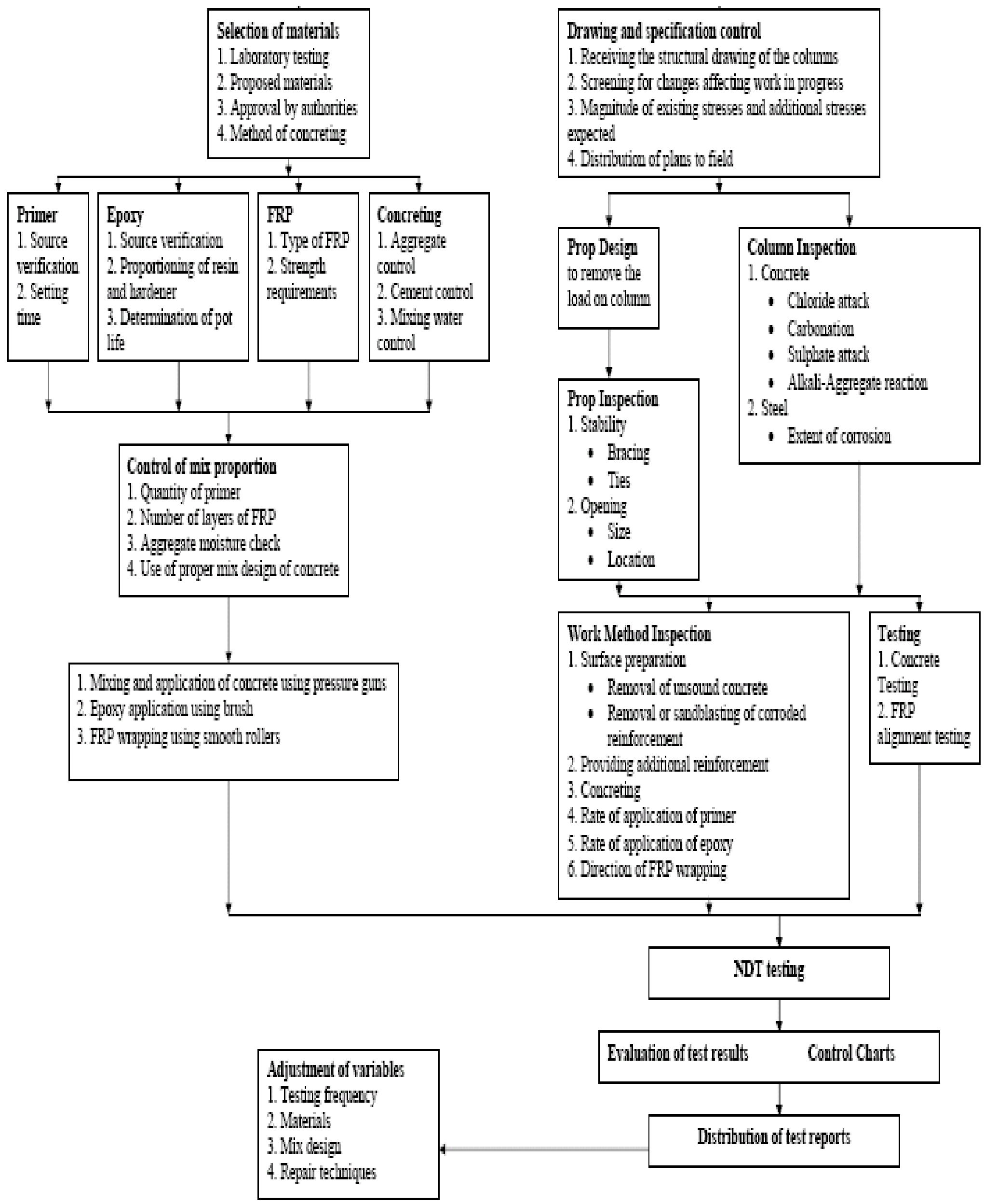


Figure 15.2 Flow Diagram for Quality Control for Column Repair

15.6 DOCUMENTATION

Documentation enables communication of intent and consistency of action. Its use contributes to:

a) Achievement of conformity to customer requirements and quality improvement,
b) provision of appropriate training,
c) repeatability and traceability,
d) provision of objective evidence, and
e) evaluation of effectiveness and continuing suitability of the quality management system.

Generation of documentation should not be an end in itself but should be a value adding activity used for control and improvement.

15.6.1 Documentation Types

The following types of documents are used in quality management systems:

a) documents that provide consistent information, both internally and externally, about the organization's quality management system; such documents are referred to as quality manuals;

b) documents that describe how the quality management system is applied to a specific product, project or contract; such documents are referred to as quality plans;

c) documents stating requirements; such documents are referred to as specifications;

d) documents stating recommendations or suggestions; such documents are referred to as guidelines;

e) documents that provide information about how to perform activities and processes consistently; such documents can include documented procedures, work instructions and drawings;

f) documents that provide objective evidence of activities preformed or results achieved; such documents are referred to as records.

Each organisation determines the extent of documentation required and the media to be used. This depends on factors as the type of organisation, the complexity and interaction of processes, customer requirements, the applicable regulatory requirements, the demonstrated ability personnel, and extent to which it is necessary to demonstrate fulfillment of quality management system requirements.

15.6.2 Document Control

Documents required by the quality management system shall be controlled. A documented procedure shall be established to define controls needed:

a) to approve documents for adequacy prior to issue,
b) to review an update as necessary and re approve documents,
c) to ensure that changes and the current revision status of documents are identified,
d) to ensure that relevant versions of applicable documents are available at points of use,
e) to ensure that documents of external origin are identified and their distribution controlled,
f) to ensure that documents remain legible and readily identifiable and
g) to prevent the unintended use of obsolete documents, and to apply suitable identification to them if they are retained for any purpose.

Records are the special type of document and shall be established & maintained to provide evidence of conformity to requirements and of the effective operation of the quality management system. Records shall remain legible, readily identifiable and retrievable. A documented procedure shall be established to define the controls needed for the identification, storage, protection, retrieval, retention time and disposition of records.

15.6.3 Model Documents

Typical model documents used in quality management system for retrofitting works are shown below in Tables 15.1 to 15.4.

Table 15.1 Equipment and Instrumentation Report

Equipment/ Instrument	Procedure	Frequency of Calibration	Acceptance Criteria	Corrective Action (CA)	Person Responsible for CA	Reference No

Table 15.2 QA Management Reports

Type of Report	Frequency (Daily, Weekly, Monthly)	Projected Delivery Dates	Persons Responsible for Report Preparation	Report Recipients

Table 15.3 Special Personnel Training and Certification Report

Specialized Training – Course Title or Description	Training Provider	Training Date	Personnel Receiving Training	Location of Records and Certificates

Table 15.4 Removal of Concrete and Restoration of Section

Project No: Project Name:

Description	Yes	No	Not Applicable
1. Have the perimeters of existing spalls been identified?			
2. Are the limits of concrete removal for each have been identified in the plans?			
If yes, did the contractor remove any concrete beyond the identified areas?			
Did the contractor obtain the engineers approval to remove concrete beyond the identified areas?			
3. Have cracks within solid concrete greater than specified standards been epoxy injected?			
4. After removal of defective areas, did the contractor inspect and clean the substrate from dust, grease, curing compounds?			
5. Has all exposed steel has been clean prior to concrete placement?			
6. Did the contractor used approved materials and method of application including manufacturers technical specification?			

15.7 TEMPORARY CONSTRUCTION

Temporary structures, including scaffolding, personnel protection systems, work platforms, trestles, temporary support systems for existing infrastructure, and lifting, rigging, and material handling systems are critical elements of the overall construction plan.

The temporary structures used on a specific project are determined via a careful evaluation of the project scope of work and existing site conditions. Each project presents its own unique set of constraints and opportunities.

15.7.1 Scaffolding

As per the definition given in IS 4014 Part I (1967), a scaffold is a structure used in construction, maintenance, repair and demolition work and enables person to obtain access to and egress from and to perform work, or which enables material to be taken to any place where such a work is being performed. Also scaffolding is defined as a support structure placed on the exterior of a building that is being constructed or renovated in order to provide easy worker access to otherwise out-of-reach points It most often consists of a framework of interlocked metal or wooden tubes, holding upright long boards which support the actual workers.

A scaffold's frame is categorized into three major components: standards, ledgers, and transoms. Standards or upright tubes provide the actual vertical mass of the structure. Supports are secured at the bottom of the scaffolding by a firm base plate, which locks the tube in by means of a shank, and are further supported by periodic attachments to the wall of the structure itself.

Ledgers are long horizontal tubes which run in the same direction as the building's face and are laid in pairs to define the edges of the work surface. These derive additional support from the transoms, shorter tubes which extend from each ledger at a ninety degree angle to form a grid and provide the lateral support needed to hold up the work boards.

Transoms adjacent to the standards are known as main transoms, while those in between the main transoms are referred to as intermediate transoms. The boards themselves are made of seasoned wood. IS 2750 and IS 4014 Part I deals with specification of steel scaffoldings and steel tubular scaffoldings respectively. IS 4014 Part II deals with the safety regulations for steel tubular scaffoldings.

15.7.2 Formwork

Formwork is the surface, supports and framing used to define the shape of concrete until the concrete is self supporting. Formwork consists of the forms on which the concrete is poured, the supports to withstand the loads imposed by the forms and the concrete and any bracing added to ensure stability. All materials and equipment used in formwork construction must be fit for the intended purpose and meet design specifications.

When specifying the design of the formwork system, a designer of formwork must allow for all loads that can be expected to be applied during construction, including loads applied by (Formwork Code of Practice, 2006):

- the formwork deck, supporting members and formwork frames;
- any false decks that may be provided;
- concrete pouring techniques (eg. pump);

- the concrete pour which includes both the weight of the concrete and dynamic factors applied. The concrete pour rate and pour sequence must be specified;
- workers on the formwork deck and false decks;
- stacked materials;
- crane lifted materials on both the complete and incomplete formwork deck;
- An allowance for wind loading is particularly important for vertical forms; and
- environmental loads including forces due to water flowing around the formwork.

The practice of formwork design should cover important elements such as design verification and documentation, documentation control, work program, mock up, and training of formwork installation.

15.7.3 Shoring

Excavating is recognized as one of the most hazardous construction operations. Shoring system means a structure such as a metal hydraulic, mechanical or timber shoring system that supports the sides of an excavation and which is designed to prevent cave-ins. Shoring is the provision of a support system for trench faces used to prevent movement of soil, underground utilities, `roadways, and foundations. Shoring or shielding is used when the location or depth of the cut makes sloping back to the maximum allowable slope impractical. Shoring systems consist of posts, wales, struts, and sheeting.

15.8 SAFETY

With an ever increasing awareness of construction jobsite safety and the cost of workmen's compensation claims, owners, contractors, and architect/engineers are realizing the benefits of planning construction work early in the life of a project, and supporting that plan with a dedicated project constructability program and an engineered approach to project execution. The construction engineers assigned to the project play critical roles in the implementation of these programs.

In all repair and retrofit works the safety of the workers should be given utmost importance. Safety and good health of workers results in productivity and quality which leads to greater workmen motivation.

15.8.1 Safety Training

The overall aims and objectives of Health and Safety Training induction include:

1. To ensure that safety requirements are appreciated by all the employees of the company
2. To enable a person to identify the hazard that he or his co-workers are exposed to and then control them.
3. To enable a person to seek positive improvements in his own health and safety through education.

Construction involves various activities and each activity is different. A worker working on scaffolding should be trained to work on it, similarly a person working at height should have gone through a fall protection training program so that he knows how to protect himself or rather avoid such an incidence.

15.8.2 Personal Protective and Life Saving Equipment

Hazards exist in every workplace in many different forms: sharp edges, falling objects, flying sparks, chemicals, noise and a myriad of other potentially dangerous situations. Controlling a hazard at its source is the best way to protect workers. Building a barrier between the hazard and the employees is an engineering control; changing the way in which workers perform their work is a work practice control. When engineering, work practice and administrative controls are not feasible or do not provide sufficient protection, employers must provide personal protective equipment (PPE) to their employees and ensure its use. Personal protective equipment, commonly referred to as "PPE", is equipment worn to minimize exposure to a variety of hazards. Examples of PPE include such items as gloves, foot and eye protection, protective hearing devices (earplugs, muffs), hard hats, respirators and full body suits. A good guiding principle is to select PPE that will provide a level of protection greater than the minimum required to protect employees from hazards.

<u>Eye and Face Protection</u>

Workers can be exposed to a large number of hazards from flying particles, molten metal, liquid chemicals, acids or caustic liquids, chemical gases or vapors, potentially infected material or potentially harmful light radiation that poses danger to their eyes and face. Many occupational eye injuries occur because workers are not wearing any eye protection while others result from wearing improper or poorly fitting eye protection.

Selecting the most suitable eye and face protection for employees should take into consideration the following elements:

a. Ability to protect against specific workplace hazards.
b. Should fit properly and be reasonably comfortable to wear.
c. Should provide unrestricted vision and movement
d. Should be durable and cleanable.
e. Should allow unrestricted functioning of any other required PPE.

Some of the most common types of eye and face protection includes Safety spectacles, goggles, welding shields and face shields.

Head Protection

Protecting employees from potential head injuries is a key element of any safety program. A head injury can impair an employee for life or it can be fatal. Wearing a safety helmet or hard hat is one of the easiest ways to protect an employee's head from injury. Hard hats can protect employees from impact and penetration hazards as well as from electrical shock and burn hazards.

Foot and Leg Protection

Situations in which an employee should wear foot and/or leg protection include:

a. When heavy objects such as barrels or tools might roll onto or fall on the employee's feet
b. Working with sharp objects such as nails or spikes that could pierce the soles or uppers of ordinary shoes
c. Exposure to molten metal that might splash on feet or legs
d. Working on or around hot, wet or slippery surfaces
e. Working when electrical hazards are present.

Foot and leg protection choices includes leggings, metatarsal guards, toe guards, combination foot and shin guards and safety shoes

Hand and Arm Protection

Potential hazards include skin absorption of harmful substances, chemical or thermal burns, electrical dangers, bruises, abrasions, cuts, punctures, fractures and amputations. Protective equipment includes gloves, finger guards and arm coverings or elbow-length gloves.

Hearing Protection

Employee exposure to excessive noise depends upon a number of factors, including:

a. The loudness of the noise as measured in decibels (dB).
b. The duration of each employee’s exposure to the noise.
c. Whether employees move between work areas with different noise levels.
d. Whether noise is generated from one or multiple sources.

Generally, the louder the noise, the shorter the exposure time before hearing protection is required. For instance, employees may be exposed to a noise level of 90 dB for 8 hours per day (unless they experience a Standard Threshold Shift) before hearing protection is required. On the other hand, if the noise level reaches 115 dB hearing protection is required if the anticipated exposure exceeds 15 minutes. Some types of hearing protection includes single-use earplugs, pre-formed or molded earplugs and earmuffs

15.8.3 Hand and Power Tools

Hand and power tools are extensively used in repair and restoration works. These tools help us to easily perform tasks that otherwise would be difficult or impossible. However, these simple tools can be hazardous and have the potential for causing severe injuries when used or maintained improperly. Special attention toward hand and power tool safety is necessary in order to reduce or eliminate these hazards.

15.8.4 Demolition

Demolition work involves many of the hazards associated with construction. However, demolition incurs additional hazards due to unknown factors such as: deviations from the structure's design introduced during construction, approved or unapproved modifications that altered the original design, materials hidden within structural members, and unknown strengths or weaknesses of construction materials. To counter these unknowns, all personnel involved in a demolition project must be fully aware of these types of hazards and the safety precautions to take to control the hazards. The ANSI A10.6-1990 standard for demolition operations defines "demolition" as the dismantling, razing, or wrecking of any fixed building or structure or any part thereof. In looking at safety in relation to demolition sites and all workplace safety matters, the three think safe steps are to be remembered which are:

1. Spot the hazard
2. Assess the risk
3. Make the changes

The demolition sequence is usually in reverse order to construction.

- Are emergency procedures in place?
- Is first aid available on site?
- Is fall protection in place?
- Is there a management plan and procedures for hazardous substances?
- Is there direct supervision by a competent person?
- Is debris cleared at regular intervals?
- Is access/egress to work areas clear?
- Are exclusion zones marked and is unauthorized entry prohibited?
- Are floors not overloaded with debris and equipment; and are floors back-propped?
- Is there engineering support for induced collapse, back-propping of floors, pre-stressed concrete members and damaged buildings/structures?

REFERENCES

1. IS 4014 (Part 1): 1967, “Code of practice for steel tubular scaffolding Part 1 Definitions and materials”, Bureau of Indian Standards, New Delhi.
2. IS 4014 (Part 2): 1967, “Code of Practice for Steel Tubular Scaffolding - Part II : Safety Regulations for Scaffolding”, Bureau of Indian Standards, New Delhi.
3. IS/ISO 10005: 1995, “Quality Management - Guidelines for Quality Plans”, Bureau of Indian Standards, New Delhi.
4. IS/ISO 9001: 2000, “Quality Management Systems – Requirements”, Bureau of Indian Standards, New Delhi.
5. IS/ISO 9000: 2000, “Quality Management Systems - Fundamental and Vocabulary”, Bureau of Indian Standards, New Delhi.

6. IS 2750: 1964, “Specification for Steel Scaffoldings”, Bureau of Indian Standards, New Delhi.
7. SP 53: 1992, Hand operated hand tools - Safety code for the use, care and protection, Bureau of Indian Standards, New Delhi.
8. Arditi, D., and Gunaydin, H. M. (1999). ‘‘Perceptions of process quality in building projects’’, Journal of Management in Engineering, ASCE, 15(2), 43–53.
9. Chris Hendrickson (2003). Project management for construction – fundamental concepts for owners, engineers, architects and builders, Department of Civil Engineering, Carnegie Mellon University, USA. http://www.ce.cmu.edu/pmbook/ accessed on 10 May, 2006.
10. Calder, Doug A.J.(1997). “Construction quality auditing”, Journal of Management in Engineering, ASCE, 13(6), 26 – 28.
11. “Formwork Code of Practice”, 2006, Department of Industrial Relations, Queensland Government, Australia
12. Juran, J. M., ed. (1988), Juran’s quality control handbook, 4th Ed., McGraw-Hill, New York.
13. Logothetis, N. (2004), Managing for Total Quality: From Deming to Taguchi & SPC, Prentice Hall, New Delhi.
14. O’Brien, J. J. (1989), Construction inspection handbook, 3rd Ed., Van Nostrand Reinhold, New York.
15. PMBOK (2000), Project management body of knowledge, Project Management Institute, Pennsylvania, USA.

16

RETROFIT CASE STUDIES

16.1 OVERVIEW

Two case studies are provided in this chapter. The first case study is an example of a multi-storeyed reinforced concrete (RC) building. The material is presented under the following topics: rapid visual screening, data collection, preliminary evaluation, detailed evaluation and retrofit. The second case study is an example of a heritage masonry building. The material of this case study is in abeyance.

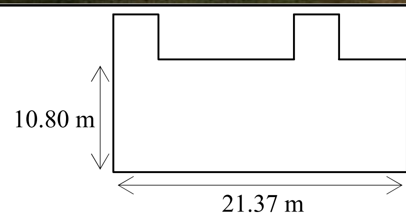
16.2 MASONRY BUILDING

The building is a single storeyed load bearing masonry building. It is one of several such bungalow type residential buildings built in Delhi by the British government during the early part of the 20^{th} century. At present these buildings are considered as heritage buildings and accommodate members of the parliament or senior government officials and their families. The building under study was evaluated for seismic resistance as required by IS 1893: 2002, and based on the provisions of IS 1905: 1987.

16.3 RAPID VISUAL SCREENING

The rapid visual screening (RVS) procedure as per Chapter 3 is presented for illustration. The survey identifies the building as Type C and the recommended action is to evaluate in detail for the need for retrofitting to achieve Type C^+ or D.

Seismic Zone IV Important Building



Building Name
Use: Residential
Address: No. 4, Kamaraj Lane, New Delhi
Other Identifiers
No. of Storeys: 1 Year Built: 1920s
Total Covered Area; all floors (sq. m): 310.8
Ground Coverage (sq. m): 310.8
Soil Type: Sandy clay Floor Type: Type II
Foundation Type: wall strip
Roof Type: Jack arch
Structural Components
Wall Type: BB Earthen UCR CCB
Thickness of Wall: 345mm Slab Thickness
Mortar Type: Mud Lime Cement
Vertical R/F Bars: Corners T-junctions Jambs
Seismic Bands: Plinth Lintel Eaves

OCCUPANCY	SPECIAL HAZARDS	FALLING HAZARDS
Important buildings: Hospitals, schools; emergency buildings; large community halls; power stations, VIP residences.	1.High water table and if sandy soil, liquefiable site indicated.	1.Chimneys ☐
		2.Parapets ☐
*Any building having more than 100 occupants may be treated as **important***	2. Land slide prone site	3.Cladding ☐
	3. Severe vertical irregularity	
Ordinary buildings: Other buildings having occupants < 100	4. Severe plan irregularity	4.Others √

Recommended action:
1) A, A^+ or B, B^+: evaluate in detail for need of reconstruction or possible retrofitting to achieve type C^+ or D.
2) C: evaluate in detail for need for retrofitting to achieve type C^+ or D.
3) Wood: evaluate in detail for retrofitting.
4) If any Special Hazard found, re-evaluate for possible prevention/retrofitting.
5) If any of the falling hazard is present, either remove it or strengthen against falling.

Surveyor's signature
Name:
Date:

Executive Engineers's signature:
Date of survey:

Probable damageability in few/many buildings

Building type	Masonry Building			
	A / A^+	B / B^+	C / C^+	D
Damageability in Zone IV	G5 / G4	G5 / G4	G4 / G3	G3

Figure 16.1 Rapid visual screening data sheet for masonry building(Zone IV, important building)

16.4 DATA COLLECTION

The building is considered as a heritage building. The building is a load bearing masonry structure. The walls are made of unreinforced burnt-clay brick masonry. The plinth level of the building is at 0.45 m above the ground level. Since the building was built prior to the development of seismic-resistant design guidelines, there are no beams or bands at the plinth, lintel or roof levels. The roof is made of lime concrete and is in the form of jack arches in between steel joists. The spaces above the arches are filled to have a flat roof. The roof is split into two levels. The two levels are at 3m and 5m height from the base of the building.

Figure 16.2 shows the site plan. Two photographs, detailed plan and two elevations are given in Figures 16.3, 16.4 and 16.5, respectively. Figure 16.6 shows a sectional elevation. Figure 16.7 shows typical cross-sections of the foundations of the walls.

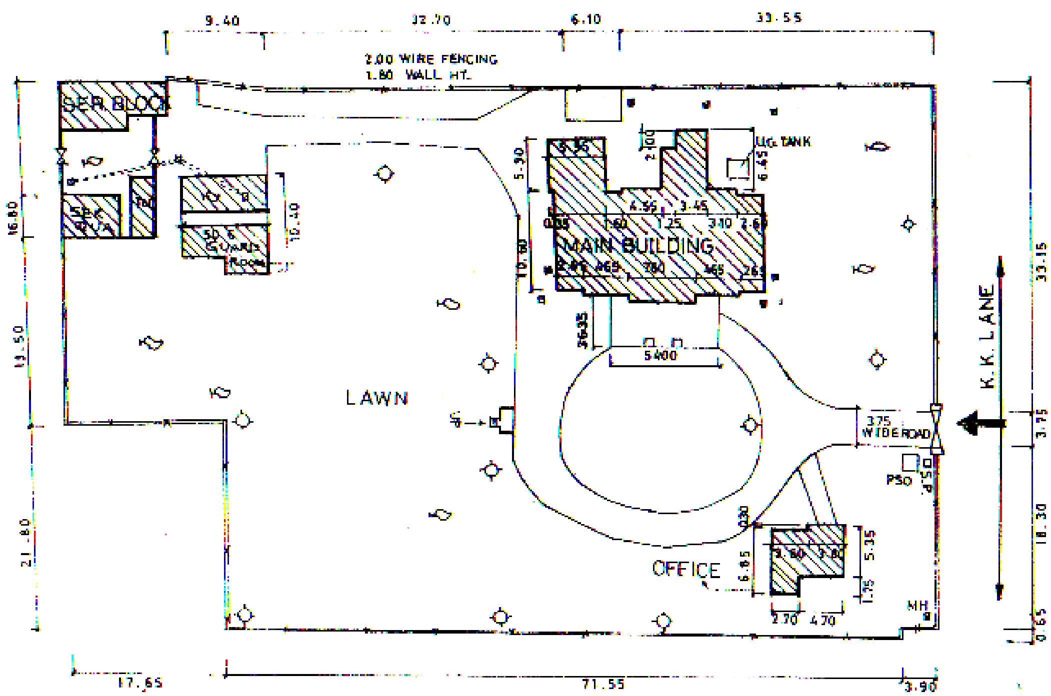


Figure 16.2 Site plan

a) Front view

b) Rear view

Figure 16.3 Photographs of the building

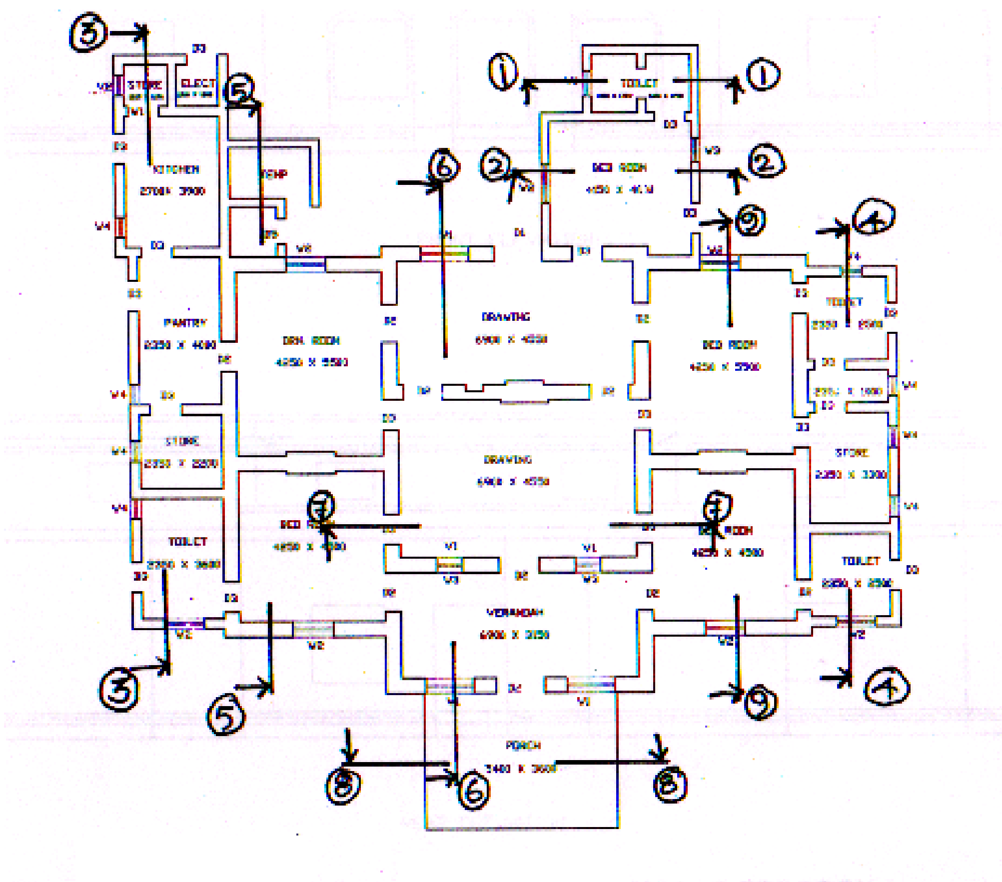
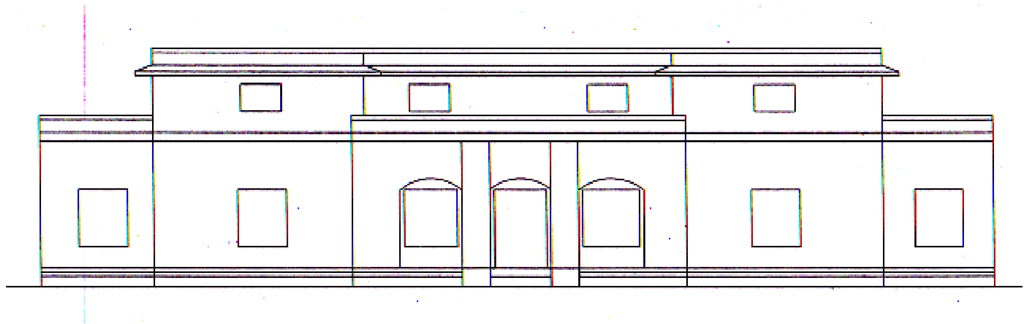
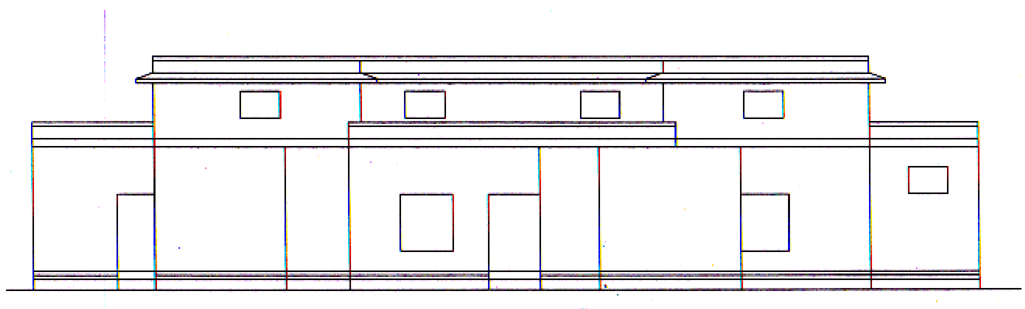


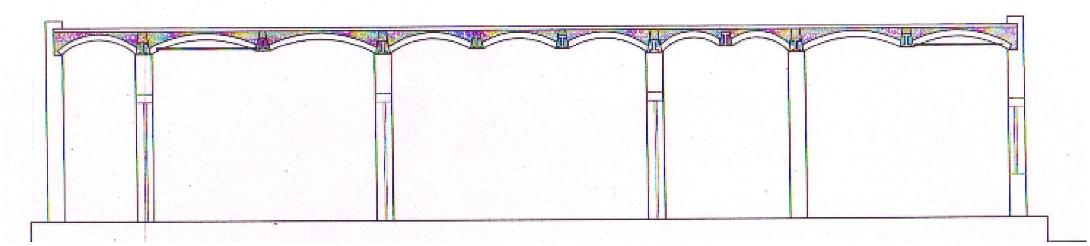
Figure 16.4 Plan of the building



a) Front

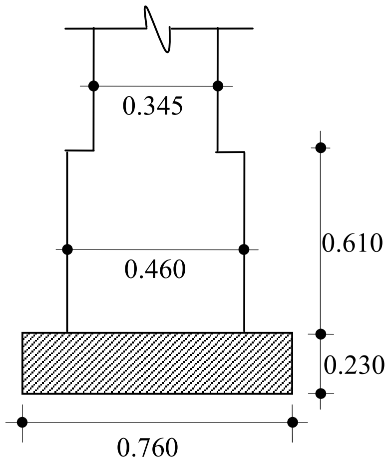


b) Rear



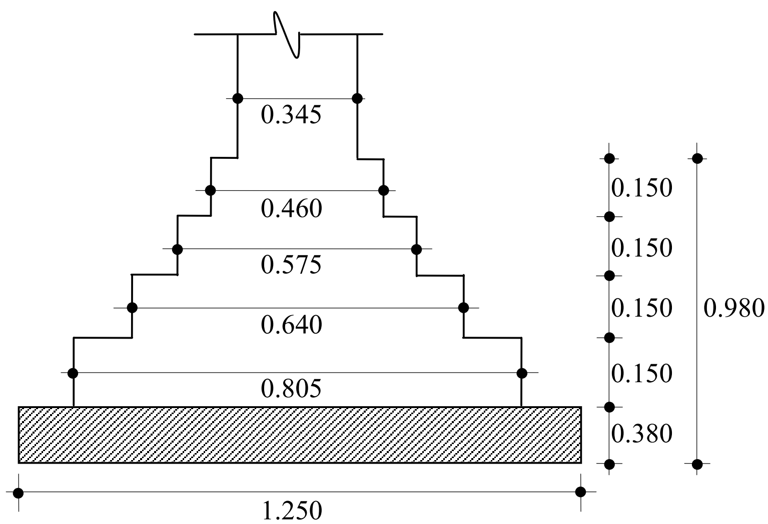
c) Elevation along Section 3-3

Figure 16.5 Elevations of the building



(a) Outer wall of verandah

(Dimensions are in meter)



(a) Main wall inside

(Dimensions are in meter)

Figure 16.6 Cross-sections of foundation

The data collection as per the procedure in Chapter 3 is shown in the following tables. Some of the information of the building was not available.

Table 16.1 Building survey data sheet: General data

S. No.	Description	Information	Notes
Building Description			
1	Address of the building • Name of the building • Plot number • Locality / Township • District • State	… 4, Kamaraj Lane, New Delhi	There is no specific name for the building
2	Name and type of owner / tenant	Government	Private / government
3	Name of builder	…	Not known
4	Name of architect	…	Not known
5	Name of engineer	…	Not known
6	Use of building	Residential	Residential / office / commercial / industrial
7	Year of construction and subsequent remodelling, if any	In 1920s	Subsequently weathering course laid with water proofing.
8	Plan size (approximate)	21.4m × 18.1m	
9	Building height	5 m	
10	Number of storeys above ground level	1	
11	Number of basements below ground level	0	
12	Type of structure • Load bearing wall • RC frame • RC frame and shear wall • Steel frame	Load bearing wall	
13	Open ground storey	~~Yes~~ / No	
14	Roof-top water tank, heavy machinery or any other type of large mass	Yes / ~~No~~	Water tank.

15	Architectural features	Sun-shades over windows and cantilever slab at portico	
16	Expansion / Separation joints	No	
17	Photograph / sketch	Attached	Attach with sheet
Survey			
1	Visited building site	Yes / ~~No~~	
2	Structural drawings available	~~Yes~~ / No	
3	Architectural drawings available	Yes / ~~No~~	
4	Geotechnical report available	~~Yes~~ / No	
5	Construction specifications available	~~Yes~~ / No	
6	Designer contacted	~~Yes~~ / No	
Exposure condition			
1	Environment	Hot in summer Dry, cold in winter	Hot / temperate / cold Dry / wet / humid Industrial / coastal etc.
2	Deterioration noticed	Aging and moisture patches	
Geotechnical and geological data			
1	Type of soil	Medium (assumed)	Soft / medium / hard rock IS 1893:2002, Table 1
2	Type of foundation	Wall strip footing	Isolated / combine footing / pile / raft
3	Seismic zone	IV	IS 1893: 2002, Figure 1
4	History of past earthquakes		

Variables for analysis			
1	Dead loads (unit weights) • Masonry • Concrete • Steel • Other materials	10 kN/m^3 25 kN/m^3	IS 875: 1987 (Part 1)
2	Imposed (live) loads • Floor loads • Roof loads	2.0 kN/m^2 1.5 kN/m^2	IS 875: 1987 (Part 2)
3	Wind loads	Not considered	IS 875: 1987 (Part 3)
4	Snow loads	Not considered	IS 875: 1987 (Part 4)
5	Safe bearing capacity	Not available	IS 1904: 1986
6	Importance factor, *I*	1.5	IS 1893: 2002, Table 6
7	Seismic zone factor, *Z*	0.24	IS 1893: 2002, Table 2
8	Response reduction factor, *R*	1.5	IS 1893: 2002, Table 7
9	Fundamental natural period, *T*	~0.1s for both directions	IS 1893: 2002, Cl. 7.6
10	Horizontal seismic coefficient (A_h)	0.2	IS 1893: 2002, Cl. 6.4
11	Seismic design lateral force	1343 kN	IS 1893: 2002, Cl.
12	Any other assumed data		

Table 16.2 Building survey data sheet: Building Data (load bearing masonry buildings)

S. No.	Description	Information	Notes
1	Building category	E	IS 4326: 1993, Section 7 Read with IS 1893: 2002 Building — Zones: II III IV V Ordinary: B C ~~D~~ E Important: C D E F
2	Type of wall masonry	Burnt-clay bricks	Age of brick not known, but more than 40 years.
3	Type and mix of mortar	Lime mortar	IS 4326: 1993, Cl. 8.1.2
4	Size and position of the openings • Minimum distance (b_5) • Ratio $(b_1+b_2+b_3)/l_1$ or $(b_6+b_7)/l_2$ • Minimum pier width between consecutive openings (b_4) • Vertical distance (h_3) • Ratio of wall height-to-thickness • Ratio of wall length between cross wall to thickness		IS 4326: 1993, Table 4, Figure 7.
5	Horizontal seismic band • At plinth level • At window sill level • At lintel level • At ceiling level • At eave level of sloping roof • At top of gable walls • At top of ridge walls	~~Yes~~ / No	 IS 4326: 1993, Cl. 8.4.6 IS 4326: 1993, Cl. 8.4.2 IS 4326: 1993, Cl. 8.4.3 IS 4326: 1993, Cl. 8.4.4
6	Vertical reinforcing bar • At corners and T-junction of wall • At jambs of doors and window opening	~~Yes~~ / No	 IS 4326: 1993, Cl. 8.4.8 IS 4326: 1993, Cl. 8.4.9
7	Integration of prefab roofing/flooring elements through reinforced concrete screed	~~Yes~~ / No	IS 4326: 1993, Cl. 9.1.4
8	Horizontal bracing in pitched truss • In horizontal plane at the level of ties • In the slopes of pitched roofs	Yes / No	

The thickness of the walls of the main portion of the building is 350 mm and that at the periphery is 230 mm. There are no visible cracks in the walls or roof. The roof weight got increased due to weather proofing. The weather proofing has been done along with drainage. The openings above the fire place have been closed. The following weaknesses of the building are noted.

Weakness 1: The building is more than 50 years old.

Weakness 2: The building does not have continuous lintel or plinth beams.

Weakness 3: The roof is made with jack arches and hence, does not have suitable diaphragm action. Moreover, the roof is split into two levels leading to diaphragm discontinuity.

Weakness 4: The cantilever sunshade slabs over the exterior doors and windows do not have proper anchorages.

16.5 PRELIMINARY EVALUATION

The preliminary evaluation was done as per the procedure in Chapter 3.

16.5.1 Strength-related Checks

Using the expression $T_a = 0.09\ h/\sqrt{d}$ (Section 7.6, IS 1893: 2002), the fundamental period of the building was calculated for both the X- and Y- directions as follows. The value of height (h) was taken up to the upper roof level.

Table 16.3 Calculation of time periods

	X	Y
h (m)	5	5
d (m)	21.37	18.15
T (s)	0.097	0.106

The spectral acceleration coefficient (S_a/g) corresponding to each of the periods is 2.5. This is available from the response spectrum for Type II soil (Figure 2, IS 1893: 2002). For a building in Zone IV, Z = 0.24 (Table 2, IS 1893: 2002). For an unreinforced load bearing masonry building, R = 1.5 (Table 7, IS 1893: 2002). Although it is an important building, since it does not accommodate large number of people I is taken as 1.0 (Table 6, IS 1893: 2002).

$$\therefore \text{ Horizontal seismic coefficient, } A_h = \frac{ZI}{2R}\left(\frac{S_a}{g}\right) = \frac{0.24\times1.0\times2.5}{2\times1.5} = 0.2$$

$\therefore$ Design seismic base shear, $V_B = A_h \times W = 0.2 \times 6716$ kN $\cong$ 1343 kN.

The value of W = 6716 kN is calculated based on the unit weights as explained later under Subsection Equivalent Static Analysis. Considering the age and the remaining life span of the building, an age factor of 0.6 is selected. Also, to consider the uncertainties of the material properties, a knowledge factor of 0.6 is applied. Thus, the final base shear is V_B × (age factor / knowledge factor) = 1343 kN.

Since there is only one storey, the full base shear is assumed to act in that storey. However, since the roof is split into two levels, namely at 3m and 5m, the base shear is distributed to the two levels as per Clause 7.7.1, IS 1893: 2002 (Table 16.4).

Table 16.4 Distribution of base shear over the height of the building

Roof level	Seismic Weight, W_i (kN)	Height, h_i (m)	Lateral Force, Q_i (kN)
1	4898.1	3	661.3
2	1817.6	5	681.7
		Total	1343.0

<u>Shear Stress in Masonry Walls</u>

The average shear stress is approximately calculated as $v_{avg} = V_j/A_w$ = 1343 kN/20m^2
= 0.07N/mm^2

16.5.2 Evaluation Statements

The evaluation statements under the relevant categories are presented together in Table 16.5.

Table 16.5 Evaluation statements

Building system	C / NC / NA
Load path:	C
Adjacent buildings:	C
Mezzanines:	C
Deterioration of concrete:	C
Masonry units:	–
Masonry joints:	–
Cracks in infill walls:	C
Cracks in boundary columns:	C
Post-tensioning anchors:	NA
Concrete wall cracks:	NA
Deterioration of steel:	C
Vertical irregularities	
No weak storey:	NC
No soft storey:	NC
No mass irregularity:	C
No vertical geometric irregularity:	C
No vertical discontinuities:	C
Plan irregularities	
No Torsion irregularity:	C
No diaphragm discontinuity:	C
Load bearing un-reinforced masonry walls	
Lateral load resisting system	

Redundancy:	C
Shear stress check:	
Proportions:	NC
Masonry lay-up:	NA
Diaphragms	
Diaphragm openings at walls:	
Diaphragm openings at exterior walls:	
Beam, girder and truss supports:	NA
Connections	
Wall anchorage:	NC
Transfer to walls:	NC
Geologic site hazards	
Liquefaction	–
Slope failure	NA
Surface fault rupture	–
Foundations	
Foundation performance:	C
Overturning:	C
Ties between foundation elements:	C

16.6 DETAILED EVALUATION

The detailed evaluation based on a linear analysis is done as per the procedure in Chapter 8. A few aspects of the computational model are first discussed.

16.6.1 Modelling of Walls

In order to model the wall segments as piers, the layout of the walls was identified based on grid lines as shown in Figure 16.7. The gravity load from the roof is distributed to the walls based on the tributary load distribution method which is conventional to most load bearing wall buildings.

The lateral stiffness of each wall segment is computed considering both bending and shear deformations, as explained in Chapter 6. The stiffness of a wall segment assumed to be fixed both at the top and bottom is given as follows.

$$k = \frac{1}{(\frac{H^3}{12EI}) + (\frac{1.2H}{GA})}$$

$$= \frac{Et}{(\frac{H}{L})^3 + 3(\frac{H}{L})}$$

In the above expression, the cross-sectional area of a segment $A = Lt$, the shear modulus $G = E/2(1 + \nu)$, the moment of inertia $I = tL^3/12$, H, L and t are the height, length and thickness of the segment, respectively, and ν is the Poisson's ratio taken as 0.2. The modulus of elasticity E is estimated to be 5500 MPa.

16.6.2 Modelling of Roofs

The roofs are not modelled explicitly. A roof made of jack arches on steel beams and filler material above the arches is assumed to be a rigid diaphragm. However, it is necessary to check the diaphragm forces to be within the limits.

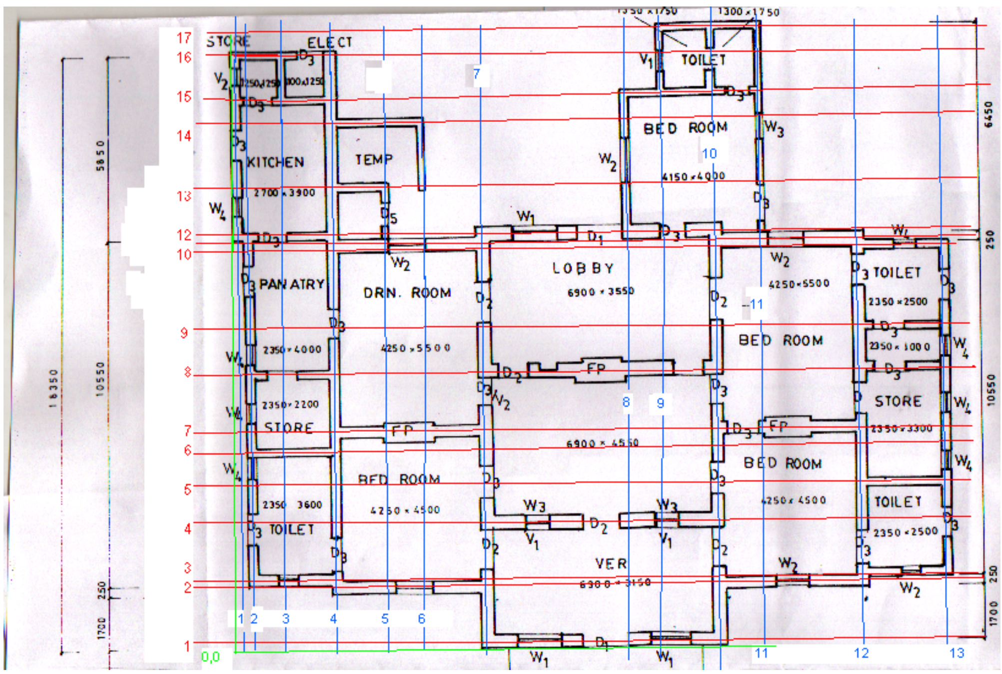


Figure 16.7 Plan showing grid lines

16.6.3 Equivalent Static Analysis

<u>Dead Load Data</u>

The average thickness of the jack arch roof and the unit weight are assumed as 200 mm and 25 kN/m^3, respectively.
$\therefore$ Dead load of roof = 5 kN/m^2.

For the walls with thickness 230 mm, the thickness including plaster is considered to be 250 mm. Assuming unit weight of bricks as 20 kN/m^3, the unit weight of 230 mm thick wall = 5 kN/m^2.

For the wall with thickness 460 mm, the thickness including plaster is considered to be 500 mm. $\therefore$ Unit weight of 460 mm thick wall = 10 kN/m^2

Live Load Data

Live load on roof = 1.0 kN/ m^2 (only for gravity load combinations).

Wind load is not considered.

Calculation of Seismic Weight

Total dead load at 5m level	1550.95 kN
Total dead load at 3m level	871.38 kN
Total wall load along north-south direction	2127.95 kN
Total wall load along east-west direction	2165.41 kN
Total =	6715.69 kN

Calculation of Base Shear

The total seismic weight of the building includes the weights of the slabs at two levels of roof and upper half of the walls. The weight of lower half of the walls is excluded. Live load on the roof is not considered. The calculations for the fundamental time period (T_a), the spectral acceleration coefficient (S_a/g), the horizontal seismic coefficient (A_h), the design seismic base shear (V_B) and the equivalent static force in each level (Q_i) are illustrated in the section of Preliminary Evaluation. Each force acts at the design centre of mass of the respective level.

Centres of Mass and Rigidity

The centre of mass (CM) and the centre of rigidity (CR) are calculated based on the procedure given in Chapter 3. A wall segment in a particular direction is assumed to resist horizontal load in its own plane. Any resistance to out-of-plane horizontal load is neglected. The wall segments are assumed to be fixed both at the top and bottom.

The value of the eccentricity for the design centre of mass (for each level) is calculated as per Clause 7.9.2, IS 1893: 2002. The torsion at each level is calculated as the product of the static force and the design eccentricity at that level.

Load Combinations

Three probable load combinations are considered. Each combination is associated with a potential failure mode.

Table 16.6 Load combinations

Load case	Load combination	Resultant action			Remarks
		Axial force	Moment	Shear	
1	DL + LL	$P_1 = P_D + P_L$	-	-	Result in large axial force
2	0.75(DL + LL+ EL)	$P_2 = 0.75(P_D + P_L + P_E)$	$M_2 = 0.75\ M_E$	$V_2 = 0.75V_E$	Result in large axial and bending force for lateral force
3	0.75(0.9 DL - EL)	$P_3 = 0.75(0.9\ P_D - P_E)$	$M_3 = 0.75\ M_E$	$V_3 = 0.75\ M_E$	Result in low axial load which causes low moment capacity

Calculation of Allowable Compressive and Tensile Stresses

The allowable stresses of the walls are calculated as per IS 1905: 1987. Considering the crushing strength of brick to be 35 MPa and the type of mortar as H1, the basic compressive stress for masonry (f_{bc}) = 2.5 N/mm^2 (Table 8). The stress is modified by the reduction factor for slenderness (k_s) (Clause 5.4.1.1) and the area reduction factor for small wall (k_a) (Clause 5.4.1.2). The shape modification factor (Clause 5.4.1.3) is taken as 1.

Thus, allowable stress under direct compression (F_a) = $k_s\, k_a\, f_{bc}$.

Allowable compressive stress under bending (F_b) = 1.25 F_a.

Allowable tensile stress under bending (F_t) = 0.07 MPa

When both direct compressive stress (f_a) and flexural compressive stress (f_b) act, the following interaction formula must be satisfied.

$$\frac{f_a}{F_a} + \frac{f_b}{F_B} \leq 1.0$$

The allowable stresses under direct compression and under flexure for typical wall segments are tabulated below.

Table 16.7 Allowable stresses

Wall segment	Slenderness ratio	k_s	k_a	F_a(MPa)	F_b (MPa)	F_t (MPa)
1	9.13	0.92	1.00	2.30	2.88	-0.07
2	5.22	1.00	1.00	2.50	3.13	-0.07
3	5.22	1.00	1.00	2.50	3.13	2.50
4	9.13	0.92	1.00	2.30	2.88	-0.07

<u>Check for Shear Stresses</u>

The shear force in each wall segment is calculated as the sum of the direct shear due to the lateral load (V_D) and the shear due to torsion (V_T), as explained in Chapter 6. The total shear stress is compared with the permissible shear stress (f_s). As per IS 1905: 1987, f_s is the least of a) 0.5 MPa, b) 0.1+0.2 f_d and c) 0.125 $\sqrt{f_m}$. Here, f_d is the direct compressive stress in the wall segment and f_m is the crushing strength of the bricks.

16.6.4 Evaluation Results

<u>Check for Stresses</u>

For each load case, the demand-to-capacity ratio (the stress demand divided by the allowable stress) for each wall segment is computed. Typical results are tabulated.

Load Case 1: (DL + LL)

The stresses under axial force for typical wall segments are compared with the allowable values under direct compression.

Table 16.8 Demand-to-capacity ratios for Load Case 1

Wall segment	P_1(kN)	f_{a1}(MPa)	DCR = (f_{a1}/F_a)
1	130.3	0.51	0.22
2	187.8	0.74	0.30
3	371.7	0.56	0.22
4	229.9	0.56	0.24

Load Case 2: 0.75(DL + LL + EL)

In this case, the compressive stresses in a wall segment due to both axial force and bending are first compared with the respective allowable values. Next, the combined action is checked using the interaction formula.

Table 16.9 Demand-to-capacity ratios for Load Case 2

Wall segment	P_2 (kN)	f_{a2} (MPa)	(f_{a2}/F_a)	M_2 (kN-m)	f_{b2} (MPa)	(f_{b2}/F_b)	(f_{a2}/F_a) + (f_{b2}/F_b)
1	313.9	1.26	0.55	33.2	0.70	0.24	0.79
2	264.4	1.06	0.42	52.2	1.13	0.36	0.78
3	309.7	0.46	0.19	187.2	0.58	0.19	0.38
4	418.4	1.17	0.51	85.9	0.67	0.23	0.74

Load Case 3: 0.75(0.9DL – EL)

In this case, first the compressive or tensile stress in a wall segment due to axial force is compared with the respective allowable values. Next, the tensile stress due to bending is combined with the stress due to axial force. The resultant stress is compared with the allowable value.

Table 16.10 Demand-to-capacity ratios for Load Case 3

Wall segment	P_3 (kN)	f_{a3} (MPa)	(f_{a3}/F_a)	M_3 (kN-m)	f_{b3} (MPa)	$f_3 = f_{a3} - f_{b3}$	(f_3/F_t)
1	-151.1	-0.60	8.57	33.2	0.70	1.30	18.57
2	-29.8	-0.12	1.71	52.2	1.13	1.25	17.86
3	154.8	1.23	0.09	187.2	0.58	0.35	5.00
4	-194.1	-0.47	6.71	85.9	0.67	1.16	16.57

16.6.5 Observations

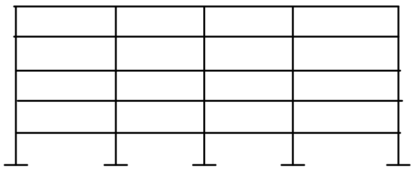
The observations from the detailed evaluation are summarised.
a) Load Case 3 generates the critical the demand-to-capacity.
b) In each of the wall segment, the demand-to-capacity ratio exceeds 1.0.
c) The building needs to be retrofitted.

16.7 REINFORCED CONCRETE BUILDING

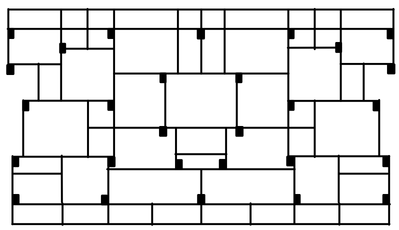
The building is a five storey residential building located in Zone V. The ground storey of the building is an open ground storey to accommodate car parking. Thus the deficiency due to open ground storey is highlighted. Two retrofit schemes are illustrated. In the first scheme, shear walls are placed at suitable locations throughout the height of the building. The second scheme involves addition of infill walls in the open ground storey and concrete jacketing of the ground storey columns.

16.8 RAPID VISUAL SCREENING

The rapid visual screening (RVS) was done as per the procedure in Chapter 3. The RVS data sheet is shown in Figure. 16.8. The results indicate the requirement of a detailed analysis.



Elevation to Scale



Plan to Scale

Building Name: –
Use: Residential
Address: –

Other Identifiers:
No. Stories: 5
Year Built: 1999
Total Floor Area (Sq.m): 351.5

Photograph

OCCUPANCY		SPECIAL HAZARDS	FALLING HAZARDS
Important buildings School, Health, Assembly, Office, Commercial, Historic, Emergency Service, Industrial. Ordinary buildings	Max. Number of Persons 0-10 11-50 51-100 >100 Residents: ~ 50 Floating:	High W T (Within 8m) Liquefiable (if sandy soil) Site. Land Slide Prone Site Severe Vertical Irregularity	Chimney, Parapets, Cladding other

Building Type	Masonry Building				R C or Steel Frame Building				URM Infill	Wood
	A /A$^+$	B / B$^+$	C / C$^+$	D	C / C$^+$	D	E / E$^+$	F		
Damage Grade in Zone V	G5 / G4	G5 / G4	G4 / G3	G3	G4 / G3	G3	G2 / G1	G1	G4	G4

Note: +sign indicates higher strength hence somewhat lower damage expected as 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:

1) A, A$^+$ or B, B$^+$: evaluate in detail for need of reconstruction or possible retrofitting to achieve type C or D.
2) C, C$^+$: evaluate in detail for need for retrofitting to achieve type D
3) URM infill: evaluate for need of reconstruction or possible retrofitting to level D
4) Wood: evaluate in detail for Retrofitting
5) In Case of Special Hazard, evaluate for possible

Surveyor's signature –
Name: –
Date: –

Figure 16.8 Rapid Visual Screening data sheet

16.9 DATA COLLECTION

The building is a RC framed structure. There are infill walls in all the storeys, except the ground storey. In the ground storey there are three infill walls only around the stair case. The wall thickness is 120 mm for both the exterior and interior infill walls. The floor plan is same up to fourth floor while at the roof level few beams are absent. The concrete floor slab is 150 mm thick at every level. The foundation system is pile foundation with groups of 4 or 6 under reamed piles. Each pile is of 300 mm diameter, reinforced with 6 longitudinal bars of 12 mm diameter and 6 mm diameter links at 175 mm centre-to-centre. The piles are more than 11m deep below the ground level. During the site visit no deterioration of the concrete or rusting of the steel bars or visible cracks was observed.

The beam layouts and designations for the first four floors and the roof are shown in Figures 16.9 and 16.10, respectively. Figure 16.11 shows the column layout and designations. Some of the columns have same sections. The size and reinforcement details for the beams and column sections (at the faces of the joints) are given in Tables 16.11 and 16.12, respectively. Figure 16.12 and Table 16.13 show the reinforcement details and the designated columns for the different column sections, respectively.

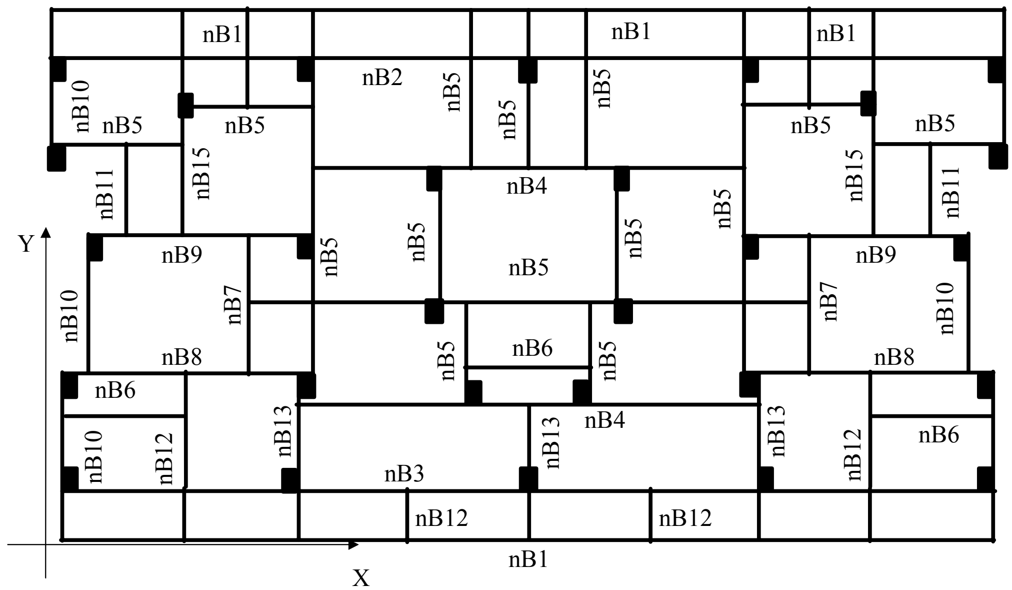


Figure 16.9 Floor framing plan showing beam locations and designations

(Prefix ‘n’ represents floor number)

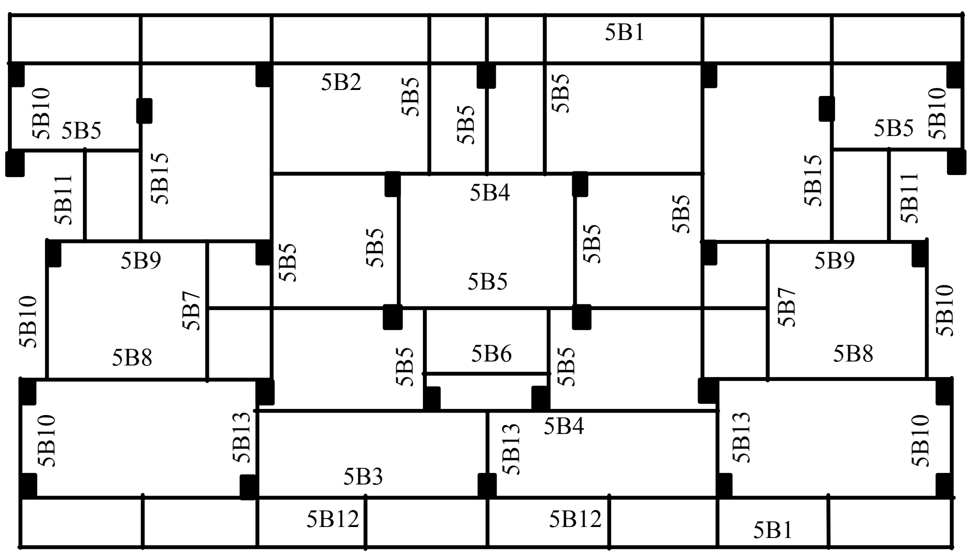


Figure 16.10 Roof framing plan showing beam locations and designations

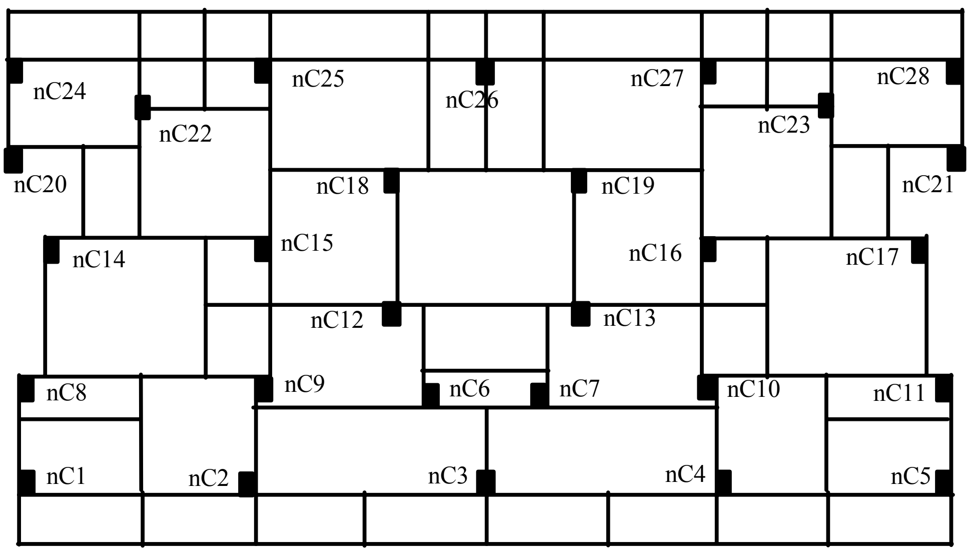


Figure 16.11 Floor framing plan showing column locations and designations

(Prefix 'n' represents storey below)

Table 16.11 Details of beam sections at column faces

Beam	Size (mm × mm)	Longitudinal Reinforcement		Transverse Reinforcement (spacing in mm)
		Top	Bottom	
nB1	150 × 500	2-#20	2-#20	2-#8 @ 75 c/c
nB2	300 × 500	6-#20, 1-#16	4-#20	2-#8 @ 75 c/c
nB3	200 × 450	6-#20	4-#20	2-#8 @ 75 c/c
nB4	250 × 500	4-#20, 2-#16	4-#20	2-#8 @ 75 c/c
nB5	250 × 500	4-#16	2-#16	2-#8 @ 75 c/c
nB6	150 × 500	2-#16	2-#16	2-#8 @ 75 c/c
nB7	300 × 500	6-#20	3-#20	2-#8 @ 75 c/c
nB8	250 × 500	4-#20, 2-#16	3-#20	2-#8 @ 75 c/c
nB9	250 × 500	2-#16	3-#16	2-#8 @ 75 c/c
nB10	150 × 500	3-#16	4-#16	2-#8 @ 75 c/c
nB11	250 × 450	6-#20	2-#20	2-#8 @ 75 c/c
nB12	250 × 500	4-#20	2-#20	2-#8 @ 75 c/c
nB13	250 × 500	7-#20	3-#20	2-#8 @ 75 c/c
nB14	300 × 500	6-#20	4-#20	2-#8 @ 75 c/c
nB15	300 × 500	4-#20, 2-#12	4-#20	2-#8 @ 75 c/c
nB16	300 × 500	4-#20, 2-#16	3-#20	2-#8 @ 75 c/c
nB17	200 × 450	4-#20, 2-#12	4-#20	2-#8 @ 75 c/c
nB18	250 × 500	4-#20, 2-#12	3-#20	2-#8 @ 75 c/c
nB19	300 × 500	4-#20, 2-#12	3-#20	2-#8 @ 75 c/c

Prefix ‘n’ in a beam number represents the floor number. The rebar designation is as per SP 34: 1987, *Handbook on Concrete Reinforcement and Detailing*. 2-#20 represents 2 numbers of 20 mm diameter deformed bars.

Table 16.12 Details of column sections at the beam faces

Column Section	Size (mm × mm)	Longitudinal Reinforcement	Transverse Reinforcement (spacing in mm)
C1	400 × 450	8-#20	#6 @ 100 c/c
C2	400 × 450	6-#20, 2-#16	#6 @ 100 c/c
C3	400 × 450	4-#20, 4-#16	#6 @ 100 c/c
C4	400 × 500	8-#20	#6 @ 100 c/c
C5	400 × 500	6-#20, 2-#16	#6 @ 100 c/c
C6	400 × 500	4-#20, 4-#16	#6 @ 100 c/c
C7	400 × 450	10-#20	#6 @ 100 c/c

C8	400 × 450	8-#20, 2-#16	#6 @ 100 c/c
C9	400 × 500	10-#20, 2-#16	#6 @ 100 c/c
C10	400 × 500	10-#20	#6 @ 100 c/c
C11	400 × 500	8-#20, 2-#16	#6 @ 100 c/c

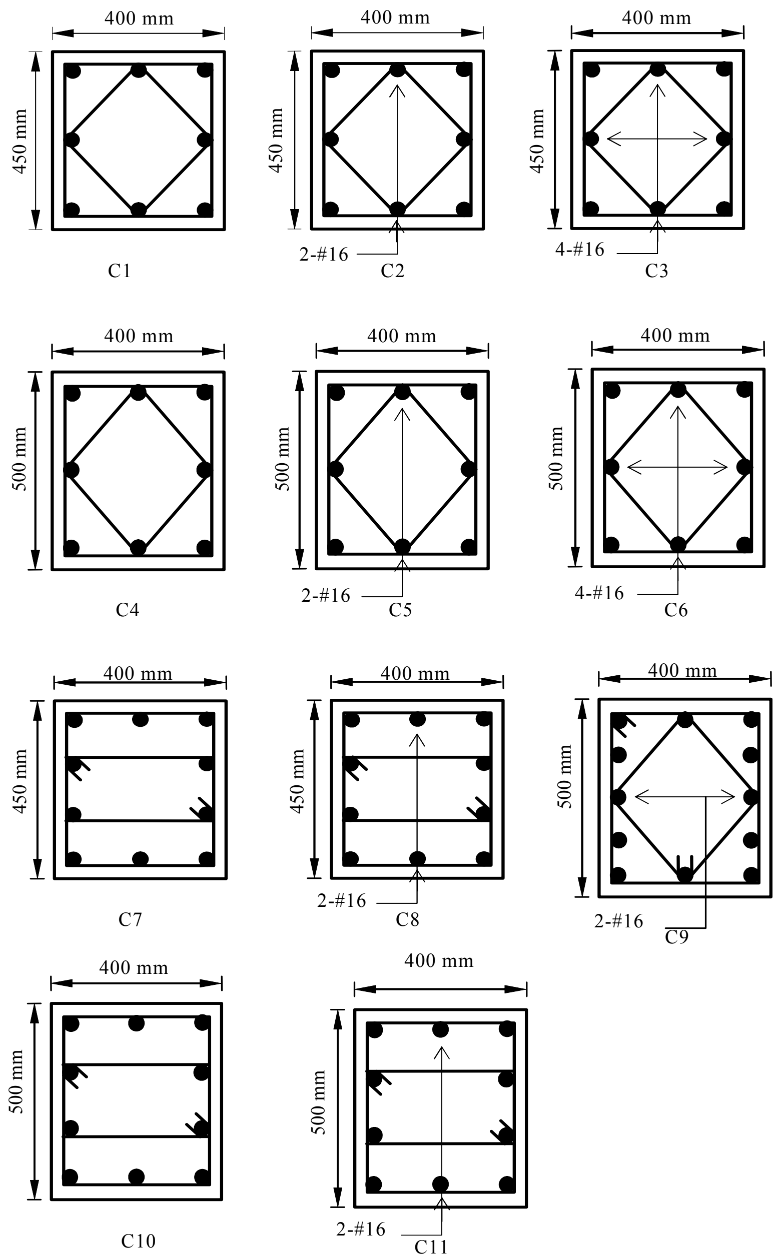


Figure 16.12 Reinforcement details of the columns at the beam faces (The diameter of a bar is 20 mm if not mentioned)

Table 16.13 Designated columns for the different column sections

Section	Column Number
C1	1C1, 1C5, 1C20, 1C26, 2C1, 2C5, 2C20, 2C26, 3C1, 3C5, 3C20, 3C26
C2	4C1, 4C5, 4C20, 4C26, 5C2, 5C3, 5C4, 5C21, 5C22, 5C23, 5C24, 5C25
C3	5C1, 5C5, 5C20, 5C26
C4	1C6, 1C11, 1C12, 1C17, 1C27, 1C28, 2C6, 2C11, 2C12, 2C17, 2C27, 2C28, 3C6, 3C11, 3C12, 3C17, 3C27, 3C28, 5C7, 5C8, 5C9, 5C10, 5C13, 5C16
C5	4C6, 4C11, 4C12, 4C14, 4C15, 4C17, 4C27, 4C28, 5C14, 5C15
C6	5C6, 5C11, 5C12, 5C17, 5C27, 5C28
C7	1C2, 1C3, 1C4, 1C21, 1C22, 1C23, 1C24, 1C25, 2C2, 2C3, 2C4, 2C21, 2C22, 2C23, 2C24, 2C25, 3C2, 3C3, 3C4, 3C21, 3C22, 3C23, 3C24, 3C25
C8	4C2, 4C3, 4C4, 4C21, 4C22, 4C23, 4C24, 4C25
C9	1C14, 1C15, 1C18, 1C19, 2C14, 2C15, 2C18, 2C19, 3C14, 3C15, 3C18, 3C19
C10	1C7, 1C8, 1C9, 1C10, 1C13, 1C16, 2C7, 2C8, 2C9, 2C10, 2C13, 2C16, 3C7, 3C8, 3C9, 3C10, 3C13, 3C16, 4C18, 4C19
C11	4C7, 4C8, 4C9, 4C10, 4C13, 4C16, 5C18, 5C19

The data collection as per the procedure in Chapter 3 is shown in the following tables. Some information of the building is not disclosed.

Table 16.14 Building survey data sheet: General data

S. No.	Description	Information	Notes
Building Description			
1	Address of the building • Name of the building • Plot number • Locality / Township • District • State	 … … … … …	
2	Name and type of owner / tenant	Private	Private / government
3	Name of builder	…	

4	Name of architect	…	
5	Name of engineer	…	
6	Use of building	Residential	Residential / office / commercial / industrial
7	Year of construction and subsequent remodelling, if any	1999	
8	Plan size (approximate)	25.2m × 13.9m	
9	Building height	15.7 m	
10	Number of storeys above ground level	5	
11	Number of basements below ground level	0	
12	Type of structure • Load bearing wall • RC frame • RC frame and shear wall • Steel frame	RC frame	
13	Open ground storey	Yes / ~~No~~	
14	Roof-top water tank, heavy machinery or any other type of large mass	Yes / ~~No~~	
15	Architectural features		
16	Expansion / Separation joints	No	
17	Photograph / sketch		Attach with sheet
Survey			
1	Visited building site	Yes / ~~No~~	
2	Structural drawings available	Yes / ~~No~~	
3	Architectural drawings available	~~Yes~~ / No	
4	Geotechnical report available	~~Yes~~ / No	
5	Construction specifications available	~~Yes~~ / No	
6	Designer contacted	~~Yes~~ / No	
Exposure condition			

1	Environment	Hot Wet	Hot / temperate / cold Dry / wet / humid Industrial / coastal etc.
2	Deterioration noticed	No	
Geotechnical and geological data			
1	Type of soil	Medium (assumed)	Soft / medium / hard rock IS 1893:2002, Table 1
2	Type of foundation	Pile	Isolated / combine footing / pile / raft
3	Seismic zone	V	IS 1893: 2002, Figure 1
4	History of past earthquakes	Major earthquakes in 1897 and 1950.	
Variables for analysis			
1	Dead loads (unit weights) • Masonry • Concrete • Steel • Other materials	10 kN/m^3 25 kN/m^3	IS 875: 1987 (Part 1)
2	Imposed (live) loads • Floor loads • Roof loads	2.0 kN/m^2 1.5 kN/m^2	IS 875: 1987 (Part 2)
3	Wind loads	Not considered	IS 875: 1987 (Part 3)
4	Snow loads	Not considered	IS 875: 1987 (Part 4)
5	Safe bearing capacity	Not available	IS 1904: 1986
6	Importance factor, *I*	1.0	IS 1893: 2002, Table 6
7	Seismic zone factor, *Z*	0.36	IS 1893: 2002, Table 2
8	Response reduction factor, *R*	3.0	IS 1893: 2002, Table 7

9	Fundamental natural period, T	0.28 seconds along X 0.36 seconds along Y	IS 1893: 2002, Cl. 7.6
10	Horizontal seismic coefficient (A_h)	0.15 for both X and Y	IS 1893: 2002, Cl. 6.4
11	Seismic design lateral force	2878 kN	
12	Any other assumed data		

Table 16.15 Building survey data sheet: Building Data (moment resisting frame)

S. No.	Description	Information	Notes
1	Type of building • Regular frames • Regular frames with shear wall • Irregular frames • Irregular frames with shear wall • Open ground storey	Regular frames with open ground storey	IS 1893: 2002, Cl. 7.1
2	Horizontal floor system • Beams and slabs • Waffle slab • Ribbed floor • Flat slab with drops • Flat plate without drops	Beams and slabs	
3	System of interconnecting foundations • Plinth beams • Tie beams	No inter-connection	IS 1893: 2002, Cl. 7.12.1
4	Grades of concrete used in different parts of building	M15	
5	Method of analysis	–	
6	Computer software used	–	
7	Base shear a) Based on approximate fundamental period b) Based on dynamic analysis c) Ratio of a)-to-b)	 2878 kN 1768 kN 1.63	IS 1893: 2002, Cl. 7.5.3
8	Distribution of seismic forces along the height of building	Parabolic	IS 1893: 2002, Cl. 7.7.1

9	Torsion included	–	IS 1893: 2002, Cl. 7.9
10	The columns of soft ground storey specially designed	–	IS 1893: 2002, Cl. 7.10
11	Clear minimum cover provided in • Footing • Column • Beams • Slabs • Walls	Not available	IS 456: 2000, Cl. 26.4
12	Ductile detailing of RC frame		
	• Type of reinforcement used	Fe 415	IS 456: 2000, Cl. 5.6
	• Minimum dimension of beams	150 mm × 150 mm	IS 13920: 1993, Cl. 6.1
	• Minimum percentage of reinforcement of beams at any cross section	1.072	IS 13920: 1993, Cl. 6.2.1
	• Spacing of transverse reinforcement at any section of beam	100 mm c/c	IS 13920: 1993, Cl. 6.3.5
	• Spacing of transverse reinforcement in 2d length of beam near the ends	75 mm c/c	
	• Ratio of shear capacity to flexural capacity of beams		IS 13920: 1993, Cl. 6.3.3
	• Beam bar splices location and spacing of hoops in the splice	Not available	IS 13920: 1993, Cl. 6.2.6
	• Minimum dimension of columns	400 mm × 450 mm	IS 13920: 1993, Cl. 7.1
	• Maximum percentage of reinforcement in column	1.77	IS 456: 2000, Cl. 26.5.3
	• Confining stirrups near ends of columns and in beam-column joints – Diameter – Spacing	 6 mm 100 mm	IS 13920: 1993, Cl. 7.4, Section 8
	• Column bar splices location and spacing of hoops in the splice	Not available	IS 13920, Cl. 7.2.1
	• Ratio of shear capacity of columns to flexural capacity of supported beams		IS 13920: 1993, Cl. 7.3.4

16.10 PRELIMINARY EVALUATION

The preliminary evaluation was done as per the procedure in Chapter 3.

16.10.1 Quick Checks

Using the expression $T_a = 0.09\, h/\sqrt{d}$ (Section 7.6, IS 1893: 2002), the fundamental periods of the building are 0.28 s and 0.36 s for the X- and Y- directions, respectively. The spectral acceleration coefficient (S_a/g) corresponding to each of the periods is 2.5. This is available from the response spectrum for Type II soil (Figure 2, IS 1893: 2002). For a building in Zone V, Z = 0.36 (Table 2, IS 1893: 2002). For an ordinary moment resisting frame, R = 3 (Table 7, IS 1893: 2002). For residential building, I = 1.0 (Table 6, IS 1893: 2002).

$\therefore$ Horizontal seismic coefficient, $A_h = \dfrac{ZIS_a}{2Rg} = \dfrac{0.36 \times 1.0 \times 2.5}{2 \times 3} = 0.15$.

$\therefore$ Design seismic base shear, $V_B = A_h \times W = 0.15 \times 19190$ kN $\cong$ 2878 kN.

The value of W = 19190 kN is calculated from the building plan.

The base shear V_B is distributed to the floor levels as per the following expression (Clause 7.7.1, IS 1893: 2002).

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

Table 16.16 shows the distribution of the base shear over the height of the building.

Table 16.16 Distribution of base shear over the height of the building

Floor No.	Seismic Weight, W_i (kN)	Height, h_i (m)	Lateral Force, Q_i (kN)
1	4250	3	65
2	4110	6	251
3	4110	9	564
4	4110	12	1003
5	2610	15	995
		Total	2878

<u>Shear Stress in Columns</u>

The lateral load resisting frames along X- and Y-directions were identified. Tables 16.17a and b show the average shear stresses in the columns at each storey along X- and Y- directions, respectively.

Table 16.17a Average shear stress in columns along X-direction

Storey No.	n_f	n_c	A_c (m^2)	V_j (kN)	v_{avg} (MPa)
G	9	24	4.6	2878	1.00
1	9	24	4.6	2813	0.98
2	9	24	4.6	2562	0.89
3	9	24	4.6	1998	0.69
4	9	24	4.6	995	0.35

Table 16.17b Average shear stress in column along Y-direction

Storey No.	n_f	n_c	A_c (m^2)	V_j (kN)	v_{avg} (MPa)
G	8	18	3.48	2878	1.49
1	8	18	3.48	2813	1.46
2	8	18	3.48	2562	1.33
3	8	18	3.48	1998	1.03
4	8	18	3.48	995	0.51

Axial Stress in Columns

The average axial stress (f_{avg}) in the ground storey columns is calculated from the approximate axial force and a representative cross-sectional area of the columns. The calculated values for the lateral load acting in either direction are given in Table 16.18.

Table 16.18 Axial stress in columns

	V_B (kN)	n_f	h (m)	L (m)	P (kN)	f_{avg} (MPa)
X-direction.	2878	9	15	5.92	506.4	2.8
Y-direction.	2878	8	15	3.60	936.8	4.7

Storey Drift

The values of the storey drift ratio for X- and Y- directions are shown in Tables 16.19a and b, respectively. Since the columns in the ground storey are supported on piles, they are assumed to be fixed at the bottom.

Table 16.19a Frame Drift Ratio along X-direction

Storey	Storey Height (m)	V_c (kN)	Drift Ratio
G	3	200	0.016
1	3	196	0.016
2	3	178	0.014
3	3	138	0.011
4	3	70	0.006

Table 16.19b Frame Drift Ratio along Y-direction

Storey	Storey Height (m)	V_c (kN)	Drift Ratio
G	3	298	0.022
1	3	292	0.021
2	3	266	0.019
3	3	206	0.015
4	3	102	0.007

Strong column – Weak beam

In the strong direction of the columns (about major axis)

$\sum$ Moment capacities of the columns = 484 kNm

1.2 $\sum$ Moment capacities of the beams = 536 kNm.

In the weak direction of the columns (about minor axis)

$\sum$ Moment capacities of the columns = 328 kNm

1.2 $\sum$ Moment capacities of the beams = 408 kNm.

16.10.2 Evaluation Statements

The evaluation statements under the relevant categories are presented together in Table 16.20. A number of statements are non-compliant because of the presence of open ground storey and absence of reinforcement detailing in the design drawings. Some of the statements are left blank because of insufficient data.

Table 16.20 Evaluation statements

Building system	C / NC / NA
Load path:	C
Adjacent buildings:	C
Mezzanines:	C
Deterioration of concrete:	C
Masonry units:	–
Masonry joints:	–
Cracks in infill walls:	C
Cracks in boundary columns:	C
Post-tensioning anchors:	NA

Concrete wall cracks:	NA
Deterioration of steel:	C
Vertical irregularities	
No weak storey:	NC
No soft storey:	NC
No mass irregularity:	C
No vertical geometric irregularity:	C
No vertical discontinuities:	C
Plan irregularities	
No Torsion irregularity:	C
No diaphragm discontinuity:	C
RC Moment resisting frames	
Lateral load resisting system	
Redundancy:	C
Shear stress check: v_{avg} > (Larger of 0.7 or 0.15√15 = 0.7 N/mm^2)	NC
Axial stress check: f_{avg} > (0.24 × 15 = 3.6 M/mm^2)	NC
Flat slab frames:	NA
Prestressed frame elements:	NA
Captive columns:	C
No shear failures: For a typical column, say 1C1, V_{uR} = 250 kN, M_{uR} = 236 kNm, h = 3 m. $\therefore V_{uR} > 2 \times 1.4\, M_{uR}/h$	C
Strong column-weak beam:	NC
Column bar splices:	NC
Column tie spacing:	C

Beam bars:	C
Beam bar splices:	NC
Stirrup spacing:	C
Bent-up bars:	C
Joint reinforcing:	NC
Deflection compatibility: Members are not identified as secondary components	NA
Flat slabs:	NA
Wall connections:	NC
Proportions of wall:	NC
Connections	
Column connection:	C
Uplift at pile caps:	C
Geologic site hazards	
Liquefaction	–
Slope failure	NA
Surface fault rupture	–
Foundations	
Foundation performance:	–
Overturning:	C
Ties between foundation elements:	NC

16.11 DETAILED EVALUATION

The detailed evaluation based on the linear analysis was done as per the procedure in Chapter 8. A few aspects of the computational model are first discussed.

16.11.1 Material Properties

It was not possible to conduct a condition assessment of the building. Hence, the material properties were taken from the design drawings. The material properties are given in Table 16.21.

Table 16.21 Materials properties

Material	Characteristic Strength (MPa)	Modulus of Elasticity (MPa)
Concrete (M 15)	15.0	19365.0
Reinforcing Steel (Fe 415)	415.0	2×10^5
Brick infill		1237.5

16.11.2 Modelling of Members

Figure 16.13 shows the 3D model of the building. Some of the modelling issues are mentioned.

Infill Walls

Two models were developed to consider the infill walls. In the first model (designated as 'with infill stiffness'), the infill walls were modelled as equivalent struts. Their masses were distributed on the respective supporting beams. This represents the case when no infill wall fails due to out-of-plane bending. The increase in stiffness of the building can lead to an increase in base shear and hence, forces in the beams and columns. In the second model (designated as 'without infill stiffness'), the stiffness due to the infill walls was neglected but their masses were included. Thus, there was no equivalent strut in any storey. This represents the case when all the infill walls fail under out-of-plane bending. The absence of the equivalent struts can lead to increased moments and shears in the beams and columns.

There is a lift core made up of RC walls. Since it is not integrally connected either to the frames, it was ignored in the model.

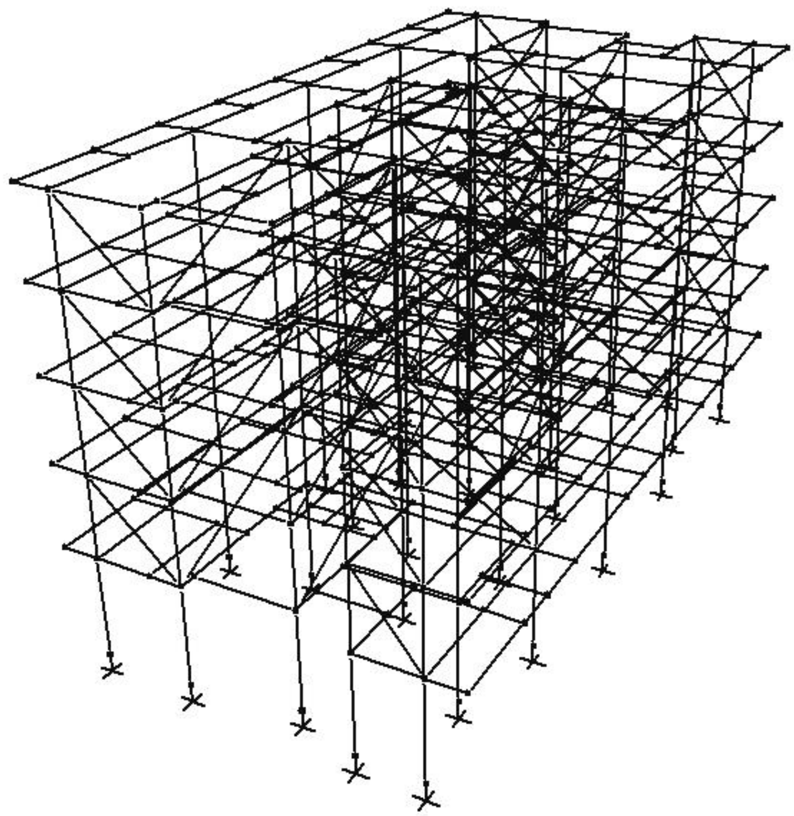


Figure 16.13 3D computer model of the building (with infill stiffness)

Column Ends at Foundation

The foundation system for the building is a pile foundation with groups of 4 or 6 piles. In the model, fixity was considered at the top of the pile caps. The effect of soil-structure interaction was ignored in the analyses. In the superstructure the beams are having eccentric connection at the columns. This was neglected in the model.

16.11.3 Equivalent Static Analysis

Calculation of Base Shear

The calculations for the fundamental time period (T_a), the spectral acceleration coefficient (S_a/g), the horizontal seismic coefficient (A_h), the design seismic base shear (V_B) and the equivalent static force in each floor (Q_i) are illustrated in the section of Preliminary Evaluation. Here, the values for both the models, with infill stiffness and with infill stiffness, are presented in Table 16.22. For the model with infill stiffness, T_a is calculated using the expression $T_a = 0.09\ h/\sqrt{d}$. For the model without infill stiffness, the corresponding expression is $T_a = 0.075\ h^{0.75}$ (Section 7.6, IS 1893: 2002). The distribution of base shear along the height of the building is shown in Table 16.23.

Table 16.22 Calculations for base shear of the building

		T_a (s)	S_a/g	A_h	W (kN)	V_B (kN)
With infill stiffness	X-direction	0.28	2.5	0.150	19190	2878
	Y-direction	0.36	2.5	0.150	19190	2878
Without infill stiffness	X-direction	0.59	2.3	0.138	19190	2649
	Y-direction	0.59	2.3	0.138	19190	2649

Table 16.23 Distribution of base shear at different floor levels

Floor no.	W_i (kN)	h_i (m)	Q_i (kN)	
			With infill stiffness	Without infill stiffness
1	4250	3	65	60
2	4110	6	251	231
3	4110	9	564	519
4	4110	12	1003	923
5	2610	15	995	916
		Total	2878	2649

Centres of Mass and Rigidity

The centre of mass (CM), the centre of rigidity (CR) and the static eccentricity in each direction (e_{si}) for each floor of the building were calculated for each model. The locations of CM and CR are with respect to the origin shown in Figure 16.24. The values of the eccentricities for the design centres of mass (for each floor) were calculated as per Clause 7.9.2, IS 1893: 2002. To reduce computation, for each floor only two of the four design centres of mass were considered for analysis. CM_1 represents the design centre of mass at the maximum distance from CR. CM_2 represents the design centre of mass diagonally opposite to CM_1. The calculated values are shown in the following tables.

Table 16.24 Calculations for CM and CR (with infill stiffness)

Floor	Seismic weight (kN)	Lumped mass (Ton)	CM (m)		CR (m)		e_{si} (m)	
			X	Y	X	Y	X	Y
5	2610	266	12.40	6.77	12.54	7.13	0.14	0.36
4	4110	419	12.56	7.14	12.53	7.26	0.03	0.12
3	4110	419	12.56	7.14	12.53	7.26	0.03	0.12
2	4110	419	12.56	7.14	12.53	7.26	0.03	0.12
1	4250	433	12.56	7.14	12.59	7.44	0.03	0.30

Table 16.25 Calculations for design centres of mass (with infill stiffness)

Floor	$e_{di\,1}$ (m)		$e_{di\,2}$ (m)		CM_1 (m)		CM_2 (m)	
	X	Y	X	Y	X	Y	X	Y
5	– 1.12	– 0.34	1.47	1.24	11.42	6.79	14.01	8.37
4	– 1.23	– 0.58	1.30	0.88	11.30	6.68	13.83	8.14
3	– 1.23	– 0.58	1.30	0.88	11.30	6.68	13.83	8.14
2	– 1.23	– 0.58	1.30	0.88	11.30	6.68	13.83	8.14
1	– 1.23	– 0.40	1.30	1.15	11.36	7.04	13.89	8.59

Table 16.26 Calculations for CM and CR (without infill stiffness)

Floor	Seismic weight (kN)	Lumped mass (Ton)	CM (m)		CR (m)		e_{si} (m)	
			X	Y	X	Y	X	Y
5	2610	266	12.40	6.77	12.51	7.42	-0.11	-0.65
4	4110	419	12.56	7.14	12.51	7.54	0.05	-0.40
3	4110	419	12.56	7.14	12.51	7.54	0.05	-0.40
2	4110	419	12.56	7.14	12.51	7.54	0.05	-0.40
1	4250	433	12.56	7.14	12.59	7.44	-0.03	-0.30

Table 16.27 Calculations for design centres of mass (without infill stiffness)

Floor	$e_{di\,1}$ (m)		$e_{di\,2}$ (m)		CM_1 (m)		CM_2 (m)	
	X	Y	X	Y	X	Y	X	Y
5	– 1.15	– 0.05	1.42	1.67	11.36	7.37	13.93	9.09
4	– 1.21	– 0.30	1.33	1.30	11.30	7.24	13.84	8.84
3	– 1.21	– 0.30	1.33	1.30	11.30	7.24	13.84	8.84
2	– 1.21	– 0.30	1.33	1.30	11.30	7.24	13.84	8.84
1	– 1.23	– 0.40	1.30	1.15	11.36	7.04	13.89	8.59

The selected design centres of mass for each floor are shown in the following figure.

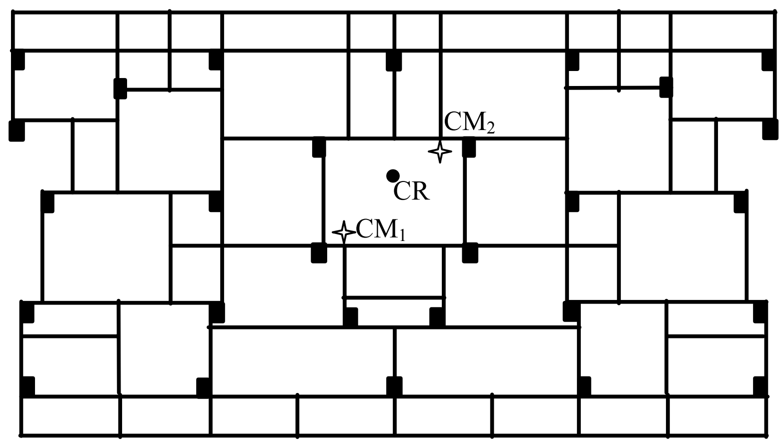


Figure 16.14 Locations of design centres of mass

16.11.4 Evaluation Results

From the equivalent static analysis, the values of the Demand-to-Capacity Ratios (DCRs) for a few ground floor beams and columns are given in Tables 16.28 and 16.29, respectively. The results show that a number of elements do not satisfy the requirement of DCR ≤ 1.0 for flexure. However, the DCR for shear is always less than 1.0 for both beams and columns.

Table 16.28 Demand-to-Capacity Ratios (DCRs) in beams

Beams	With infill stiffness		Without infill stiffness	
	DCR in Flexure	DCR in Shear	DCR in Flexure	DCR in Shear
1B1	0.95	0.62	1.4	0.6
1B2	1.12	0.76	0.3	0.3
1B3	1.22	0.65	1.5	0.9
1B4	1.00	0.75	1.1	0.5
1B5	1.39	0.89	1.5	0.9
1B6	0.79	0.21	1.5	0.9
1B7	0.79	0.20	1.1	0.5
1B8	1.33	0.81	1.5	0.9

1B9	1.01	0.70	0.3	0.3
1B10	2.52	1.46	1.4	0.6
1B11	0.17	0.22	1.5	0.7
1B12	0.53	0.34	1.7	0.8
1B13	0.76	0.65	1.4	0.6
1B14	0.88	0.48	1.2	0.5
1B15	1.16	0.92	1.2	0.5

Table 16.29 Demand-to-Capacity Ratios (DCRs) in columns

Columns	With infill stiffness			Without infill stiffness		
	DCR in Flexure	DCR in Shear		DCR in Flexure	DCR in Shear	
		V2	V3		V2	V3
1C1	1.40	0.41	0.55	1.1	0.4	0.1
1C2	1.78	0.65	0.34	1.4	0.6	0.2
1C3	1.72	0.60	0.74	1.3	0.3	0.9
1C4	1.67	0.56	0.40	1.5	0.4	0.1
1C5	2.18	0.60	0.79	1.2	0.2	0.8
1C6	1.58	0.47	0.73	1.2	0.4	0.7
1C7	1.81	0.66	0.69	1.5	0.4	0.7
1C8	1.80	0.71	0.70	1.4	0.6	0.5
1C9	1.60	0.50	0.74	1.4	0.2	0.8
1C10	1.40	0.47	0.80	1.4	0.4	0.1
1C11	2.10	0.63	1.04	1.9	0.2	0.7
1C12	1.72	0.49	0.65	1.4	0.3	0.9
1C13	1.87	0.66	0.88	1.6	0.6	0.6
1C14	1.56	0.65	0.84	1.5	0.4	0.9
1C15	1.63	0.68	0.86	1.4	0.5	0.2
1C16	1.64	0.55	1.04	1.1	0.1	0.2
1C17	1.93	0.62	0.90	1.5	0.4	0.7
1C18	1.58	0.59	0.74	1.3	0.2	0.1

1C19	1.77	0.69	0.84	1.4	0.5	0.9
1C20	1.63	0.52	0.52	1.4	0.6	0.2
1C21	1.70	0.61	0.46	1.2	0.3	0.1
1C22	1.97	0.70	0.50	1.3	0.4	0.2
1C23	1.97	0.69	0.60	1.3	0.2	0.5
1C24	1.79	0.71	0.60	1.4	0.4	0.7
1C25	1.71	0.63	0.63	1.5	0.3	0.5
1C26	2.02	0.57	0.80	1.0	0.6	0.7

The storey drifts for the models are shown in the following two figures. The values satisfy the limit of 0.4% (Clause 7.11.1, IS 1893: 2002).

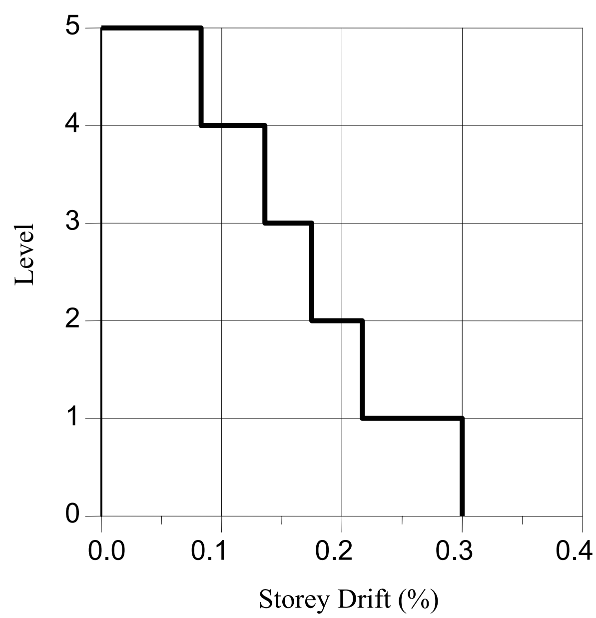


Figure 16.15 Storey drift under design seismic lateral force (with infill stiffness)

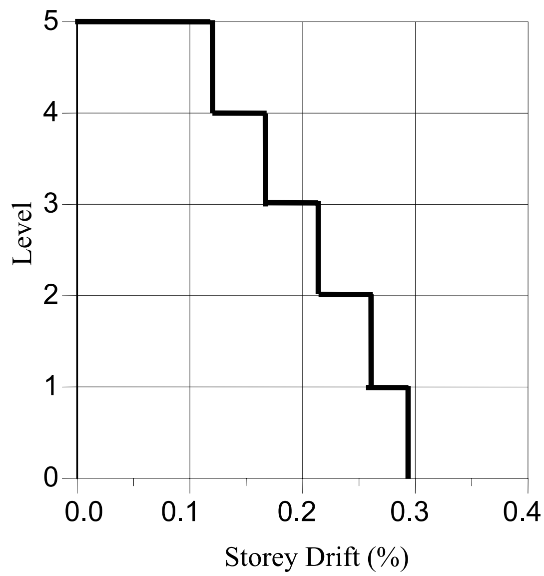


Figure 16.16 Storey drift under design seismic lateral force (without infill stiffness)

16.11.5 Pushover Analysis

The results of pushover analysis are briefly demonstrated. The pushover curves for the building with and without infill stiffness in Y-direction are shown in the following figure. Similar curves can be obtained for the pushover in X-direction.

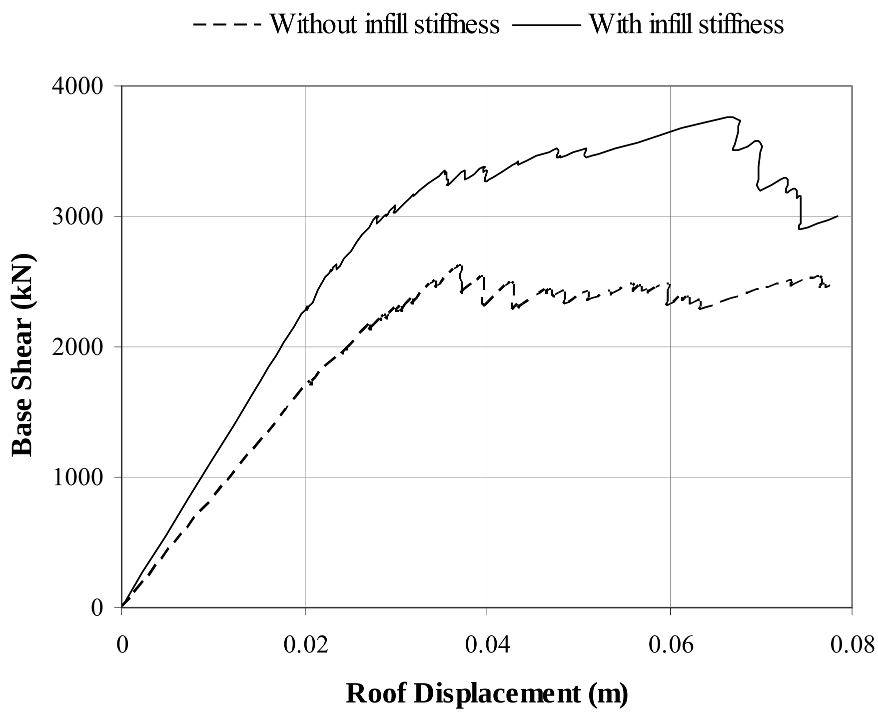


Figure 16.17 Pushover curves along Y-direction

The pushover analyses in either direction failed to give a performance point for both the models, with and without infill stiffness.

16.11.6 Observations

The observations from the detailed evaluation are summarised.

a) The equivalent static analysis results show that a number of beams and columns are deficient in flexure.

b) However, all the beam and column sections have adequate shear capacity.

c) The building complies with the drift requirement of IS 1893: 2002.

d) The pushover analyses in either direction failed to give a performance point before the collapse.

The building needs to be retrofitted.

16.12 RETROFIT

16.12.1 Retrofit Scheme 1

Two retrofit schemes are illustrated. In the first scheme, 230 mm thick shear walls are placed at a few locations throughout the height of the building. This is a global retrofit strategy. The following figure shows the locations of the new walls. This will cause least intervention in the functional requirement of car parking.

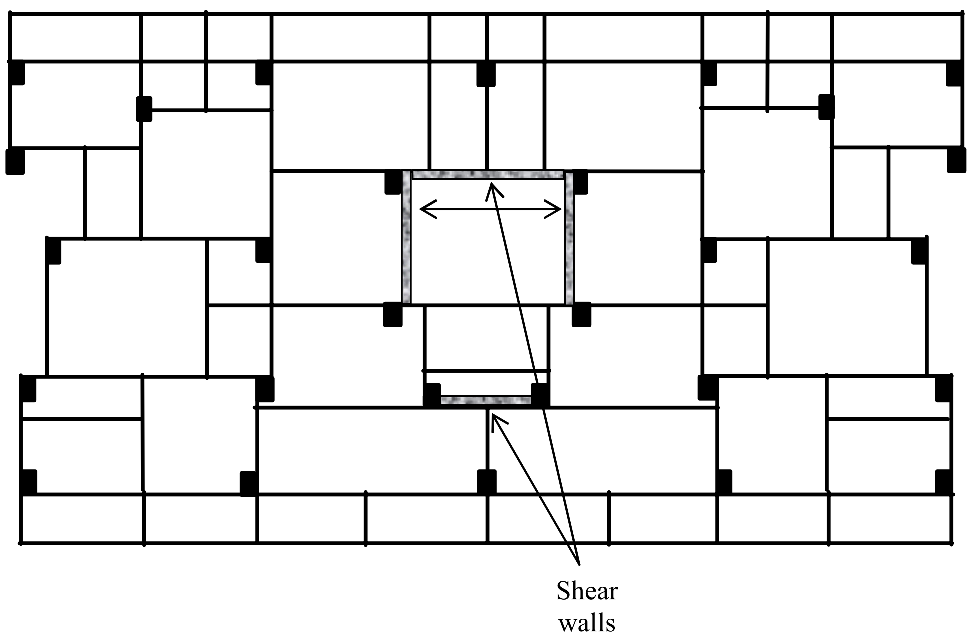


Figure 16.18 Locations of shear walls

16.12.2 Evaluation Results

The model for the existing building (model with infill stiffness) was modified to incorporate the retrofit scheme. From the equivalent static analysis, the values of the Demand-to-Capacity Ratios (DCRs) for a few ground floor beams and columns are given in Tables 16.30 and 16.31, respectively, along with the corresponding values for the existing building. The results show that the values of DCR have reduced for the retrofitted building. However, a few members do not satisfy the requirement of DCR ≤ 1.0 for flexure. These members need strengthening such as by concrete jacketing.

Table 16.30 Demand-to-Capacity Ratios (DCRs) in beams

Beams	Retrofitted		Existing	
	DCR in Flexure	DCR in Shear	DCR in Flexure	DCR in Shear
1B1	0.98	0.40	0.95	0.62
1B2	0.63	0.61	1.12	0.76
1B3	0.62	0.71	1.22	0.65
1B4	0.51	0.53	1.00	0.75
1B5	0.77	0.60	1.39	0.89
1B6	0.71	0.18	0.79	0.21
1B7	0.72	0.18	0.79	0.20
1B8	0.68	0.69	1.33	0.81
1B9	0.48	0.56	1.01	0.70
1B10	1.67	0.84	2.52	1.46
1B11	0.14	0.17	0.17	0.22
1B12	0.43	0.33	0.53	0.34
1B13	0.71	0.65	0.76	0.65
1B14	0.60	0.48	0.88	0.48
1B15	0.96	0.76	1.16	0.92

Table 16.31 Demand-to-Capacity Ratios (DCRs) in columns

Columns	Retrofitted			Existing		
	DCR in Flexure	DCR in Shear		DCR in Flexure	DCR in Shear	
		V2	V3		V2	V3
1C1	0.92	0.39	0.49	1.40	0.41	0.55
1C2	0.94	0.61	0.31	1.78	0.65	0.34
1C3	0.94	0.56	0.69	1.72	0.60	0.74
1C4	0.94	0.53	0.37	1.67	0.56	0.40

1C5	1.01	0.57	0.73	2.18	0.60	0.79
1C6	0.87	0.44	0.65	1.58	0.47	0.73
1C7	0.84	0.63	0.62	1.81	0.66	0.69
1C8	1.02	0.67	0.64	1.80	0.71	0.70
1C9	1.03	0.46	0.68	1.60	0.50	0.74
1C10	0.95	0.44	0.75	1.40	0.47	0.80
1C11	0.99	0.60	0.96	2.10	0.63	1.04
1C12	0.74	0.46	0.58	1.72	0.49	0.65
1C13	0.79	0.63	0.79	1.87	0.66	0.88
1C14	1.19	0.61	0.75	1.56	0.65	0.84
1C15	1.47	0.64	0.77	1.63	0.68	0.86
1C16	0.79	0.51	0.95	1.64	0.55	1.04
1C17	0.75	0.59	0.83	1.93	0.62	0.90
1C18	0.80	0.55	0.66	1.58	0.59	0.74
1C19	0.83	0.64	0.76	1.77	0.69	0.84
1C20	0.57	0.49	0.47	1.63	0.52	0.52
1C21	0.77	0.56	0.41	1.70	0.61	0.46
1C22	0.98	0.65	0.44	1.97	0.70	0.50
1C23	1.07	0.64	0.55	1.97	0.69	0.60
1C24	1.02	0.66	0.55	1.79	0.71	0.60
1C25	0.76	0.59	0.58	1.71	0.63	0.63
1C26	0.83	0.53	0.73	2.02	0.57	0.80

16.12.3 Retrofit Scheme 2

This retrofit scheme consists of global and local retrofit strategies. For the global strategy, full brick walls (230 mm) are continued in the ground storey at a few symmetrical locations of the building. The following figure shows the locations of the new walls. For the local strategy, all the ground storey columns were strengthened by concrete jacketing. The detailing for the jacketing can be as per the recommendations of the chapter on Retrofit of Reinforced Concrete Buildings.

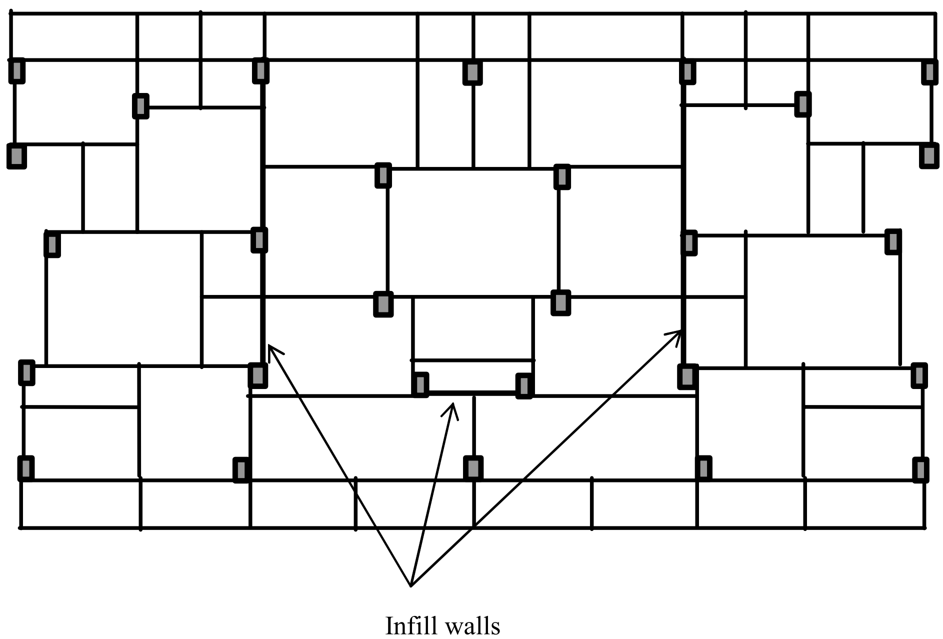


Figure 16.19 Locations of infill walls and column jacketing in ground storey

16.12.4 Evaluation Results

By the equivalent static analysis it is observed that the DCR values for the members of the retrofitted building do not reduce substantially. Hence, this scheme cannot be justified by the equivalent static analysis. However, the pushover analysis shows encouraging results. The pushover curves in Y-directions for the existing and retrofitted buildings are shown in the following figure. The scheme increases the lateral strength of the building substantially. The change in lateral stiffness is marginal. The ductility of the overall behaviour has increased. The pushover analyses in both the directions gave performance points. The building experiences a drift of about 1.0% at the performance point, which is acceptable. Thus, the retrofit scheme is acceptable based on the pushover analysis.

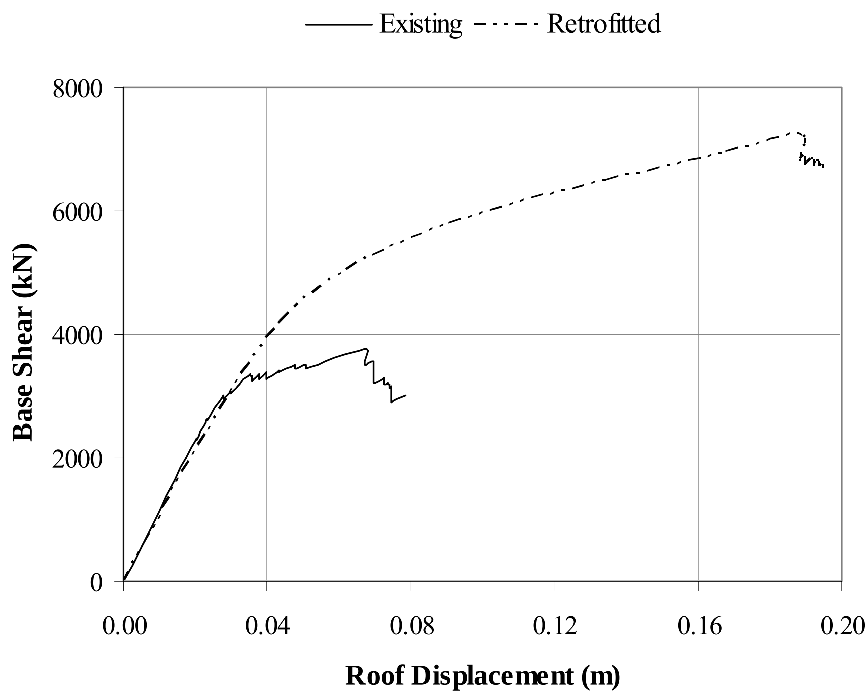


Figure 16.20 Comparison of pushover curves along Y-direction

16.13 SUMMARY

The seismic evaluation and retrofit of a multi-storeyed RC building is illustrated. The seismic evaluation includes rapid visual screening, data collection, preliminary evaluation and the detailed evaluation. It was not possible to conduct a condition assessment of the building. The structural model for the detailed evaluation is explained. The equivalent static analysis results show that a number of beams and columns are deficient in flexure. The pushover analyses in either direction failed to give a performance point before the collapse. So the building needs to be retrofitted.

Two retrofit schemes are illustrated. In the first scheme, shear walls are placed at suitable locations throughout the height of the building. This scheme can be justified by the equivalent static analysis. The second scheme involves addition of infill walls in the open ground storey and concrete jacketing of the ground storey columns. This scheme gives a satisfactory performance based on the pushover analysis. The second scheme is expected to be economical and to cause less disruption as compared to the first scheme.