CAPACITY BASED EARTHQUAKE RESISTANT DESIGN OF RC FRAMED BUILDING WITH A SOFT STOREY

Thesis Report submitted in partial fulfillment of requirement for

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Certificate

This is to certify that the thesis entitled, **" Capacity Based Earthquake Resistant Design Of RC Framed Building with a Soft Storey"** submitted by Ms. Dipali Sharma in partial fulfillment of the requirements for the award **of Master of Technology** Degree in **Civil Engineering** at the Jaypee university of Information Technology, (Deemed University), Waknaghat, Distt. Solan. (H.P.) is an authentic work carried out by her under my supervision and guidance .

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University / Institute for the award of any Degree or Diploma.

This is certified that the above statement made by the candidates is correct to the best of our knowledge.

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Signature of the student
Name of Student
Date

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Notations

SYMBOL	DESCRIPTION
ABS	Maximum Absolute response
CQC	Complete Quadratic Combination
ESA	Equivalent Static Analysis
IS	Indian Standards
MF	Multiplication Factor
E	Modulus of Elasticity
L	Span Length
M	Bending Moment
SRSS	Square Root of Sum of Squares
V_B	Design base shear
S_a/g	Average response acceleration
A_h	Design horizontal seismic coefficient
W	Seismic weight of the building
Z	Zone factor
I	Importance factor
R	Response reduction factor
d	Base dimension of the building
P	Axial load on the member

ABSTRACT

KEYWORDS : open ground storey, equivalent static analysis, response spectrum analysis, plastic hinges , soft storey irregularity.

Earthquakes in different parts of the world demonstrated the disastrous consequences and vulnerability of inadequate structures. Many reinforced concrete (RC) framed structures located in zones of high seismicity in India are constructed without considering the seismic codal provisions. The vulnerability of inadequately designed structures represents seismic risk to occupants. The main cause of failure of reinforced concrete frames during seismic motion is the soft storey sway mechanism or column sway mechanism. If the frame is designed on the basis of strong column-weak beam concept the possibilities of collapse due to sway mechanisms can be completely eliminated. In multi storey frames, this can be achieved by allowing the plastic hinges to form, in a predetermined sequences only at the ends of all the beams while the columns remain essentially in elastic stage and by avoiding shear mode of failures in columns and beams. This procedure for design is known as Capacity based design which would be the future design philosophy for earthquake resistant design of multi storey multi bay reinforced concrete frames.
Depending on the foundations resting on soft or hard soils, the displacement boundary conditions at the bottom of foundations can be considered as hinged or fixed.

The main objective of present study deals with the two problems regarding soft storey. The first problem deals with the comparative performance of open ground storey with continuous and discontinuous shear wall designed according to response spectrum dynamic analysis and the latter one deals with the aim is to present a detailed worked out example on seismic analysis and capacity based design of four-storey reinforced concrete frame building. There is discontinuity in lateral load resisting path because of discontinued shear wall and soft storey irregularity. Parametric studies on displacement, inter storey drift and peak storey shear have been carried out using response spectrum analysis and increase the flexural capacities of columns instead of beams, using capacity based design concept. At the end the building designed according to strong column weak beam concept have greater flexural capacities as compared to beams.

CHAPTER 1

EARTHQUAKE AND SEISMICITY

1.1. Earthquake

1.1Earthquake and Seismicity

An earthquake is the sudden , sometimes violent moment of the earth's surface from the release of energy in the earth's crust. This energy can be generated by a sudden dislocation of segments of the earth crust or by a volcanic eruption or magnetic activity. Most of the earthquakes , however are caused by the dislocations of the crust. An earthquake is also known as quake, tremor. The sudden release of the energy creates seismic waves. The seismicity, seismism or seismic activity of an area refers to the frequency, type and size of earthquakes experienced over a period of time.

1.2. Nature and occurrence of earthquake

When there is sudden localized disturbance in the rocks waves similar to those caused by a stone thrown into a pool spread out through the earth. An earthquake generates the same disturbance. The maximum effect of an earthquake is felt near its source, diminishing its distance from the source. The vibrations felt in the bedrocks are called shocks. Some of the earthquake are followed by the foreshocks and large earthquakes are followed by the aftershocks. Foreshocks are usually interpreted as being caused by the plastic deformation or small ruptures. Aftershocks are usually due to the fresh ruptures or readjustment of fractured rocks. The point of generation of an earthquake is known as focus, centre or hypocentre. The point on the earth's surface directly above the focus is called the epicentre. The depth of focus from the epicenter is called the focal depth. Seismic destruction propagates from the focus through a limited region of the surrounding earth's body which is called the focus region.

1.3.Seismic Waves

The large strain energy released during an earthquake travels in the form of seismic waves in all directions through the earth's layers reflecting and refracting at each interface .These waves can be classified as body waves consisting of P waves (which is also known as primary, longitudinal, compressional waves) and S waves (which is known as secondary , tranverse ,shear waves) and another one is surface waves consisting of love waves and Rayleigh waves.

1.4. Causes of Earthquake

Earthquake are vibrations or oscillations of the ground surface caused by a transient disturbance of the elastic or gravitational equilibrium of the rocks at or beneath the surface of the earth. The disturbance and the consequent movements give rise to elastic impulses or waves. Two theories explains the causes of the earthquake

1.4.1. Elastic Rebound Theory

The elastic rebound theory attributes the occurrence of tectonic earthquakes to the gradual accumulation of strain in a given zone and the subsequent gradual increase in the amount of elastic forces stored. The gradual accumulation and subsequent release of stress and strain is called elastic rebound. This theory postulates that the source of an earthquake is the sudden displacement of the ground on both sides of the fault which is a result of the rupturing of the crustal rock. Most of the earthquakes occur along the boundaries of tectonic plates and are called interpolate earthquakes. The other occurring within the plates themselves away from the plate boundaries and are called intraplate earthquakes.

1.4.2. Plate Tectonic Theory

According to this theory the earth's crust consists of a number of large rigid blocks called crustal plates. These plates bear loads of land masses, water bodies or both and are in constant motion on the viscous mantle. There are three types of zones.

- Zones of divergence.
- Zones of convergence.
- Fracture zones.

1.5. Effects of Earthquakes

1.5.1. Direct Effects

Seismic waves especially surface waves through surface rock layers result in strong ground motion. Such motion can damage and sometimes, completely destroy buildings. Ground shaking may compound the problem in areas with very wet ground in filled land, near the coast or in locations that have high water table. This is known as **liquefaction.** Liquefaction occurs when a material of solid consistency is transformed, with increased water pressure, in to a liquefied state. Water saturated, granular sediments such as silts, sands, and gravel that are free of clay particles are susceptible to liquefaction. Liquefaction occurred during the 1811-1812 New Madrid, Missouri, the 1989 Loma Prieta, California, the 1964 Niigata, Japan, and the 1967 Caracas, Venezuela, earthquakes.

1.5.2. Landslides

Ground motion also can trigger landslides. Careful consideration should be made before developers place a building in a location that could be affected by a landslide. A fire department in California found that out the hard way. During an earthquake in their community, a landslide blocked the exits to the firehouse, and, while the fire equipment was blocked inside, the town suffered millions of dollars in damage from fires caused by the earthquake.

1.5.3. Tsunamis and Seiche

Tsunami ("Soo na me") is Japanese for tidal wave. A tsunami is caused by an earthquake, landslide, or volcanic eruption on the sea floor. During an earthquake, seismic waves can produce powerful ocean waves. These waves tend to be very deep, with long distance between the peaks. In deep water there may be no noticeable evidence of the tsunami at the surface. However, when the wave enters shallow waters, the energy is forced to the surface and produces a tall wave that travels at high speed and moves far inland. Seaside communities are usually ravaged twice—first, when the water crashes in from the sea and, second, when the water recedes and carries loose objects out to sea. Though tsunamis are not as common as earthquakes, they can cause much more damage. Here in the United States, we can experience tsunamis on the

West Coast, Alaska, and Hawaii. “Seiche” refers to the oscillation (sloshing back and forth) of water in a closed space, such as a lake, reservoir, or swimming pool. This oscillation can cause overtopping of dams and damage to structures near water.

1.5.4. Faults

If a building stands on a fault line, little can be done to protect it during an earthquake. It is extremely important to select sites for new buildings that are away from known fault surface traces.

1.6 Seismic zoning of India

The varying geology at different locations in the country implies that the likelihood of damaging earthquakes taking place at different locations is different. Thus, a seismic zone map is required to identify these regions. Based on the levels of intensities sustained during damaging past earthquakes, the 1970 version of the zone map subdivided India into five zones – I, II, III, IV and V (Figure 3). The maximum Modified Mercalli (MM) intensity of seismic shaking expected in these zones were V or less, VI, VII, VIII,and IX and higher, respectively. Parts of Himalayan boundary in the north and northeast, and the Kachchh area in the west were classified as zone V.

1.7. Building Characteristics

Several important characteristics of buildings affect performance during an earthquake. Buildings of different construction materials or configurations will respond in different ways to the same ground motion; some may collapse while others survive. Knowing how buildings respond in an earthquake helps architects, engineers, and builders design and construct buildings to withstand ground motion without collapsing. Let’s consider some of the structural characteristics of buildings that influence how they behave during an earthquake. These characteristics are natural period, damping, ductility, stiffness, drift, and building configuration.

1.7.1. Natural Period

All objects (including buildings and the ground) have a "natural period," or the time it takes to swing back and forth, from point A to point B and back again. As seismic waves move through the ground, the ground also moves at its natural period. This can become a problem if the period of the ground is the same as that of a building on the ground. When a building and the ground sway or vibrate at the same rate, they are said to resonate. When a building and the ground resonate it can mean disaster. This is because, as the building and ground resonate, their vibrations are amplified or increased, and greater stress is placed on the building. Think of a building vibrating rapidly; at some point the building will begin to shake apart.

One of the most important factors affecting the period is height. A taller building will swing back and forth more slowly (or for a longer period) than a shorter one. For example, a 4-story building might have a natural period of 0.5 seconds, while a 60-story building may have a period of as much as 7 seconds. Building height can have dramatic effects on a structure's performance in an earthquake. A taller building often suffers more damage than a shorter one because the natural period of the ground tends to match that of buildings nine stories or taller.

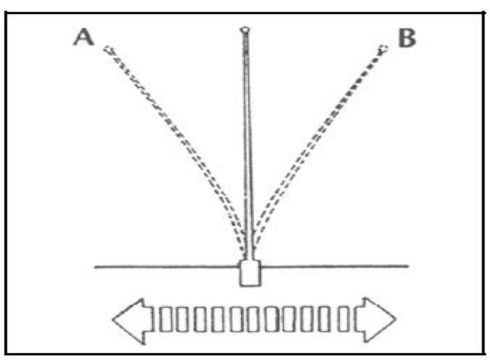


Fig 1.7.1. *Showing natural period*

1.7.2 Damping

The effects of resonance is by constructing buildings so that the vibration of a building is quickly reduced as an earthquake sets it in motion. This is called damping, the termination or retardation of the motion or vibration of a structure. Connections of nonstructural elements such as partitions, ceilings, and exterior walls can dampen a building's vibration.

1.7.3.Ductility and Strength

Ductility is another factor that can affect the performance of a building during an earthquake. Ductility is the property of certain materials to fail only after large stresses and strains have occurred. Figure 4-8 illustrates what we mean by ductility. Brittle materials, such as non-reinforced concrete, fail suddenly with minimum tensile stresses, so plain concrete beams are no longer used. Other materials, primarily steel, bend or deform before they fail. We can rely on ductile materials to absorb energy and prevent collapse when earthquake forces overwhelm a building. In fact, adding steel rods to concrete can reinforce it and give the concrete considerable ductility and strength. Concrete reinforced with steel will help prevent it from failing during an earthquake.

1.7.4. Stiffness

A building is made up of both rigid and flexible elements. For example, beams and columns may be more flexible than stiff concrete walls or panels. Less rigid building elements have a greater capacity to absorb several cycles of ground motion before failure, in contrast to stiff elements, which may fail abruptly and shatter suddenly during an earthquake. Earthquake forces automatically focus on the stiffer, rigid elements of a building. For this reason, buildings must be constructed of parts that have the same level of flexibility, so that one element does not bend too much and transfer the energy of the earthquake to less ductile elements of the building.

The 1987 failure of a parking structure in Whittier-Narrows, California, was a dramatic illustration of this phenomenon. To accommodate the natural slope of the land on which it was built, the structure was designed with long vertical supports at the front of the building and short columns at the other end, and the roof level was designed as a flat horizontal plane.

1.7.5. Drift

Drift is the extent to which a building bends or sways. Limits are often imposed on drift so a building is not designed to be *so* flexible that the resulting drift or swaying during an earthquake causes excessive damage. Figure 4-10 shows how a building can be affected by drift in an earthquake. If the level of drift is too high, a building may pound into the one next to it. Or the building may be structurally safe but nonstructural components, such as ceilings and walls, could be damaged as the building bends and the ceilings and walls are ripped away from their attachments. Of course, people in the building could be killed or injured from falling debris.

CHAPTER 2

INTRODUCTION

2.1. Introduction

This chapter comprises of two parts. The former part deals with soft storey, its formation and behavior and latter parts deals with capacity based design concept.

2.2. Soft storey

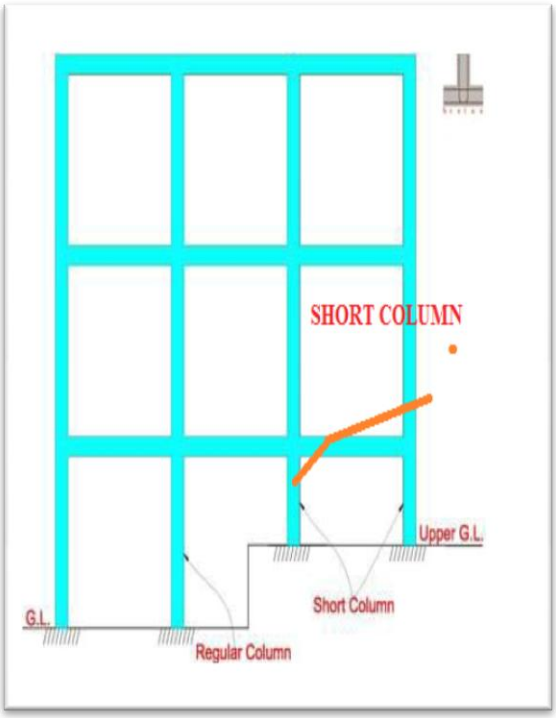
2.2.1. Introduction to Soft Storey

A soft storey is a structural anomaly attributed to the discontinuity of stiffness along the height of a structure. Severe structural damages suffered by several modern buildings during recent earthquakes illustrate the importance of avoiding sudden changes in vertical stiffness and strength. According to IS 1893 *"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."* A soft story building is a multi-storey building in which one or more floors have windows, wide doors, large unobstructed commercial spaces, or other openings in places where a shear wall would normally be required for stability as a matter of earthquake engineering design. Open ground storey (also known as soft storey) buildings are commonly used in the urban environment nowadays since they provide parking area which is most required. This type of building shows fig.2.2.1. comparatively a higher tendency to collapse during earthquake because of the soft storey effect. Large lateral displacements get induced at the first floor level of such buildings yielding large curvatures in the ground storey columns. The bending moments and shear forces in these columns are also magnified accordingly as compared to a bare frame building (without a soft storey). The energy developed during earthquake loading is dissipated by the vertical resisting elements of the ground storey resulting the occurrence of plastic deformations which transforms the ground storey into a mechanism, in which the collapse is unavoidable. The construction of open ground storey is very dangerous if not designed suitably and with proper care.

Fig.2.2.1 *Examples of open ground storey*

2.2.2. Formation of Soft Storey

One of the most common forms of discontinuity of vertical elements occur when shear walls that are present in the upper floors are discontinued in the lower floors. Formation of soft storey takes place that concentrates damage. Walls when do not go all the way to the ground but terminate at an intermediate storey level, soft storey is formed. The unequal height of column causes twisting and damage to the short columns of the building . Shear force is concentrated in the relatively stiff short columns which fails before the long columns. Long columns can be turned into short columns by the introduction of spandrels. Buildings with columns that hang and float on beams at an intermediate storey have discontinuities in load path also forms soft storey. Soft storey may results, if infills are omitted in a single storey. Even if infills are placed symmetrically and continuously, a soft storey may be formed if one or more infill panels should fail.



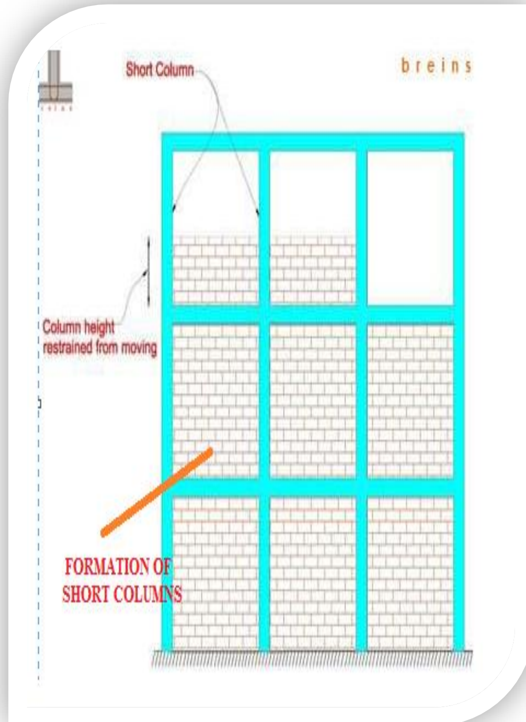


Fig.2.2.2. Formation of soft storey

2.2. 3. Behavior of Soft Storey

The buildings have been damaged due to various reasons. The principle causes of damage to buildings are soft stories. large spaces are required in buildings for parking, meeting rooms or a banking hall. Due to this functional requirement , the first storey has lesser strength and stiffness as compared to the above stories ,which are stiffened by masonry infill walls. .The stiffness of the lateral load resisting systems at those stories is quite less than the stories above or below. Due to this the above stories behave like a rigid block and most the horizontal displacement occurs in the soft storey alone. Such building act as an *Inverted Pendulum*, shows in fig 2.2.3. which swings back- and- forth and produces high stresses in the columns during earthquake. The dynamic ductility demand for such types of buildings during the earthquake gets concentrated in the soft storey and the upper stories remain elastic in nature

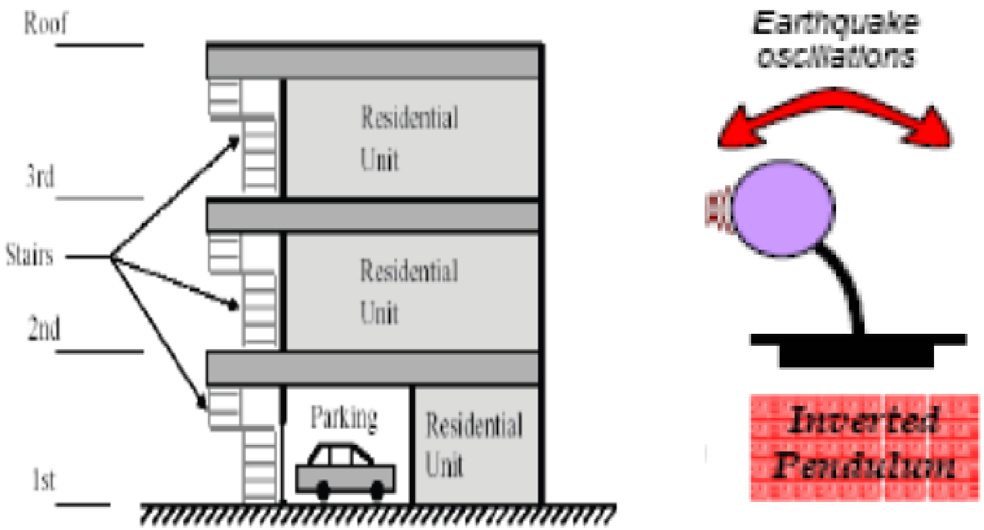


Fig. 2.2.3. *Cross section of soft storey building and its behavior*

2.2.4. Failure of Soft Storey

It has been observed that in the recent earthquake , the large number of existing buildings are vulnerable to damage or even collapse during strong ground motions. While the most of the damage of buildings is due to the soft stories .The ground storey columns were unable to provide adequate shear stiffness during the earthquakes. In addition, most of the energy developed during the earthquake is dissipated by the columns of the soft storey. In this process the plastic hinges are formed at the ends of columns which transform the soft storey into a mechanism. In such cases the collapse is unavoidable. Therefore, a soft storey deserve a special consideration in analysis and design. Such variation in storey stiffness could trigger "soft storey mechanism" failure type. Several such buildings failure due to soft storey mechanism had been observed in past earthquakes. It has been observed from the survey that the damage is due to collapse and buckling of columns. This type of failure shows in fig 2.2.4. results from the combination of several other unfavourable reasons such as torsion, excessive mass on upper floors P-Δ effects and lack of ductility in bottom storey.

Fig.2.2.4. *Collapse mechanism of a building with soft storey*

2.3 Capacity Based Design

2.3.1. Methods of Analysis

- **Equivalent Lateral Force Method**

The Equivalent lateral force method is the simplest method of analysis and requires less computational effort because the forces depend on the code based fundamental period of structures with some empirical modifier. The design base shear shall first be computed as a whole, and then be distributed along the height of buildings based on simple formulae appropriate for buildings with regular distribution of mass and stiffness. The design lateral force obtained at each floor level shall be distributed to individual lateral load resisting elements depending upon floor diaphragm action. The design lateral force or design base shear and the distribution are given by some empirical formulae given in the IS 1893.

- **Response Spectrum Analysis**

This method is applicable for those structures where modes other than the fundamental one affect significantly the response of the structure. In this method the response of Multi degree of freedom system is expressed as the superposition of modal response, each modal response being determined from the spectral analysis of Single–degree of freedom system ,which are then combined to compute the total response. There are computational advantages in using the response spectrum method of seismic analysis for prediction of displacements and member forces in structural systems. The method involves the calculation of only the maximum values of the displacements and member forces in each mode of vibration using smooth design spectra that are the average of several earthquake motions. The codal provisions as per IS: 1893 (Part 1)-2002 code for response spectrum analysis

The response spectral values depends upon the following parameters,

- Energy release mechanism
- Epicentral distance
- Focal depth
- Soil condition
- Richter magnitude
- Damping in the system
- Time period of the system

2.3.2. Methods for Seismic Design

➢ Lateral Strength Based Design:

This is most common seismic design approach adopted nowadays. It is based on providing the structure with the minimum lateral strength to resist seismic loads, assuming that the structure will behave adequately in the non-linear range. For this reason only some simple construction detail rules are needed to be satisfied.

➢ Displacement Based Design:

In this method the structure is designed to possess adequate ductility so that it can dissipate energy by yielding and survive the shock. This method operates directly with deformation quantities hence gives better insight on the expected performance of the structures. The displacement based design approach has been adopted by the seismic codes of many countries.

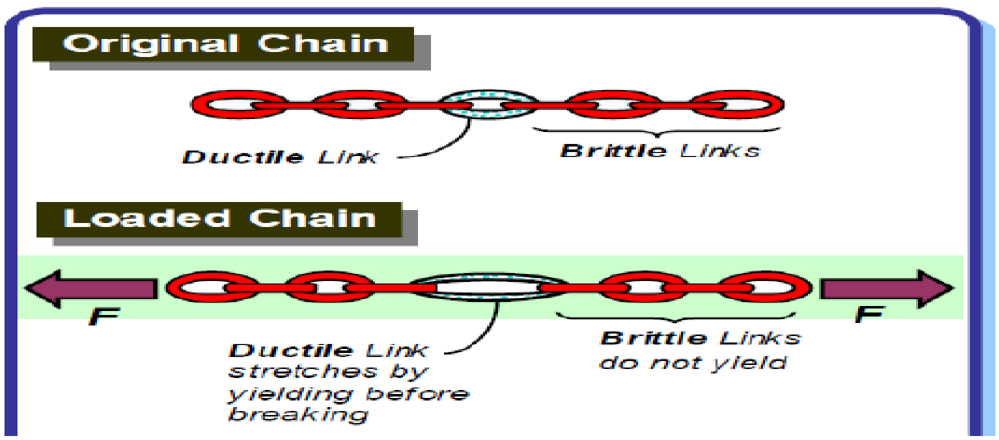
➢ Capacity Based Design

In this thesis capacity based design concept used to describe the best behavior of multi storey building during earthquake. Capacity Design is a concept or a method of designing flexural capacities of critical member sections of a building structure based on a hypothetical behavior of the structure in responding to seismic actions. This hypothetical behavior is reflected by the assumptions that the seismic action is of a static equivalent nature increasing gradually until the structure reaches its state of near collapse and that plastic hinging occurs simultaneously at predetermined locations to form a collapse mechanism simulating ductile behavior. The actual behavior of a building structure during a strong earthquake is far from that described above, with seismic actions having a vibratory character and plastic hinging occurring rather randomly. However, by applying the Capacity Design concept in the design of the flexural members of the structure, it is believed that the structure will possess adequate seismic resistance, as has been proven in many strong earthquakes in the past. A feature in the Capacity Design concept is the ductility level of the structure, expressed by the displacement ductility factor or briefly ductility factor. This is the ratio of the lateral displacement of the structure due to the Design Earthquake at near collapse and that at the point of first yielding. The basic of capacity based design lies on strong column and weak beam concept. The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The correct building components need to be made ductile. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make beams to be the ductile weak links than

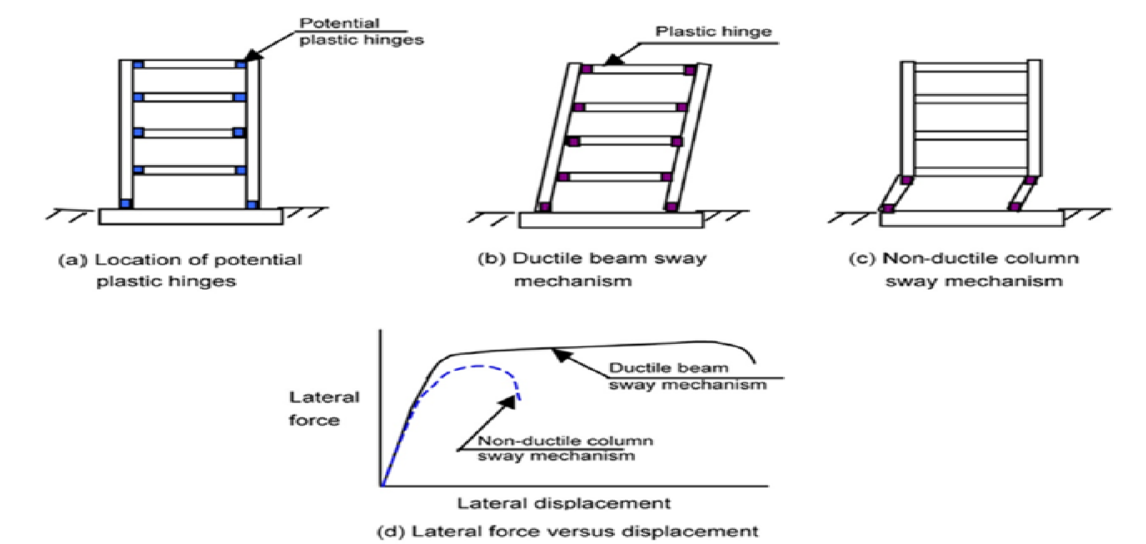
2.4. Detailed Description of Capacity Based Concept on the Basis of Two concepts

2.4.1. Chain Analogy

To highlight the simple concept of capacity design philosophy, the chain shown in Fig. 2.4.1. will be considered. The chain consists of links made of brittle and ductile materials. Each of these links will fail when elongated. Now hold the last link at either end of the chain and apply a force „F‟. Since the same force F is being transferred through all the links, the force in each link is the same i.e. F. As more and more force is applied, eventually the chain will break when the weakest link in it breaks. If the ductile link is the weak one (i.e. its capacity to take loads is less), then the chain will show large final elongation. Instead, if the brittle link is the weak one, then the chain will fail suddenly and show small final elongation. Therefore if we want to have such a ductile chain, we have to make the ductile link to be the weakest link.



(a) Links shows ductile behavior



(b)

Fig.2.4.1. (a) and (b) *Capacity Design Concept Illustrated with Ductile Chain*

2.4.2. Strong Column-Weak Beam Concept

It must be recognized that even with a weak beam strong column design philosophy which seeks to dissipate seismic energy primarily in well-confined beam plastic hinges, a column plastic hinges must still form at the base of the column. In structure with strong column weak beam concept, beam yield first than column. So column sway mechanism is avoided in the structure. Example of two frames are given below. Frame of fig.2.4.2. (a) With strong column weak beam concept and frame of fig. 2.4.2.(b) Without strong column weak Beam concept.

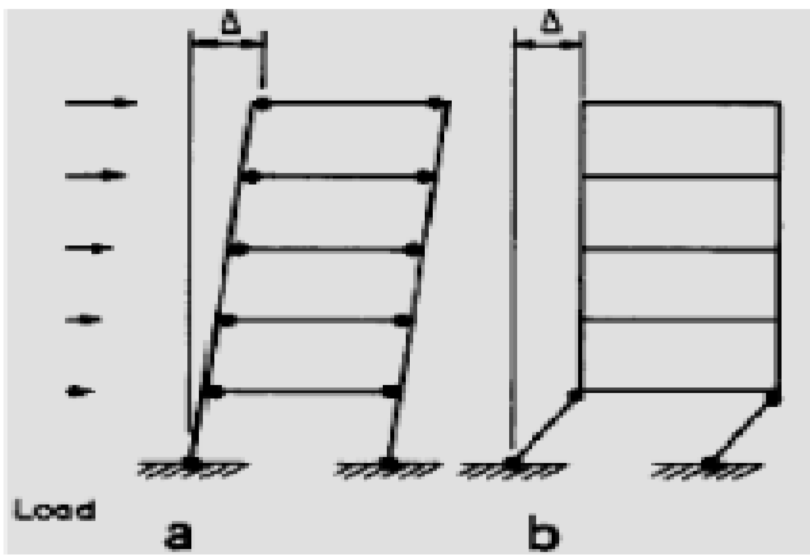


Fig. 2.4.2. *Comparison of energy dissipating mechanism with or without strong column – weak beam concept*

A comparison of the two example frames in fig. 2.4.2. shows that for the same maximum displacement Δ at roof level, plastic hinges rotations θ1in case (a) are much smaller than those in case (b), θ2. Therefore the overall ductility demand, in terms of the large deflections Δ, is much more readily achieved when plastic hinges develop in all the beams instead of only in the first storey column. The column hinge mechanism, shown in fig. 2.4.2.(b), also referred to as a soft-storey, may impose plastic hinge rotations, which even with good detailing affected regions, would be difficult to accommodate. This mechanism accounts for numerous collapses of framed buildings in recent earthquakes. In the case of the Fig.2.4.2. frame with strong column weak beam prohibit formation of column sway mechanism and only beam sway mechanism can be developed.

A capacity design approach is likely to assure predictable and satisfactorily inelastic response under conditions for which even sophisticated dynamic analysis techniques can yield no more than crude estimates.

2.5. Shear Wall

Shear walls are vertical elements of the horizontal force resisting system as shown in fig.2.5. They are typically wood frame stud walls covered with a structural sheathing material like plywood. When the sheathing is properly fastened to the stud wall framing, the shear wall can resist forces directed along the length of the wall. When shear walls are designed and constructed properly, they will have the strength and stiffness to resist the horizontal forces.

In this thesis, RC shear wall are used and design in STAAD.Pro v8i. Reinforced concrete (RC) buildings often have vertical plate-like RC walls called Shear Walls in addition to slabs, beams and columns. These walls generally start at foundation level and are continuous throughout the building height. Their thickness can be as low as 150 mm, or as high as 400mm in high rise buildings. Shear walls are usually provided along both length and width of buildings . Shear walls are like vertically-oriented wide beams that carry earthquake loads downwards to the foundation.

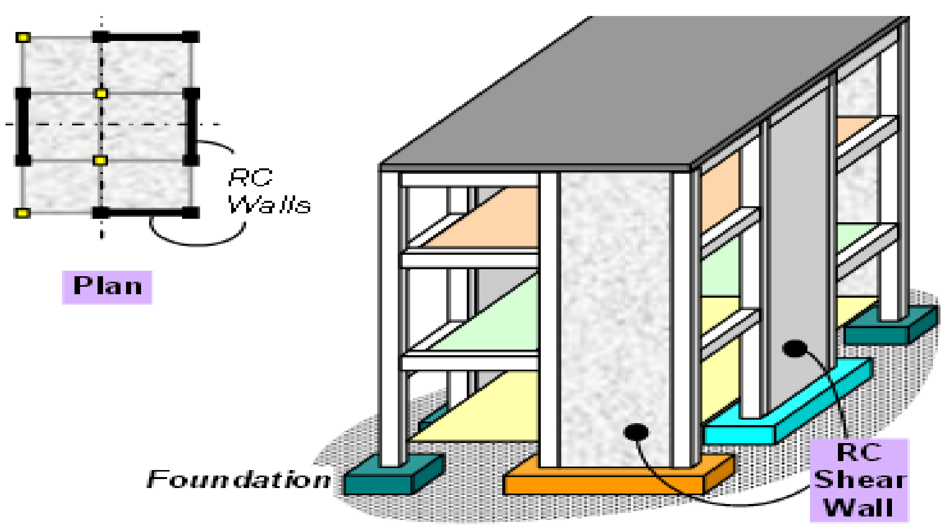


Fig. 2.5. *RC Shear wall*

2.5.1 Location of shear Wall

Most RC buildings with shear walls also have columns; these columns primarily carry Gravity loads (i.e., those due to self-weight and contents of building). Shear walls pSovide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces lateral sway of the building and thereby reduces damage to structure and its contents. Since shear walls carry large horizontal earthquake forces, the overturning effects on them are large. Thus, design of their foundations requires special attention. Shear walls should be provided along preferably both length and width. However, if they are provided along only one direction, a proper grid of beams and columns in the vertical plane (called a moment-resistant frame) must be provided along the other direction to resist strong earthquake effects. Door or window openings can be provided in shear walls, as shown in fig. 2.5.1. but their size must be small to ensure least interruption to force flow through walls. Moreover, openings should be symmetrically located. Special design checks are required to ensure that the net cross-sectional area of a wall at an opening is sufficient to carry the horizontal earthquake force. Shear walls in buildings must be symmetrically located in plan to reduce ill-effects of twist in buildings . They could be placed symmetrically along one or both directions in plan. Shear walls are more effective when located along exterior perimeter of the building – such a layout increases resistance of the building to twisting.

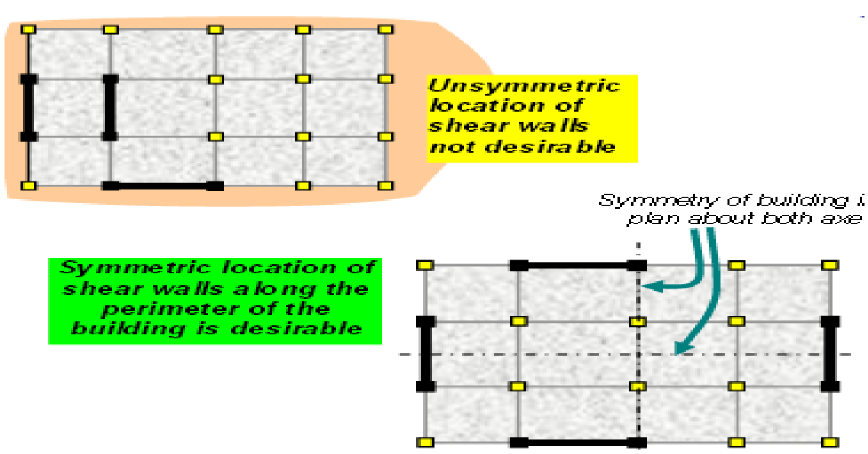


Fig.2.5.1. *Location of shear wall*

2.5.2. Advantages and Disadvantages of Shear Wall in RC Buildings

2.5.2.1. Advantages

Properly designed and detailed buildings with shear walls have shown very good performance in past earthquakes.

- Shear walls in high seismic regions require special detailing. However, in past earthquakes, even buildings with sufficient amount of walls that were not specially detailed for seismic performance (but had enough well-distributed reinforcement) were saved from collapse.
- Shear wall buildings are a popular choice in many earthquake prone countries, like Chile, New Zealand and USA. Shear walls are easy to construct, because reinforce ent detailing of walls is relatively straight-forward and therefore easily implemented at site.
- Shear walls are efficient, both in terms of construction cost and effectiveness in Minimizing earthquake damage in structural and non- structural elements (like glass windows and building contents.
- Determining the adequacy of existing floor and roofs slabs to carry the seismic forces.
- Transfer of diaphragm shear into the new shear walls by dowels.
- Adding new collectors and drag members to the diaphragm.

2.5.2.2. Disadvantages

- The discontinuity in the shear wall also a type of stiffness irregularity which makes the building as soft storey.
- The building with discontinued shear wall increased the overturning moments at foundation causing very high uplifting that needs either new foundations .
- Increases the dead load of the structure

CHAPTER 3

REVIEW OF LITERATURE

3.1. Overview

A state of the art literature review is carried out as part of the present study. This chapter presents a brief summary of the literature review. The literature review is divided into two parts. The first part deals with the seismic behavior of the RC frame building with soft storey with or without shear wall, whereas the second part of this chapter discusses about the capacity based earthquake resistant design of(G+3) RC frame building with soft storey.

3.2. Seismic behavior of RC Frame Building with Soft Storey

1) In the year of april 2014 paper was published and highlights the *"Seismic Response of RC 555Building with Soft Stories"*. This paper shows that many buildings that were collapsed during the past earthquake exhibited exactly the wrong concept i.e the weak column and strong beam which means columns failed before the beams yielded mainly due to soft storey effect. With thw help of this paper a simple understanding of soft storey is sudden change of lateral storey stiffness within the structure. . An irregularity in vertical configuration tends to create sudden changes in strength or stiffness that may concentrate earthquake forces or other forces in an undesirable way. This paper highlights the four types of failures Soft Storey Failure, Mass Irregularity failure, Plan Irregularity Failure, Shear Failure which were occured in the buildings. The main spotlight of this paper was soft storey failure, it is essential to estimate the demand and supply in the force and deformation of the members in the building. The soft storey at the parking level is the major reason for such types of failure. Two-dimensional analytical models are considered in this study. During the development of the analytical models, several issues are taken into consideration. An important topic at this stage is to evaluate easily the existence of the soft story behaviour in the structure. For this reason, two dimensional frame models for which the soft story behaviour can easily be accepted are selected for investigation. Each model in this study is named according to the total number of stories and first story height of it. For example model name “**HL3.2:6**” indicate six storey models with 3.2m heighted soft storey at ground level. The expression “HL” used for the height of the lower or first story, after that two digits (as 3.2) indicate height taken in related model as 3.2m and the digit after colon “:” indicates the numbers of stories taken in related model as 6 storey. All these analytical models are analysed according to the requirement of Indian Standard Codes. This paper concludes that the displacement estimates of the codal lateral load patterns are observed to be smaller for the lower stories and larger for the upper stories and are independent of the total number stories of the models. The uniform lateral load pattern leads to over estimations of displacements for all of the models and deformation levels. The estimations of the first mode lateral load pattern leads to more accurate displacement, the deviations on the results of this lateral load pattern decreases due to the existence of the soft stories magnitude equal to the stiffness of storey above.

2) Dande P. S.*, Kodag P. B. in there paper in the year of 2013 highlights the “ *Influence of Provision of Soft Storey in RC Frame Building for Earthquake Resistance Design”*.This paper talks about the provided strength and stiffness to the building frame by modified soft storey provision in two ways, (i) By providing stiff column & (ii) By providing adjacent infill wall panel at each corner of building frame. A 12- storey building with RC moment resisting frame with open first storey and unreinforced brick infill walls (panels) in upper storeys, chosen for this study. The building was deliberately kept symmetric in both orthogonal directions in plan to avoid torsional response under pure lateral forces. Response spectrum analysis was performed for the seven models of the building using SAP2000. The frame members were modeled with rigid end zone, the walls were modeled as panel element and floor were modeled as diaphragms rigid in plane. Deflection and other important parameters for lateral loads have been studied. Thispaper concludes that, displacement and force demands (i.e. BM & SF) in the first storey columns were very large for building with soft ground storey. It was difficult to provide such capacities in the columns of the first storey. From the fundamental time period, it has been found that when there is no infill wall (panel) i.e. for bare frame model, the time period value is more than the value predicted by code. When the bare frame model was subjected to lateral load, mass of each floor acts independently resulting each floor to drift with respect to adjacent floors. Thus the building frame behaves in the flexible manner causing distribution of horizontal shear across floors. It is clear that building with soft storey will exhibit poor performance during a strong shaking. But the open first storey was an important functional requirement of almost all the urban multistory buildings and hence cannot be eliminated. Alternative measures need to be adopted for this specific situation. The under-lying principle of any solution to this problem is in (a) increasing the stiffness of the first storey; (b) provide adequate lateral strength in the first storey**.**

3) Dr. Saraswati Setia in the year of 2012 some important things were heighted in the paper *"Seismic Response of R.C.C Building with Soft Storey".* In this paper dynamic ductility demand during probable earthquake was discussed. The stilt floor used in severely damaged or collapsed RC buildings introduced irregularity of sudden change of stiffness between ground storey and upper stories. In the upper stories brick infill walls were used due to which it increases the lateral stiffness of the frame by a factor of 3 to 4 times. For such buildings the dynamic ductility demand during probable earthquake gets only concentrated in the soft storey and the upper stories tends to elastic. In such buildings, the stiffness of the lateral load resisting systems at those stories was quite less than the stories above or below. Parametric studies on displacement, inter storey drift and storey shear have been carried out using equivalent static analysis to investigate the influence of these parameter on the behavior of buildings with soft storey. Building was analyzed with the software STAAD. Pro**.**Lateral displacement is largest in bare frame with soft storey defect both for earthquake force in X-direction as well as in Zdirection for corner columns as well as for intermediate columns. Displacement of intermediate column is more by 0.02% and 0.04% in X and Z-direction respectively w.r.t. corner column. Minimum displacement for corner column is observed in the building in which a shear wall is introduced in X-direction as well as in Z-direction. But in case of intermediate column, displacement is minimum in building having masonry infill in upper floors and with increased column stiffness of bottom story in comparison to the building with shear wall in Xdirection as well as in Z-direction. Building having masonry infill in upper floors and with increased column stiffness of bottom story and building with shear wall in core has a small first storey displacement of about 18% and 16% respectively of that of building having masonry infill in upper floors only. This implies crucial displacement may be effectively reduced if the stiffness of the first storey is made with in the order of

4) F. Hejazi11,2 S. Jilani,1,2, J. Noorzaei1,2, C. Y. Chieng1, M. S. Jaafar1,2, A. A. Abang Ali1in the year of 2011 in there paper discuss *"The effect of soft storey on structural response of high rise buildings."*This paper highlights the occurring of soft storey at the lower level of high rise buildings subjected to earthquake has been studied. Also has been tired to investigate on adding of bracing in various arrangements to structure in order to reduce soft story effect on seismic response of building. The concept of the soft story has been recognized long back by Fintel and Khan (1969). This concept is an attempt to reduce acceleration in a building by allowing the first-storey column to yield during an earthquake and produce energy-dissipation action.

- A new approach was proposed by Mo and Chang (1995) for soft story building which in this system, Teflon sliders are placed on the top of the first story reinforced concrete framed shear walls. These shear walls are framed by columns and beams, and are designed to carry a portion of the weight of the superstructure and the lateral load determined by the frictional characteristics of the Teflon sliders. The remaining first-story columns are designed for ductile behavior in order to accommodate large drifts.

- Arlekar (1997) highlighted the importance of explicitly recognizing the presence of the open first storey in the seismic analysis of buildings. The error involved in modeling such buildings as complete bare frames, neglecting the presence of infills in the upper storeys, was brought out with different analytical models.

- Kanitkar and Kanitk ar (2004) studied a five-storey building with soft storey with the intent ofreviewing the new provisions for the earthquake resistant design of structures addressed in the code IS 1893:2002.

- Lee and Co (2007) quantified the effect of the shear walls located in the lower soft stories on the general seismic response characteristics.

- Mastrandrea and Piluso (2009) proposed a design methodology for development of a collapse mechanism of a global type for eccentrically braced frames in soft story buildings.

- Tesfamariam and Liu (2010) used special index for soft story for seismic risk assessment of reinforced concrete buildings as classification vulnerability techniques.

- A 12 reinforced concrete framed storey buildings was considered to see the effect of high rise building on the earthquake and the effect of soft storey which are designed at the bottom floor of the buildings. Usually the most economical way of retrofitting of soft story building was by adding proper bracing to soft stories. For investigation on effect of different bracing installation arrangement on building in seismic response of structure with soft story at bottom, 6 models were designed with different condition using SAP 2000 software. It was found that location and numbering of bracing acts an important factor for the soft story structures to displace during earthquake. Also the soft story has been eliminated as the bracing was added to the consider floor, although the displacement at the top floor was still very high because there was no bracing at the top floor. So, result show that a bracing will only makes a different result for the storey that equipped with bracing. The horizontal and vertical movements of building which bracing installed in most bays were much reduced during earthquake compare with other models. So it shows that the use of bracing was effectively reduced effect of soft story on structure response in earthquake excitation.

5) Dr. Mizan DOĞAN Dr. Nevzat KIRAÇ Dr. Hasan GÖNEN Department of Civil Engineering, Osmangazi University Eskisehir- TURKEY in there paper *"soft-storey behaviour in an earthquake and samples of izmit-duzce"* in the year, 2002. In this paper some of the sections in the buildings were left for sales store, restaurants , bank branches ,installations and lightening. These were labeled as soft storeys in literature. In this paper, a ten storey building was considered and behaviour of storey was checked at ground level and also, one of the intermediate levels (sixth floor). The studies and investigations of the quake results, it was observed that the partitioning walls and beam filling enables building to gain rigidity . In the areas of IZMIT – DUZCE there was a great effect of a soft storey due to earthquake. Some solutions were investigated for making the soft storeys in the present constructions and in the one to be built earthquake resistant.This paper highlights, soft storey was a type of irregularity and irregularities should definitely be taken into consideration . If structural irregukarities were not taken into consideration, a construction not said to be resistant one, no matter the highest quality concrete was used. *Just like an illness in one part of the body affects the whole body , so does an irregularity in a constructions***.** The results of the studies in this paper the maximum displacement of the structure was approximately 0.0035. IZMIT quake results of soft storey. Nearly 85- 90% of the collapsed and damaged had soft storeys in them. During an earthquake, more moment and shear strength fall on the columns and walls in the entrance floors than one in the upper storeys . So, the structure is divided into two sections in terms of structural behaviour. This can be called dangerous storey instead of soft storey. The conclusions of this paper were the present soft storeys should be examined and if necessary, should be reinforced. Ratios of soft storey irregularities should be taken as SS > 0.8-9 and R > 1.5. Soft storey irregularities should be applicable in Regulations (Codes of Earthquake) and should be explained so that it would have the power of sanction. Since the behaviour of the soft storey is very different during a quake, the construction undergoes damage and it increases costs. For this reason, in regions where the risk of quake is high, we should not produce soft storeys, if necessary , quake controls should be done starting from design stage through the stage of occupancy.Columns and beams of soft storey must be designed according to IS code 1893:2002. Results of previous quakes, should be examined in detail so that they would shed light to future studies.

6) Jaswant N. Arlekar, Sudhir K. Jain and C.V.R. Murty, I.I.T.Kanpur in year 1997 studied the seismic response of RC frame buildings with soft first storey in their paper titled, "*seismic response of rc frame buildings with soft first storeys".* This paper highlighted the importance of explicitly recognizing the presence of the open first storey in the building. Immediate measures were suggested to prevent the indiscriminate use of soft first storey in buildings without considering the increased displacement, ductility, force demands in the first storey columns. This paper shows a spotlight on many past earthquakes, such as Northridge 1994, Kobe 1995 , San Fernando 1971. In this paper, a RC moment resisting frame building with open first storey and un- reinforced brick infill walls in the upper storeys, was chosen for the study. This building was located in zone III and was for residential use. Nine different models of the buildings with different wall thicknesses and different configurations for soft storey were studied. Analysis was performed using ETABS. Two analysis were performed equivalent static analysis and multi modal dynamic analysis. The damage incurred by the Aganta and Himgiri apartments in city of Jabalpur were very good examples with soft first storey.

- Himgiri apartments was a RC frame building with open first storey, one of the side of apartment was left for parking and brick infill walls on the other side. The infill portion of the building in the first storey was meant for shops or apartments. All the storeys above were filled with brick infil walls. The columns of the first storey were completely damaged with lots of other defects like spalling of concrete cover ,buckling of reinforcement bars, snapping of lateral ties, crushing of core concrete. On the other hand, the brick infill walls have less damage during earthquake. Only nominal damage were observed in the upper storeys and few cracks in the filler walls.

- The Ajanta apartments consists of the identical four storey RC building located side by side . Each of these building, have two apartments in each storey excepting the first storey, in first storey the space was left for parking. Other buildings has two apartments in the upper storeys and has only one apartment in the first storey, the other part was left for parking and has no infill walls. The damaged of columns in the building left with open first storey was more and destructive.

- Also, in Jabalpur two storey, C- shaped RC frame building (Youth hostel building) have a devastating damage to columns in the stilt buildings consisted of severe X-type cracking due to the cyclic lateral force. The upper storey have brick infill panels .This makes the upper storey very stiff as compared to the storey at stilt level. No damaged was observed in the upper storeys . The "*soft first storey*" at the stilt level is clearly the primary reason for such a severe damage.

- In this paper stiffness balancing was proposed between the first and second storey of a reinforced concrete moment resisting frame building with open first story and brick infills in upper storeys. This paper concludes, RC frame buildings with open first storey performs poorly during in strong earthquake shaking. The drift and the strength demands in the first storey columns are very large for buildings with soft ground storeys. It is not very easy to provide such capacities in the columns of the first storey. Buildings will exhibit poor performance during a strong shaking. This hazardous feature of Indian RC frame buildings needs to be recognized immediately, and necessary measures taken to improve the performance of the buildings.

3.3. Overview of Capacity Based Design

1) Sujin S. George, Valsson Varghese "*General concepts of capacity based design"* International Journal of Innovative Technology and Exploring Engineering (IJITEE) ISSN: 2278-3075, Volume-1, Issue-2, July 2012. An earthquake resisting building is one that has been deliberately designed in such a way that the structure remains safe and suffers no appreciable damage during destructive earthquake. However, it has been seen that during past earthquakes many of the buildings were collapsed due to failure of vertical members. Therefore, it is necessary to provide vertical members strong so as to sustain the design earthquake without catastrophic failure. Capacity designing is aiming towards providing vertical members stronger compared to horizontal structural elements. A structure designed with capacity design concept does not develop any suitable failure mechanism or modes of inelastic deformation which cause the failure of the structures. In capacity design of earthquake resisting structures, elements of primary lateral load resisting system are chosen suitably and designed and detailed for energy dissipation under severe inelastic deformation.

2) Amit Kumar1, Anant Kumar2 *"Analysis and capacity based earthquake resistant design of multi storeyed building"* iosr journal of engineering (iosrjen) issn (e): 2250-3021, issn (p): 2278-

8719 vol. 04, issue 08 (august. 2014), ||v1|| pp 07-13. Earthquakes in different parts of the world demonstrated the disastrous consequences and vulnerability of inadequate structures. Many reinforced concrete (RC) framed structures located in zones of high seismicity in India are constructed without considering the seismic code provisions. The vulnerability of inadequately designed structures represents seismic risk to occupants. The main cause of failure of multi-storey reinforced concrete frames during seismic motion is the soft storey sway mechanism or column sway mechanism. The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make beams to be the ductile weak links than columns. This method of designing RC buildings is called the strong-column weak-beam design method. If the frame is designed on the basis of strong column-weak beam concept the possibilities of collapse due to sway mechanisms can be completely eliminated. In multi storey frame this can be achieved by allowing the plastic hinges to form, in a predetermined sequence only at the ends of all the beams while the columns remain essentially in elastic stage and by avoiding shear mode of failures in columns and beams. This procedure for design is known as Capacity based design which would be the future design philosophy for earthquake resistant design of multi storey reinforced concrete frames.

3) Prof. Rekha Shinde Prof. Mukesh Shinde "*Performance Based Seismic Analysis of a Buildingwith Soft Storey".* "performance-based" seismic design, which can be thought of as an explicit design for multiple limit states Analyzing structures for various levels of earthquake intensity and checking some local and/or global criteria for each level has been a popular academic exercise for the last couple of decades, but the crucial development that occurred relatively recently was the recognition of the necessity for such procedures by a number of practicing engineers influential in code drafting. The main cause of failure of multistory multi-bay reinforced concrete frames during seismic motion is the soft storey sway mechanism or column sway mechanism. The seismic inertia forces generated at its floor levels are transferred through the various beams and columns to the ground. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect.The main objective of this search work is to present a detailed 3 dimensional seismic analysis and capacity based design of G+3, G+8 & G+15 storied three bay reinforced concrete frame. The report provides an introduction to the earthquake, effects, designing of the buildings and studies review of literature pertaining to performance based seismic analysis. It highlights various aspects related to the capacity based designing and explains about Limit State design which an old method of building designing. The study reveals modeling and analysis. The capacity based design of G+3; G+8 & G+15 of old and new building design methods have been modeled and analyzed. The report draws out the result of study and provides a comparative study. The project report also provides a comprehensive conclusion and offers a scope for future research as well.

4) Dalal Sejal P , Vasanwala S A, Desai A K " *Performance based seismic design of structure: A review"* InternationalJjournal of Civil and Structural engineering volume 1, no 4, 2011. Presented in this paper is an updated literature review of the Performance-based Seismic design (PBSD) method. Performance based Seismic design is an elastic design methodology done on the probable performance of the building under different ground motions. The derivative of the PBSD method, known as the Performance-based Plastic design (PBPD) method that has been widely recognized as an ideal method for use in the future practice of seismic design has also been reviewed and discussed. Performance-based Plastic design method is a direct design method starting from the pre-quantified performance objectives, in which plastic design is performed to detail the frame members and connections in order to achieve the intended yield mechanism and behavior. The findings show that a huge scope of research work is needed for development of PBPD method for other type of structures.

3.4. Objectives

The evaluation of irregular structures, such as building structures with soft stories, becomes more important as they have been seriously damaged or collapsed in the earthquakes due to their special collapse mechanisms. The soft story irregularity, which is one of the most hazardous vertical irregularities, is investigated in this thesis. The main objectives of this study can be listed as follows:

- In this thesis work is done on the soft storey, soft storey formation, behavior , its failure and best position in the buildings.
- Carrying out a complete analysis and design of 8 storey RCC building with soft storey including columns, shear walls.
- Building is designed according to strong column weak beam concept.
- Comparison of results of base shear, peak storey shear, inter storey drift , mode shapes.
- Remedies are suggested for the design of the soft storey columns and beams. Stiffness balancing concept between storey are applied.
- The thesis highlights, that the columns and beams of soft storey are designed according to codal provisions IS 1893 : 2002.

3.5. Scope of Study

Many urban multistorey buildings in India today have open first storey as an unavoidable feature. This is primarily being adopted to accommodate parking or reception lobbies in the first storeys. Failures observed in past earthquakes proved that the collapse in such buildings is predominantly due to the formation of soft-storey mechanism in the columns of the ground storey building.

The scope of this project are summarized as:

- RC framed Buildings, which is regular in plan must be taken into consideration.
- Multi storey buildings must be designed according to the stiffness balancing and ductility concept and strong column weak beam concept also useful for earthquake resistant buildings.
- The study of this project deals with two different types of buildings with and without shear wall and capacity based design concept but infill walls also taken for further work . Fixed supports are used in both the cases. All other types of support conditions will be used. . Soil-structure interaction also considered for the future study.

- However, the conclusions drawn in the present study are based on the case study of 4 storeyed and 8 storeyed building which is analysed in STAAD.Pro v8i software. For future work high rise buildings taken into account with other softwares.
- In the present study building models are analyzed using linear static analysis, dynamic analysis known as response spectrum analysis. Although nonlinear dynamic analysis being superior to other analysis procedures, so for future scope it can be very useful to check the response of soft storeys in severe earthquakes in all zones.

CHAPTER 4

PROBLEM STATEMENT

4.1. Analysis and Design of RC Frame

This thesis considered two problem statements one deals with the RC frame building with soft storey with continued and discontinued shear wall and the other one deals with the seismic behavior of (G+3) RC frame building with strong column weak beam concept known as capacity based design concept. The results of this chapter are discussed in chapter 5.

4.2.1. Problem Statement I

A 8- storey RCC office building is considered. The selection of building configuration is basically done as per IS: 456, 2000. The plan of the building is symmetrical and simple. The salient features of the building are :

- The building is located in seismic zone V , So the value of Z is taken according to zone V according to IS 1893 :2002 , Z = 0.36
- The type of soil is medium stiff.
- Importance factor, I = 1 (According to IS 1893:2002)
- The building consists of shear wall so it is designed as special moment resisting frame. R = 5.
- Size of column (450 x 450) mm
- Size of beam (300 x 250) mm
- Wall thickness 250 mm
- Live load on floor 3 KN/m² (According to IS 875 PART II)
- Live load on roof 1.5 KN/m² (According to IS 875 PART II)
- Dead load on floor 12 KN/m². (According to IS 875 PART I)
- Dead load on roof 10 KN/m² (According to IS 875 PART I)
- Pressure on full surface 2 KN/m²
- Damping 5% (For RCC Buildings)

4.2.2. Building Details

The building consists of design of shear wall. In first part, shear wall is continuous which go all the way to the ground but in second part the shear wall is discontinuous and do not go all the way to the ground. One of the most common forms of discontinuity of vertical members occurs when shear wall that are present in upper floors are discontinued in the lower floors. So the building with discontinued shear wall behaves like soft storey during strong earthquake. Changes in vertical configuration cause changes in stiffness and strength between adjacent stories of building. The response spectrum analysis is then performed for the modeled RC frame building using the computer software STAAD PRO. V8i and the respective observations are studied. During the development of the analytical models, several issues are taken into consideration. In this work it is important to evaluate the existence of the soft storey behavior in this structure. For this reasons two dimensional models are selected for which the soft storey behavior is easily detected. Having three bays in X direction each is of 5 m span except the middle one which is of 6m span and in the Z direction there are 3 bays of 4m span each. Height of each storey is taken as 3 m. The plan elevation and dimensions are shown in below fig.4.2.2.(a),(b),(c),

- **Plan of building**

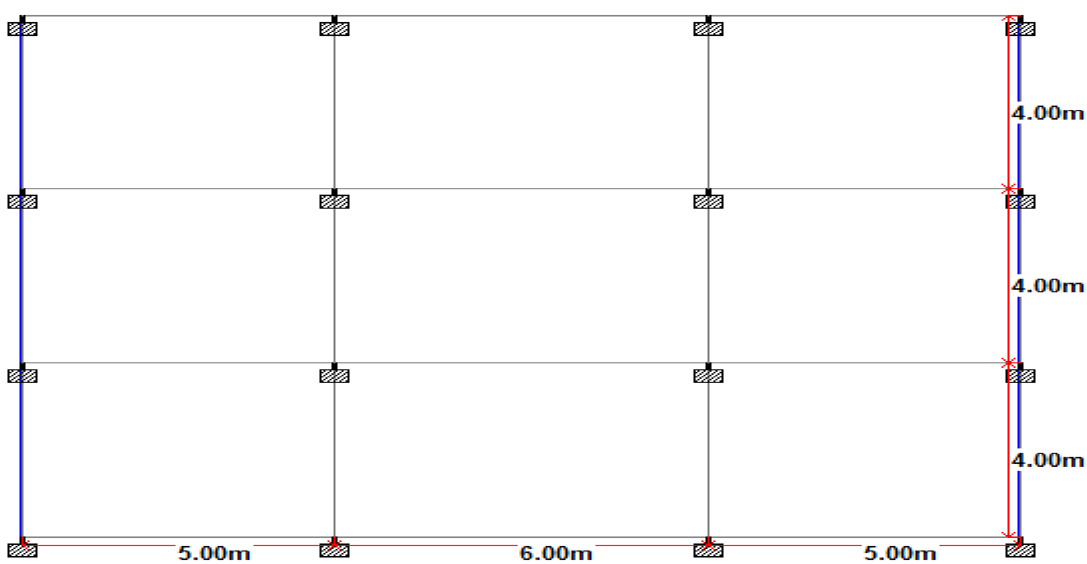


Fig. 4.2.2 (a) *Plan view of RC building*

➢ Elevation of building

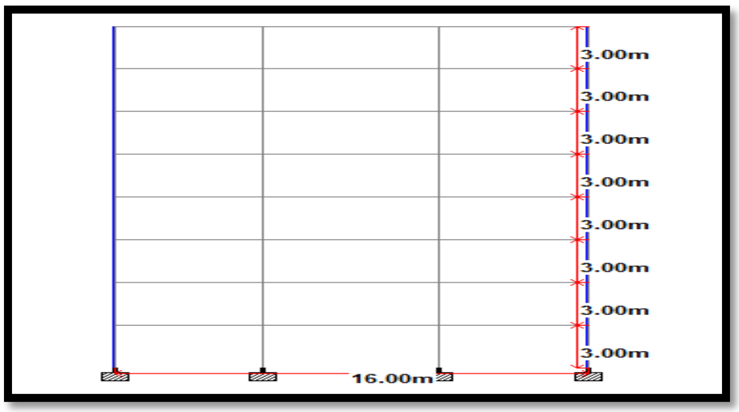
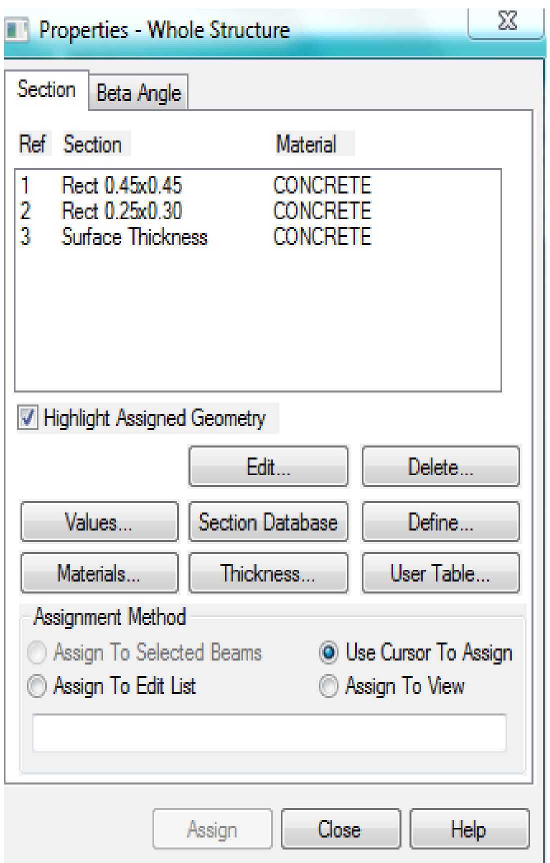


Fig 4.2.2 (b) *Elevation of RC building*

➢ Properties of building and support



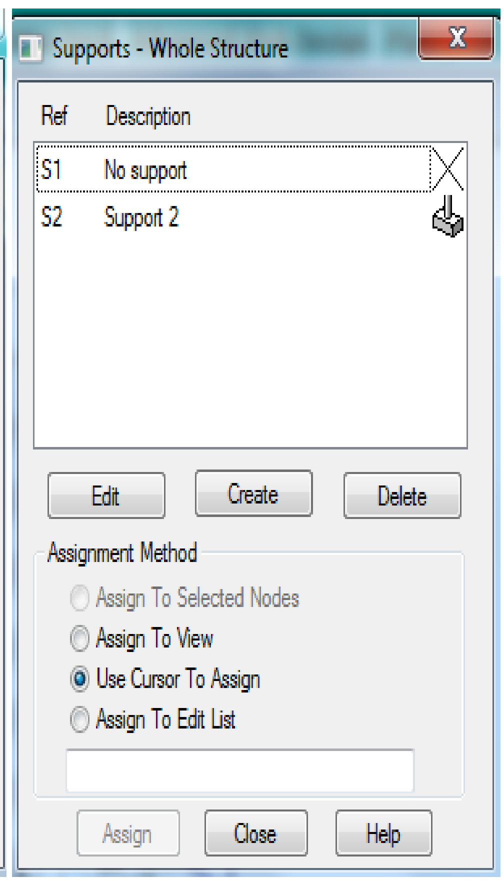


Fig. 4.2.2. (c) *Properties and supports of building*

4.2.3. Loading Combinations

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

1) $1.5\,(DL + IL)$
2) $1.2\,(DL + IL \pm EL)$
3) $1.5\,(DL \pm EL)$
4) $0.9\,DL \pm 1.5\,EL$

The loading used in this particular building is shown below

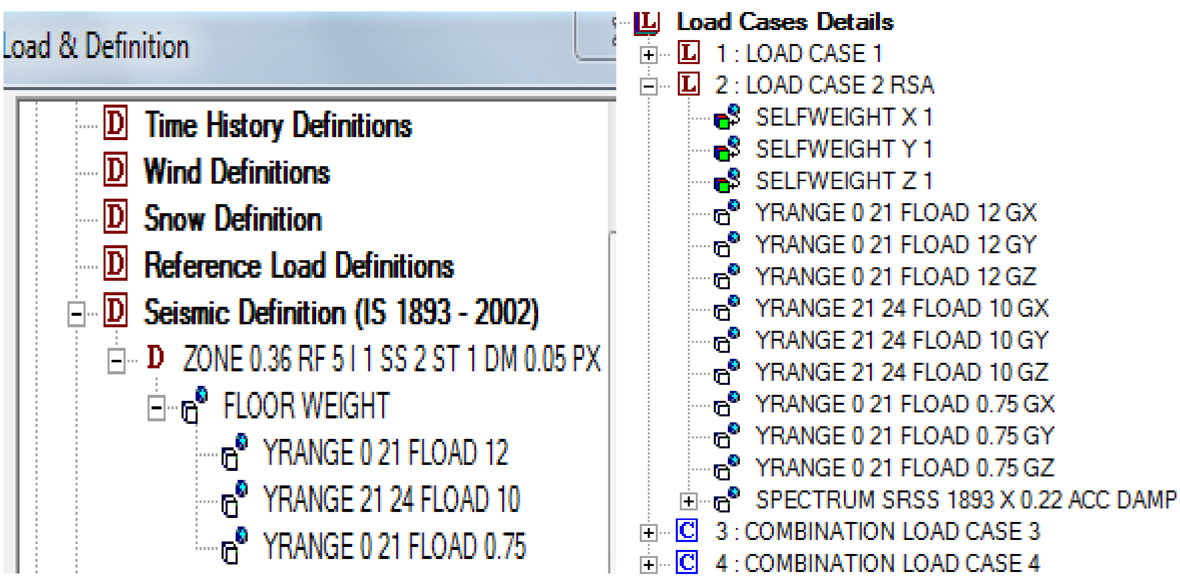
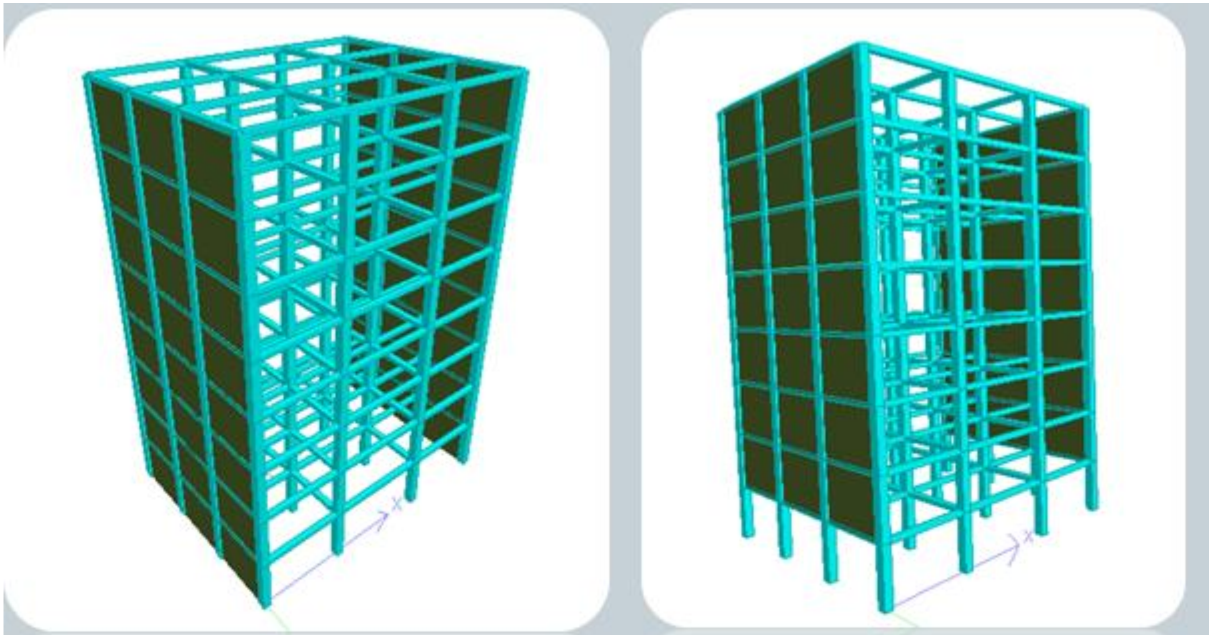


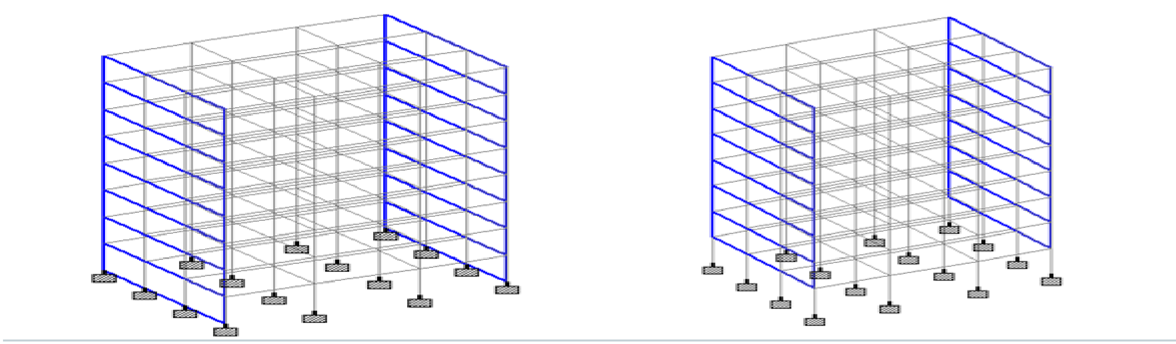
Fig 4.2.3 *Loading combinations*

4.2.4. Isometric view of Building with Continuous and Discontinuous shear wall

The isometric view is shown below in fig.4.2.4. this shows the 3D view of the building



(a)



(b)

Fig.4.2.4 (a) and (b) - *3D view and isometric view of building with continuous and discontinuous shear wall.*

4.3.1. Problem Statement II

An RC frame floor plan of a public cum office building is considered. The plan of the building is regular and has all the columns equally spaced. The building space frame is divided into a number of frames. The building is located in zone IV. The type of soil is medium stiff. The strong column weak beam concept is used so, building is special moment resisting frame. An interior frame 4-4 is taken into consideration for the analysis and design. The results of this chapter are discussed in chapter 6. The salient features of the building are:

- ✖ Type of structure --- Public cum office building
- ✖ Seismic zone --- IV
- ✖ No. of stories --- 4 (G+3)
- ✖ Ground storey ht --- 4.0 m
- ✖ Floor to floor ht --- 3.35 m
- ✖ Materials --- M 20 and Fe 415
- ✖ Seismic analysis --- Linear Static method
- ✖ Size of columns---- 300x530mm
- ✖ Size of the beams ----- 300x450mm
- ✖ Total slab depth ---- 120mm.
- ✖ **Dead Load:**
- ✖ Terrace water proofing = 1.5 kN/m² (Taken from IS 875 PART I)
- ✖ Floor finish = 0.5 kN/m² (Taken from IS 875 PART I)
- ✖ **Live Load:**

- ✖ Roof = 1.5 kN/m² (Taken from IS 875 PART II)
- ✖ Live load on Floor = 3.5 kN/m² (Taken from IS 875 PART II)
- ✖ Zone = 4 , Zone factor =0.24
- ✖ Height of the building = 14.05m
- ✖ No. of floors = 4
- ✖ Importance factor = 1
- ✖ R=5 (special RC moment resisting frame)
- ✖ The building space frame is divided into a number of frames. An interior frame 4-4 is considered for the analysis and design.

4.3.2. Building Details

The building consists of capacity based design which is known as strong column weak beam concept. The G+3 RC frame building is designed according to capacity based design concept. The step by step procedure of capacity based design is used for designing of building. The building is designed for maximum hogging and sagging moments. Members of building is designed according to IS 456 : 2000 and seismic analysis is done according to IS 1893 (PART 1) : 2002 . The linear static analysis is then performed for the modeled RC frame building using the computer software STAAD PRO.v8i and the respective observations are studied. During the development of the analytical models, several issues are taken into consideration. In this work it is important to eliminate the existence of the soft storey behavior in this structure. Seismic inertial forces generated at floor levels then transferred to beams and columns and then to the ground. So , the correct component of building made be ductile. For this reason the model is designed according to strong column weak beam design. Having three bays in X direction each is of 4.60 m span except the middle one which is of 2.30 m span and in the Z direction there are 3 bays of 4.60 m span except the middle one is 2.30 m. Floor to floor height is taken as 3.35 m and the ground storey height 4.0 m.

4.3.2.1.Plan of G+3 building

The plan of the building is shown in below fig.4.3.2.1. the plan is symmetric in nature.

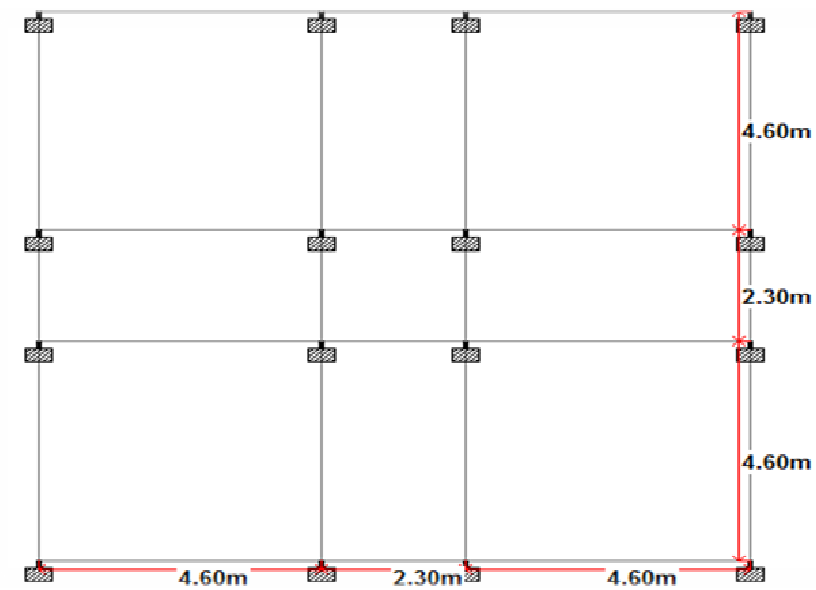


Fig.4.3.2.1.*Plan of the building*

4.3.2.2. Elevation of building

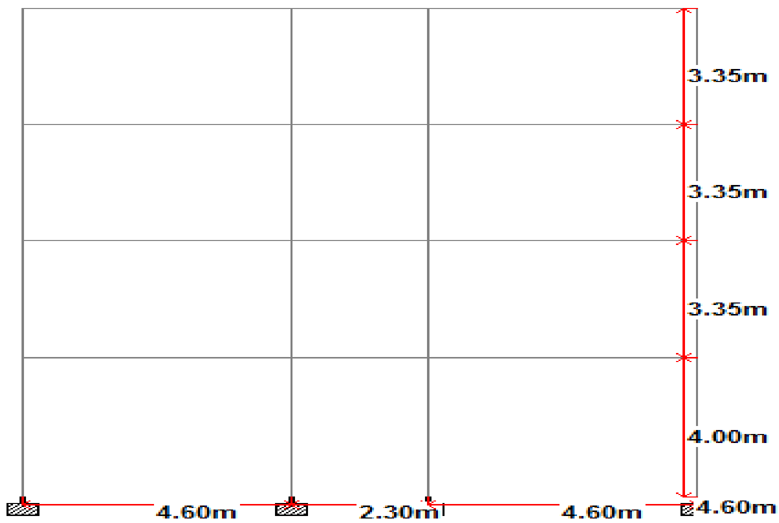


Fig.4.3.2.2.*Elevation of building*

4.3.2.3 . Isometric and 3D View of G+3 Storey Building

The isometric view and 3D view of the building with three storey is shown below in fig.4.3.2.3.

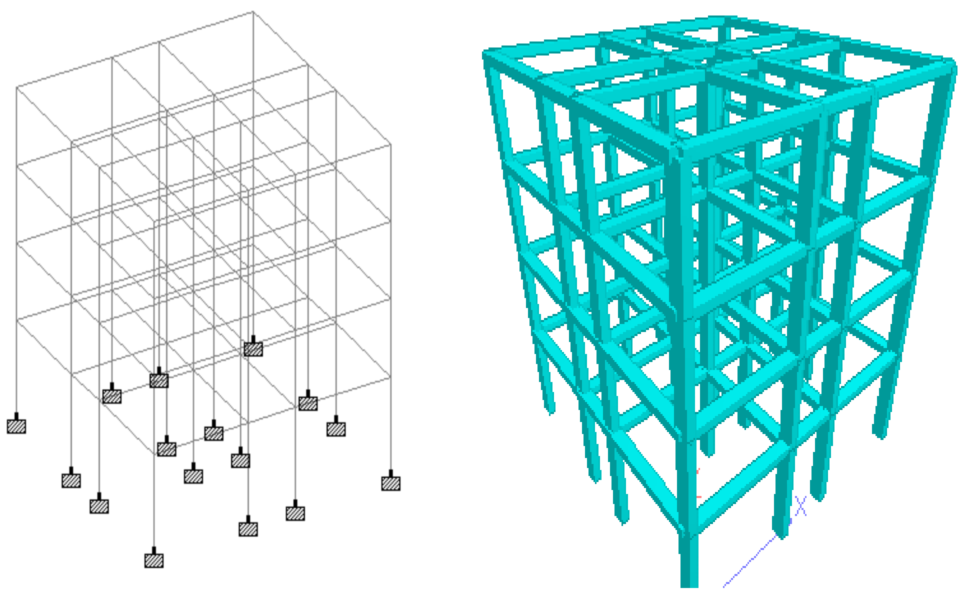


Fig 4.3.2.3. *Isometric view and 3D view*

4.3.2.4. Frame of RC with dimensioning and numbering and isometric view

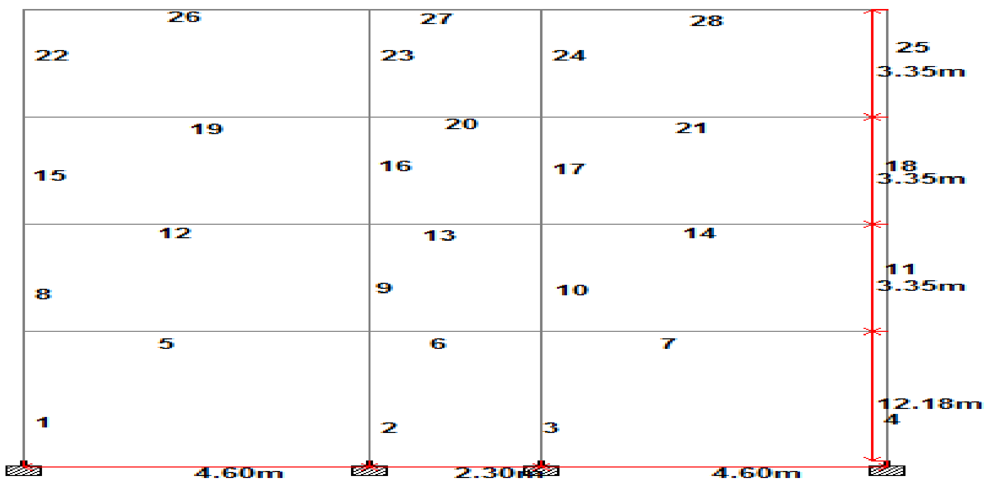


Fig.4.3.2.4. *Dimensioning and Numbering*

4.3.2.5. Properties Assigned in Building

The properties assigned to the 3 storey building is shown in below fig.4.3.2.5.

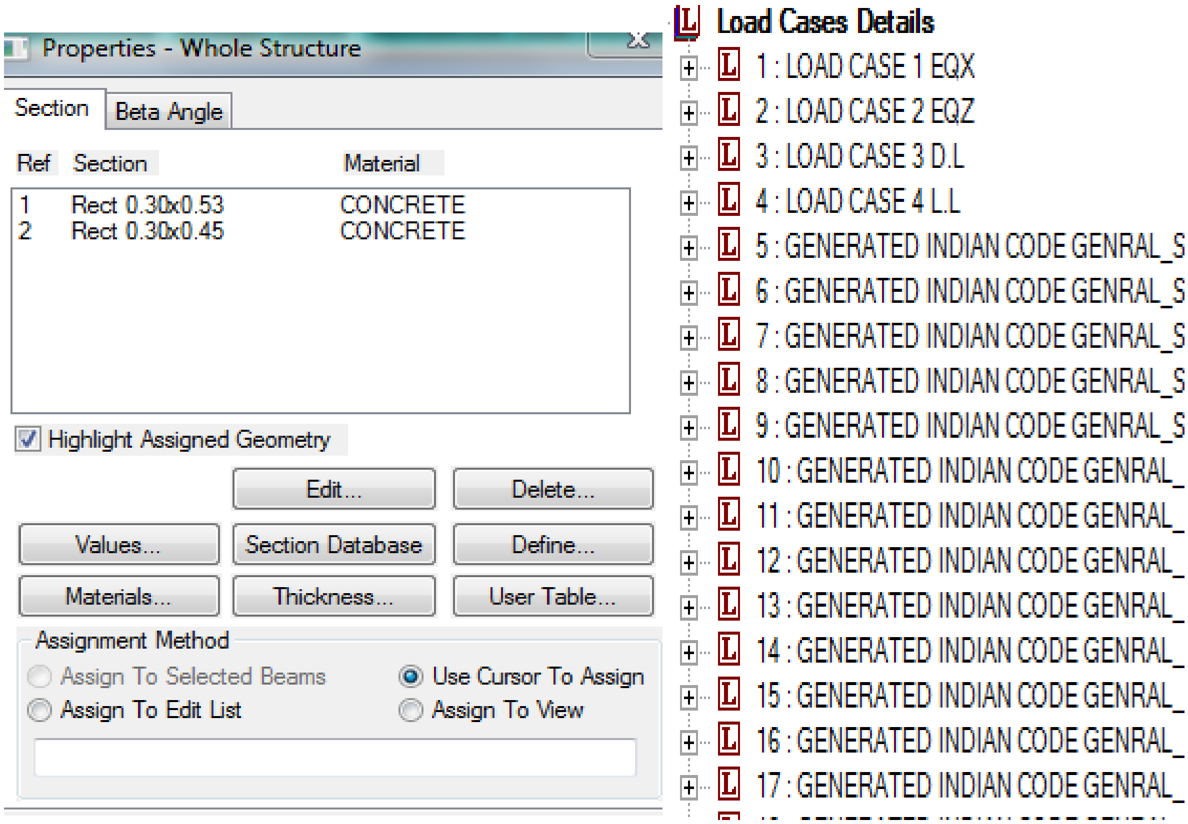


Fig.4.3.2.5.*Properties and loading*

The problem statement one and two are discussed above the solutions and analysis results are discussed in below chapters.

CHAPTER 5

RESULTS FROM RESPONSE SPECTRUM ANALYSIS

5.1. Analysis and Results of G+7 Building with Soft Storey due to Shear Wall Irregularity

Seismic analysis is a subset of structural analysis and is the calculation of the response of the building to earthquake and is a relevant part of structural design where earthquakes are prevalent. The seismic analysis of a structure involves evaluation of the earthquake forces acting at various level of the structure during an earthquake and strong ground motions. The effect of such forces on the behavior of the overall structure is calculated. The analysis may be static or dynamic in approach as per the code provisions. Analysis of structures to compute the earthquake forces is commonly based on one of the following three approaches.

1. An equivalent lateral procedure in which dynamic effects are approximated by horizontal static forces applied to the structure. This method is quasi-dynamic in nature and is termed as the Seismic Coefficient Method in the IS code.

2. The Response Spectrum Approac*h* in which the effects on the structure are related to the response of simple, single degree of freedom oscillators of varying natural periods to earthquake shaking.

3. Response History Method or Time History Method in which direct input of the time history of a designed earthquake into a mathematical model of the structure using computer analyses.

Two of the above three methods of analysis, i.e. Equivalent static analysis or Seismic Coefficient Method and Response Spectrum Method, are considered for the analysis of buildings studied here. Details of these methods are described below. The seismic method of analysis based on Indian standard 1893(PART1) :2002 is described as follows**:**

5.2. Equivalent Static Analysis

This is a linear static analysis. This approach defines a way to represent the effect of earthquake ground motion when series of forces are act on a building, through a seismic design response spectrum. This method assumes that the building responds in its fundamental mode. The applicability of this method is used in many building codes by applying factors to account for higher buildings with some higher modes, and for low levels of twisting. To account for effects due to "yielding" of the structure, many codes apply modification factors that reduce the design forces. In the equivalent static method, the lateral force equivalent to the design basis earthquake is applied statically. The equivalent lateral forces at each storey level are applied at the design 'centre of mass' locations. It is located at the design eccentricity from the calculated 'centre of rigidity (or stiffness)'. The base dimension of the building at the plinth level along the direction of lateral forces is represented as d (in meters) and height of the building from the support is represented as h (in meters) is shown in below fig.5.2. (a) and(b), (c). The response spectra functions can be calculated as follows:

For rocky, or hard soil sites

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

For medium soil sites

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

For soft soil sites

$$\frac{S_a}{g} = \begin{cases} 1 + 15\,T; & 0.00 \leq T \leq 0.10 \\ 2.50 & 0.10 \leq T \leq 0.67 \\ 1.67/T & 0.67 \leq T \leq 4.00 \end{cases}$$

Fig.5.2. (a) *Response spectrum values*

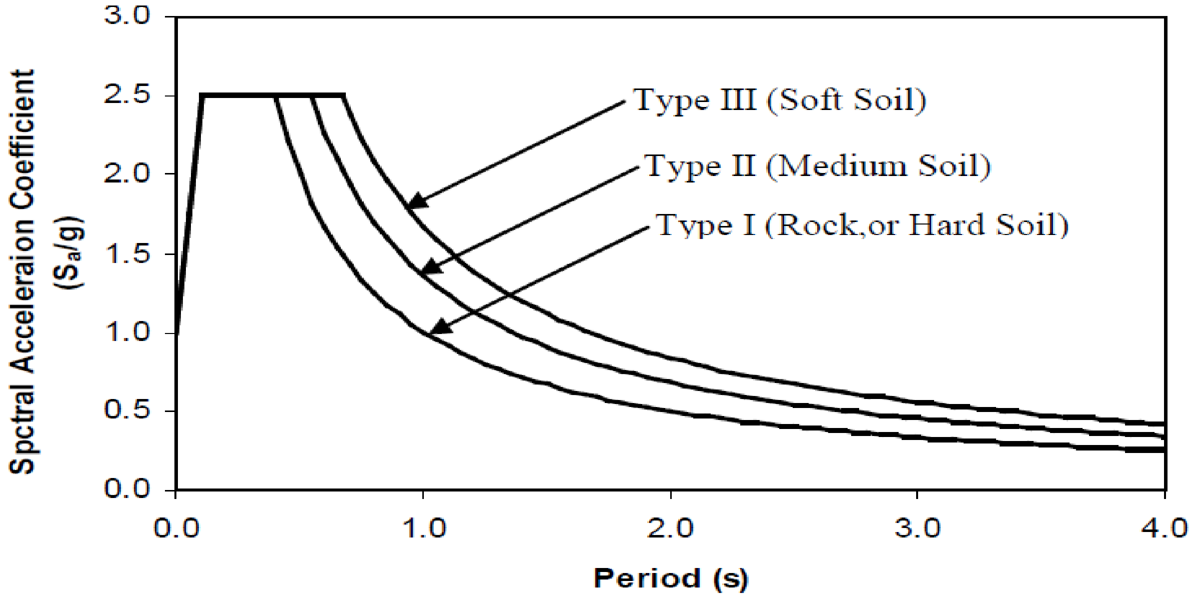


Fig. 5.2. (b) *Response spectra for 5 percent damping (IS 1893: 2002)*

The design base shear is to be distributed along the height of building as per Clause 7.7.1 of IS 1893: 2002. The design lateral force at floor i is given as follows,

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

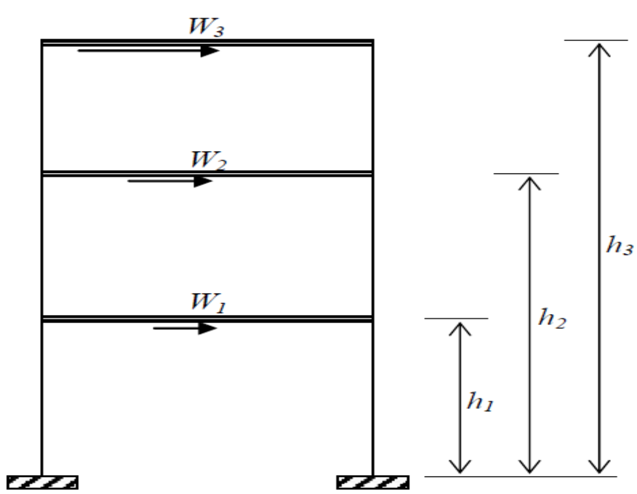


Fig.5.2 (c) *Building model under seismic load*

Where, Vb is base shear , Q is seismic weight wi is weight of one storey , hi is height of one storey and wihi known as modal participation factor.

Combination methods include the following:

- Maximum Absolute Response (ABS) - peak values are added together .
- Square Root of Sum of Squares (SRSS) - method of combining modal maxima for two-dimensional structural system.
- Complete Quadratic Combination (CQC) - a method that is an improvement on SRSS for closely spaced modes

In cases where structures are either too irregular, too tall or of significance to a community in disaster response, the response spectrum approach is no longer appropriate, and more complex analysis is often required, such as non-linear static or dynamic analysis.

5.3. Calculated Frequencies for Load Case II for Continuous Shear Wall building

In this thesis, the load case II is response spectrum load.

Table 5.3.1. *Calculated frequencies for response spectrum load*

MODE	FREQUENCY(CYCLES/SEC)	PERIOD(SEC)	ACCURACY
1	0.312	3.20351	6.927E-16
2	0.891	1.12204	4.278E-11
3	1.017	0.98317	2.565E-08
4	1.144	0.87427	3.841E-09

5	1.412	0.70839	4.602E-07
6	1.480	0.67564	6.043E-07

5.3.2. Base Shear in KN

Table 5.3.2. Base shear for G+7 storey building with continuous shear wall

BASE SHEAR	X	Y	Z
TOTAL SRSS SHER	2006.40	0.00	0.00
TOTAL ABS SHEAR	2727.59	0.00	0.00
TOTAL 10PCT SHEAR	2006.40	0.00	0.00
TOTAL CSM SHEAR	2006.40	0.00	0.00

5.3.3. Seismic Weight Calculations

The seismic weight calculations for building with continuous shear all is shown below from Staad output file .

```
RESPONSE LOAD CASE     2


CSM GROUPING  MODAL COMBINATION METHOD USED.
DYNAMIC WEIGHT X Y Z   2.427698E+04  2.427698E+04  2.427698E+04 KN
MISSING WEIGHT X Y Z  -2.645998E+03 -2.427698E+04 -8.959267E+03 KN
  MODAL WEIGHT X Y Z   2.163098E+04  2.175664E-07  1.531771E+04 KN
```

5.3.4. Peak storey shear

The peak storey shear is the sum of design lateral forces at all levels above the storey under consideration are shown in below fig.5.3.3

FLOOR	PEAK STOREY SHEAR IN KN	
-----	------------------------	
	X	Z
9	469.07	0.00
8	1130.65	0.00
7	1323.05	0.00
6	1407.19	0.00
5	1532.99	0.00
4	1741.01	0.00
3	1929.42	0.00
2	2006.40	0.00
1	2006.40	0.00

Fig. 5.3.3. *Peak storey shear*

5.3.4. Beam Graphs

The beam forces such as shear force axial force and bending force are shown below in fig.5.3.4. and graphs are taken from Staad.Pro v8i.

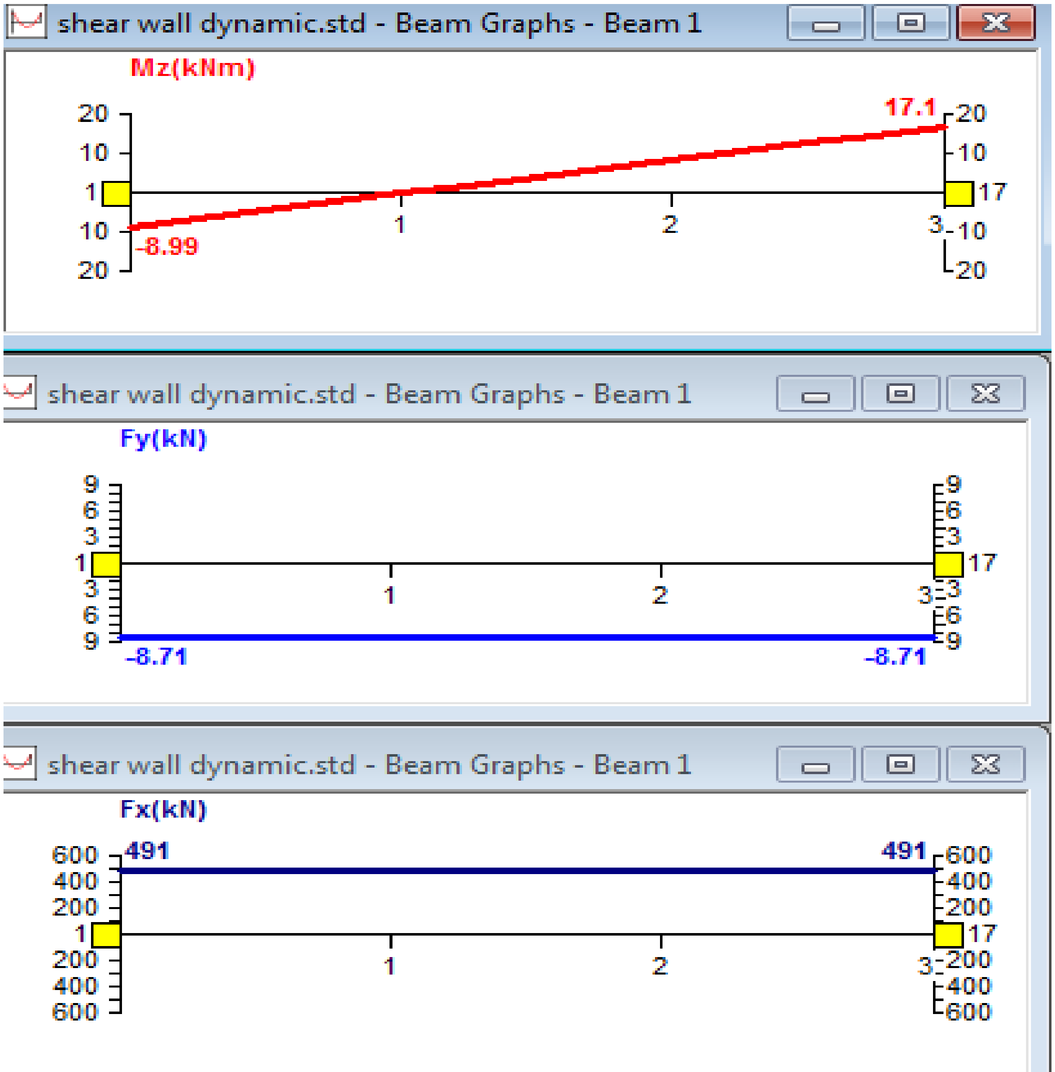


Fig.5.3.4. *Graphs showing bending moment, shear force and axial force*

5.4. Calculated Frequencies for Load Case II for Discontinuous shear wall building (soft storey)

The calculated frequencies and base shear for the discontinues shearw all building is calculated and shown in below Tables. 5.4. and 5.5

Table 5.4. Calculated Frequencies of soft storey

MODE	FREQUENCY(CYCLES/SEC)	PERIOD(SEC)	ACCURACY
1	0.308	4.24823	1.187E-16
2	0.792	1.26189	1.577E-11
3	0.974	1.02617	6.626E-09
4	1.138	0.87901	1.553E-08
5	1.321	0.75715	1.217E-07
6	1.395	0.91659	5.681E-07

5.4.1. Base Shear in kN

Table 5.4.1. *Base shear of discontinuous shear wall building*

TOTAL SRSS SHEAR	1881.17	0.00	0.00
TOTAL 10PCT SHEAR	1881.17	0.00	0.00
TOTAL ABS SHEAR	2554.76	0.00	0.00
TOTAL CSM SHEAR	1881.17	0.00	0.00

NOTE : *THE BASE SHEAR (VB) FROM RESPONSE SPECTRUM IS LESS THAN THE BASE SHEAR (Vb) CALCULATED USING EMPIRICAL FORMULA FOR FUNDAMENTAL TIME PERIOD. MULTIPLYING FACTOR (Vb/VB) IS 1.0035.*

5.4.2. Seismic Weight Calculations

The seismic weight calculations for load case 2 i.e response spectrum load for building with discontinuous shear wall is given below the dynamic weight and missing weight is shown below.

```
RESPONSE LOAD CASE     2


CSM GROUPING  MODAL COMBINATION METHOD USED.
DYNAMIC WEIGHT X Y Z   2.427698E+04  2.427698E+04  2.427698E+04 KN
MISSING WEIGHT X Y Z  -2.635986E+03 -2.427698E+04 -5.246207E+03 KN
  MODAL WEIGHT X Y Z   2.164099E+04  6.255458E-08  1.903077E+04 KN
```

5.4.3. Peak Storey Shear

The peak storey shear is less in discontinuous shear wall because the there is discontinuity in load path so, the seismic weight is less in this type of building. The values are given below in Table 5.4.3. peak storey shear is less because shear wall does not go all the way down to the ground. It is the sum of design lateral forces at all levels above the storey taken into consideration.

Table 5.4.3 *Peak storey shear with discontinuous shear wall*

FLOOR	PEAK STOREY SHEAR IN KN	
-----	------------------------	
	X	Z
9	439.98	0.00

	8	1061.60	0.00
	7	1242.82	0.00
	6	1321.49	0.00
	5	1438.01	0.00
	4	1632.17	0.00
	3	1809.12	0.00
	2	1881.17	0.00
	1	1881.17	0.00

5.5. Comparison of Modes Shapes

The mode shapes of the building with and without shear wall is shown in below fig.5.5 (a) , (b) , (c). In this thesis only three modes shapes are taken into consideration.

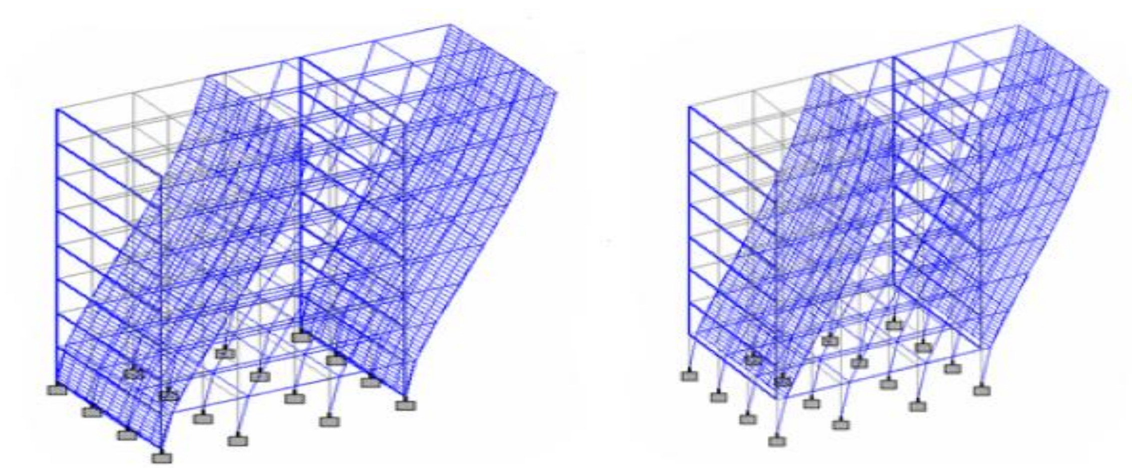


Fig 5.5. (a) *Mode shape 1*

The building discussed above using mode shape (a) with continuous shear wall behave as a single unit during earthquake. It swings back and forth and less damage occurs during strong ground motions. But the building which behaves as soft storey due to discontinuity in the load path because shear wall does not goes all down to the ground faces lots of shaking during earthquakee. The columns of soft storey building faces more stresses during earthquake.

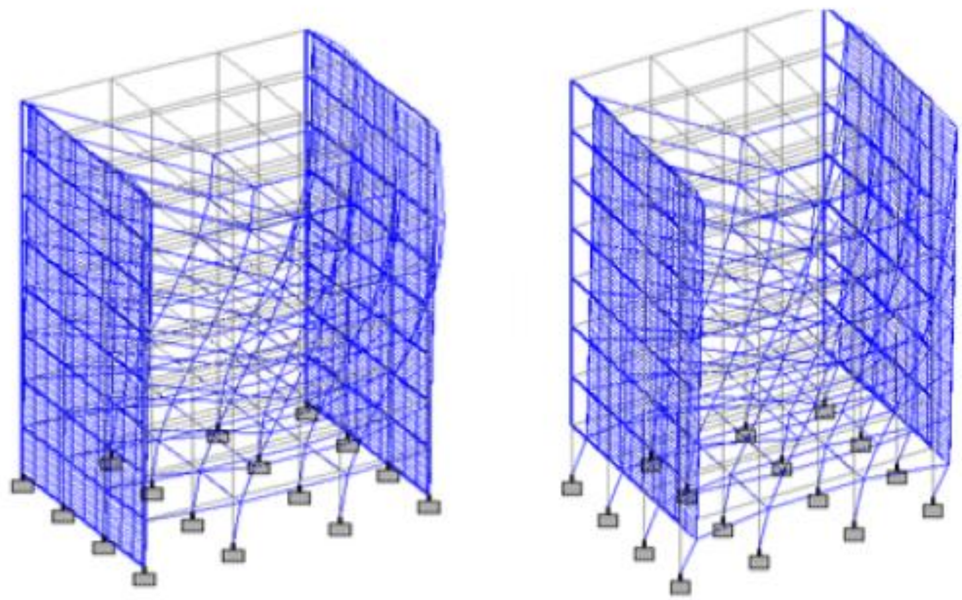


Fig 5.5 (b) *Mode Shape II*

The building with continuous shear wall have less drift discussed above using mode shape (b) but building with soft storey have large drift. Drift is the extent to which a building bends or sways. Limits are often imposed on drift so a building is not designed to be *so* flexible that the resulting drift or swaying during an earthquake causes excessive damage. If the level of drift is too high, a building may pound into the one next to it

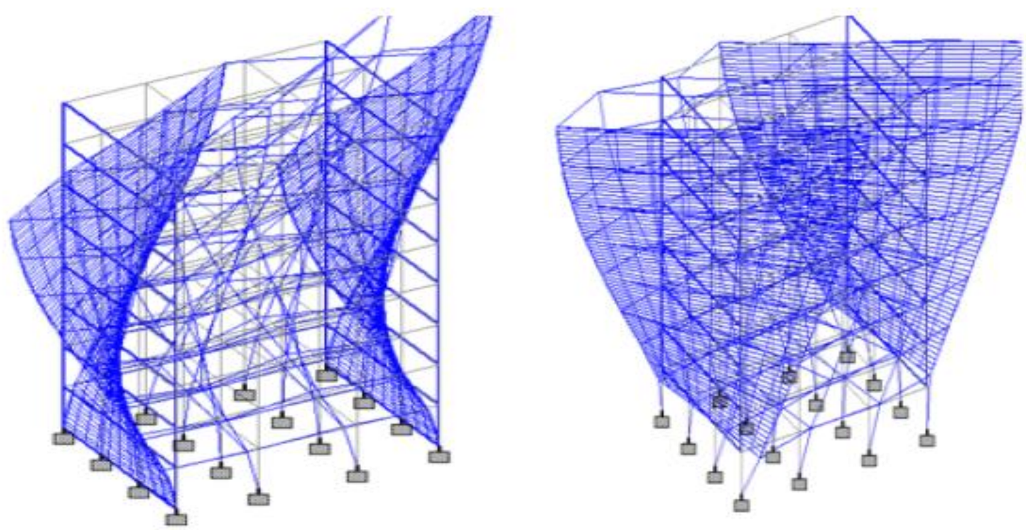


Fig. 5.5. (c) *Mode Shape III*

Twist in buildings, called torsion by engineers, makes different portions at the same floor level to move horizontally by different amounts. Mode shape (c) This induces more damage in the frames and walls on the side that moves more. Many buildings have been severely affected by this excessive torsional behaviour during past earthquakes. It is best to minimize (if not completely avoid) this twist by ensuring that buildings have symmetry in plan (i.e., uniformly distributed mass and uniformly placed lateral load resisting systems).

5.6. Comparison of Inter Storey Storey Drift

Drift in building frames is a result of flexural and shear mode contributions, due to the column axial deformations and to the diagonal and girder deformations, respectively. In low rise braced structures, the shear mode displacements are the most significant and, will largely determine the lateral stiffness of the structure. In medium to high rise structures, the higher axial forces and deformations in the columns, and the accumulation of their effects over a greater height, cause the flexural component of displacement to become dominant. The building without soft storey is shown in table 5.6.1 and wit soft storey shown in table no. 5.6.2.

Table 5.6.1. *Inter storey drift of building with continuous shear wall*

STORY	HEIGHT	LOAD	AVG. DISP(CM)		DRIFT(CM)

	(METE)		X	Z	
BASE=	0.00				
1	0.00	1	0.0000	0.0000	0.0000
		2	0.2520	0.0005	0.0000
		3	0.3277	0.0006	0.0000
		4	-0.3277	-0.0006	0.0000

2	0.30	1	0.0000	0.0000	0.0000
		2	0.1895	0.0007	0.0625
		3	0.2464	0.0009	0.0813
		4	-0.2464	-0.0009	0.0813
3	0.60	1	0.0000	0.0000	0.0000
		2	0.1913	0.0005	0.0018

Table no. 5.6.2.*Inter storey drift of building with discontinuous shear wall*

STORY	HEIGHT	LOAD	AVG. DISP(CM)		DRIFT(CM)

	(METE)		X	Z	X
BASE=	0				
1	0.00	1	0.0000	0.0000	0.0000
		2	0.0000	0.0000	0.0000
		3	0.0000	0.0000	0.0000
		4	0.0000	0.0000	0.0000
2	3.00	1	0.0000	0.0000	0.0000
		2	2.1054	0.0028	2.1054
		3	2.7370	0.0036	2.7370
		4	-2.7370	-0.0036	2.7370
3	3.30	1	0.0000	0.0000	0.0000
		2	2.5308	0.0025	0.4254
		3	3.2901	0.0032	0.5531
		4	-3.2901	-0.0032	0.5531

5.7. Limitation of drift

Deflections must be limited during earthquakes for a number of reasons, and hence provision of adequate stiffness is important. Relative horizontal deflections within the building (e.g. between one storey and the next, known as storey drift) must be limited. This is because non-structural elements such as cladding, partitions and pipe work must be able to accept the deflections imposed on them during an earthquake without failure. Failure of external cladding, blockage of escape routes by fallen partitions and ruptured firewater pipe work all have serious safety implications. Moreover, some of the columns in a building may only be designed to resist gravity loads, with the seismic loads taken by other elements, but if deflections are too great they will fail through 'P–delta' effects however ductile they are.

CHAPTER 6

RESULTS OF CAPACITY BASED DESIGN

6.1 Capacity Based Design

The basic concept of capacity based design of structures is the spreading of inelastic deformation demands in a structure in such a way so that the formation of plastic hinges takes place at predetermined positions and sequences. In multistory multi bay reinforced concrete frames plastic hinges are allowed to form only at the ends of the beams .To achieve this flexural capacity of column sections at each joint are made more than the joining beam sections. This will eliminate the possible sway mechanism of the frame. The capacity design is also the art of avoiding failure of structure in brittle mode . This can be achieved by designing the brittle modes of failure to have higher strength than ductile modes. Shear failure is brittle mode of failure hence shear capacity of all components are made higher than their flexural capacities

Step by Step Procedure for Capacity Based Design

1. Design loads i.e. dead loads, live loads and earthquake loads are calculated.

2. Seismic analysis of the frame for all load combination specified in IS 1893(Part I):2002 are done.

3. Members are designed (as per IS 456:200) for maximum forces obtained from all load combinations. Beams are designed for maximum sagging and maximum hogging moments. Provided reinforcements are calculated following the norms given in code. Columns are designed for the combination for moment and corresponding axial force providing maximum interaction effect i.e. considering the eccentricity.

4. The flexural capacities of the beams under sagging and hogging condition for the provided reinforcements are calculated.

5. The flexural capacity of columns at a joint is compared with actual flexural capacity of joining beams. If the sum of capacities of columns is less than the sum of capacities of beams multiplied by over strength factor, the column moments should be magnified by the factor (moment

magnification factor) by which they are lacking in moment capacity over beams. If the sum of the column moments is greater than sum of beam moments,
there is no need to magnify the column moments.

6. Columns are designed for the revised moments and the axial force coming on it from the analysis.

7. Shear capacity of beams are calculated on the basis or their actual moment capacities and shear reinforcements are calculated.

8. Similarly shear capacity of column is calculated on the basis of magnified moment capacities. Then the columns are designed for shear.

6.2. Analysis

The dead load and the imposed loads are calculated for the floors and the roof as shown in below table 6.2. The calculated values are as follows:

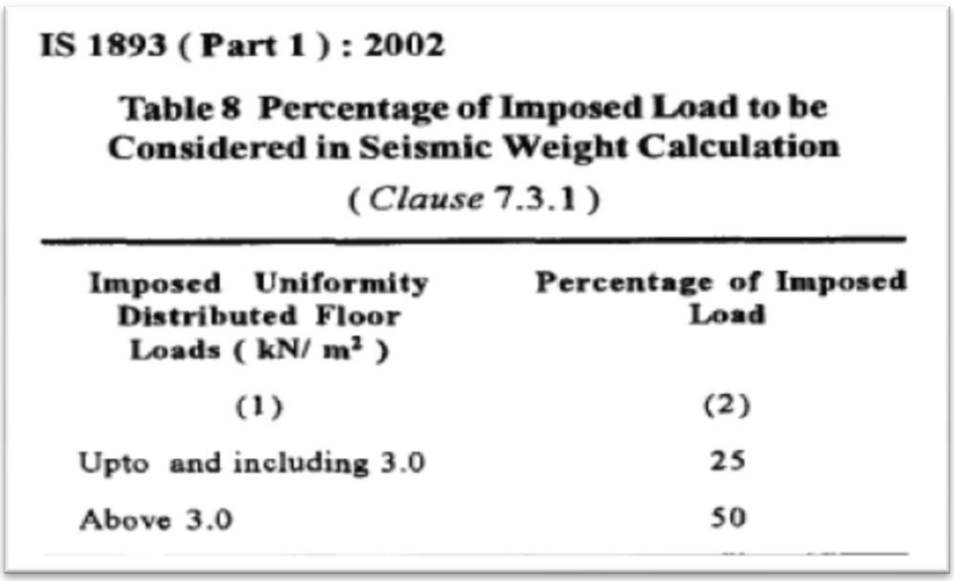
Table 6.2. *Load calculations*

FOR EXTERNAL BEAMS	FOR INTERNAL BEAMS
Total dead load on the roof = 13.684 kN/m	Total dead load on the roof =8.375 kN/m
Total dead load on the floors = 24.86 kN/m	Total dead load on the floors= 21.15 kN/m
Total imposed load on the roof =3.32 kN/m	Total imposed load on the roof = 1.75 kN/m
Total imposed load on the floors= 7.74 kN/m	Total imposed load on the floors= 4.00 kN/m

6.3. Seismic Analysis

For calculating the design seismic forces of the structure the imposed load on the roof need not be considered. According to is 1893:2002 the percentage of load taken is given below in table 6.3.

Table 6.3. *Percentage of live load.*



IS 1893 (Part 1) : 2002

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

(*Clause* 7.3.1)

Imposed Uniformity Distributed Floor Loads (kN/m^2)	Percentage of Imposed Load
(1)	(2)
Upto and including 3.0	25
Above 3.0	50

6.3.1 Input data for seismic analysis

- Zone = 4 , Zone factor =0.24
- Height of the building = 14.05m
- No. of floors = 4
- Importance factor = 1
- R=5 (special RC moment resisting frame)

The weight of each floor

- First floor = 6622kN
- II floor = 6578kN
- III floor = 6578kN
- Roof level = 5074Kn

Calculated Base shear

First floor	Q = 6.00 kN
Second floor	Q = 20.00kN
Third floor	Q = 42.50kN
Roof level	Q = 56.50kN

6.4. Dimensioning and Numbering

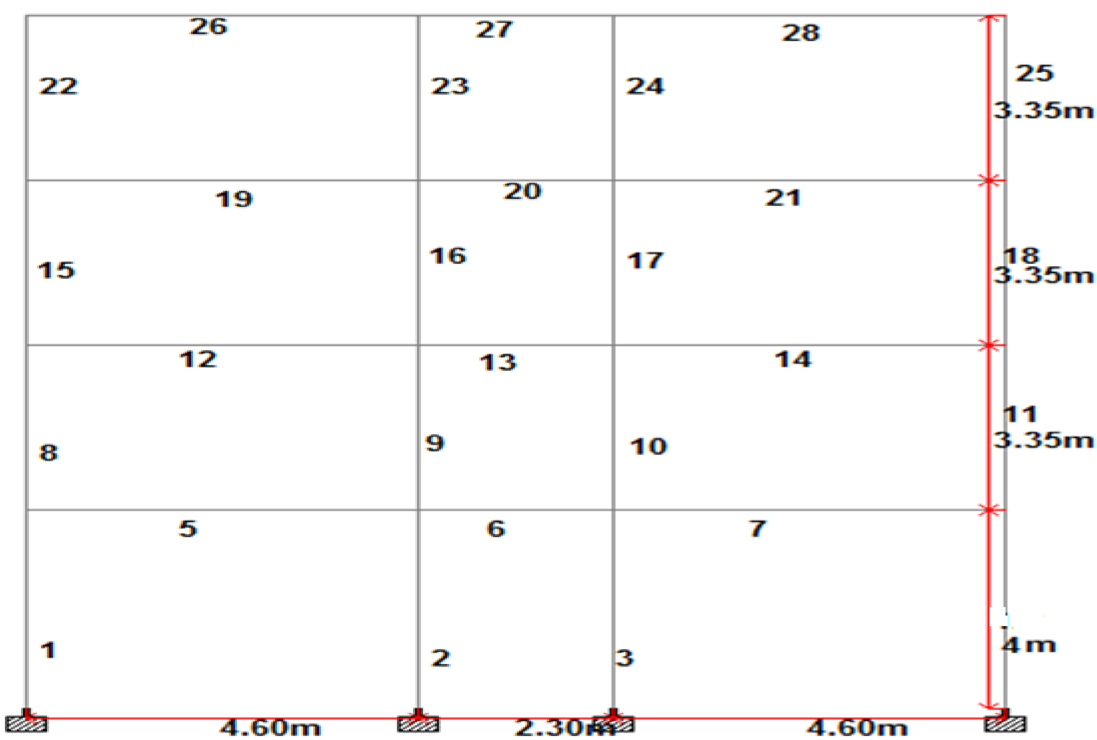


Fig.6.4 *Frame of RC (G+3) building with Dimensioning and numbering*

Using the above data, analysis of the frame is carried out with all the load combinations as per IS 1893(Part 1):2002. The maximum moments and forces for the beams and columns for all the load combinations for each member is considered for the design. The different load combinationsare:

1. 1.5(DL+IL)

2. 1.2(DL+IL+EL)

3. 1.2(DL+IL-EL)

4. 1.5(DL+EL)

5. 1.5(DL-EL)

6. 0.9DL+1.5EL

7. 0.9DL-1.5EL

6.5. Results Calculated for Maximum Hogging and Sagging Moments

The maximum hogging and sagging moments are calculated for all load combinations. Also, an interior frame 4-4 is considered for the analysis and design. After the calculations of all moments for all load combinations maximum hogging and sagging moments are considered The maximum hogging and sagging moments of beams are plotted in the model. The maximum hogging moments are shown above and sagging moments are shown below in the STAAD model of G+3 RC building with soft storey. The calculated moments for all load combinations are taken from STAAD output file and are shown below in tabular form. In hogging moment capacity calculation, the top face reinforcement in tension and bottom face reinforcement in compression , therefore hogging capacity of beam section will be calculated on the basis of reinforcement section while in sagging capacity calculation, the top face reinforcement in compression and bottom face reinforcement in tension ,flange action of slab will also be taken into account. Hence, sagging capacity of beam will be calculated on the basis of T- section as shown in below fig 6.5.

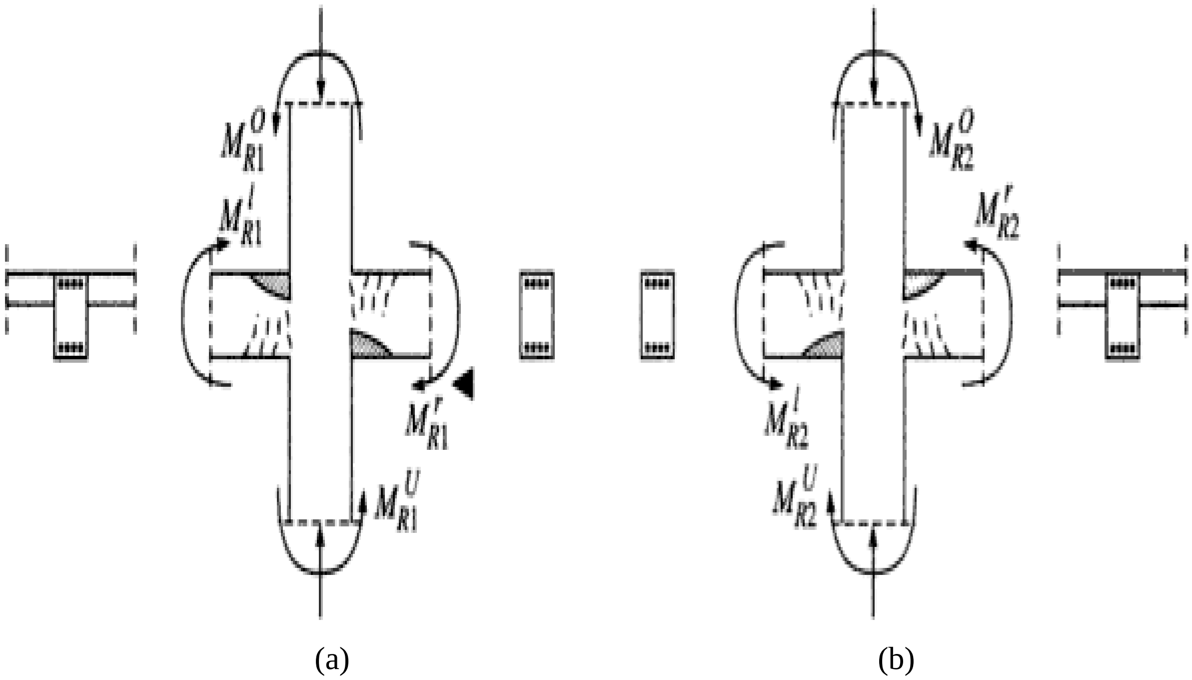


Fig.6.5. *Moment capacity verification of columns at any joint in two seismic directions*

(a) End moment capacity of beams at a joint in seismic direction 1 (loading from right to left)

(b) End moment capacity of beams at a joint in seismic direction 2 (loading from left to right

The all moments are taken from STAAD.Pro.V8i output file for all 28 beams. The moments for all load combination are calculated and given in Appendix A. On the basis of this the hogging and sagging moments are calculated and plotted below in fig .6.6.

6.6. Maximum Hogging above and Sagging below Moments

The hogging and sagging moments are plotted below for the 28 beams and shown in fig.6.6. the hogging is above and sagging is below.

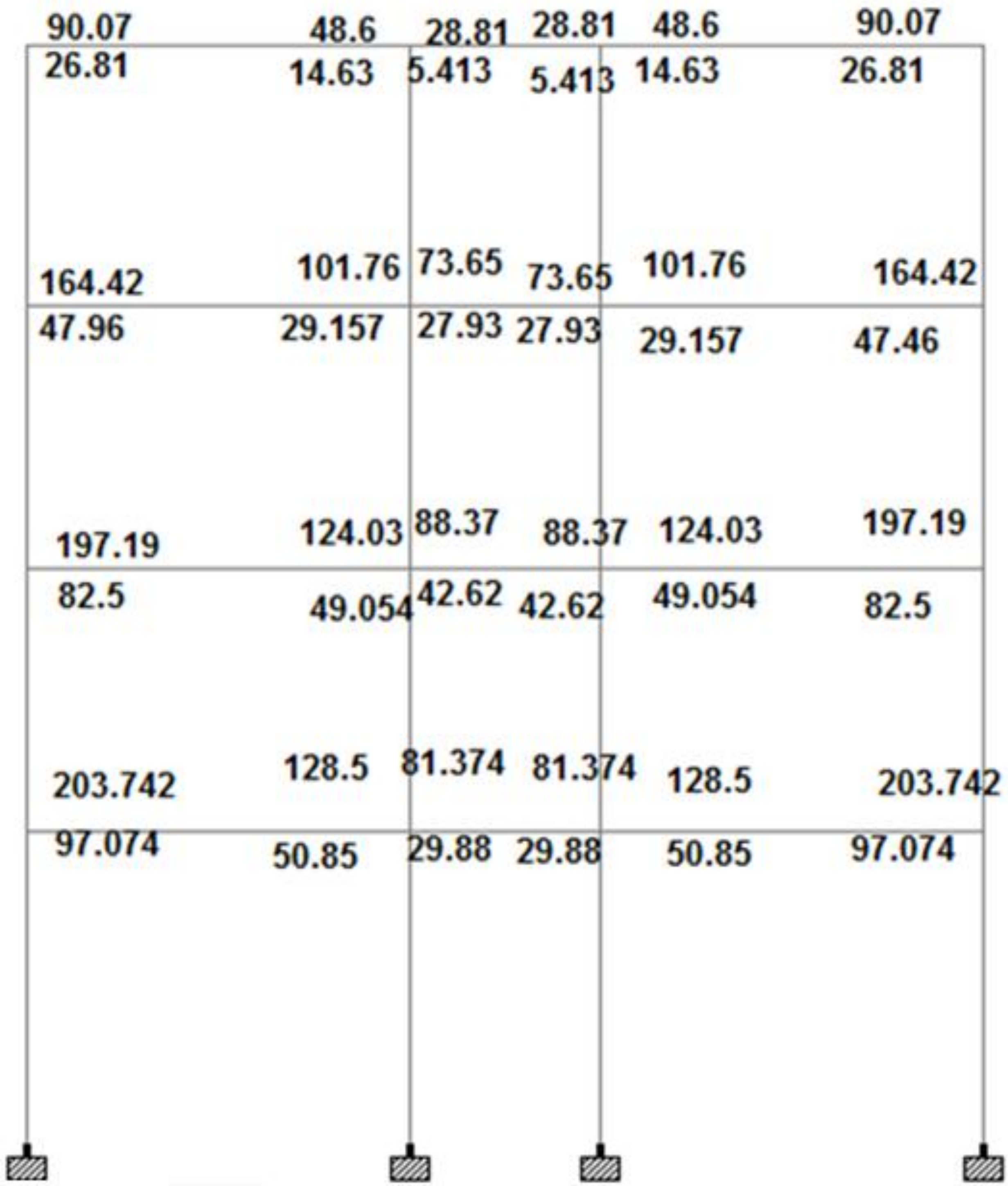


Fig.6.6 *Maximum hogging and sagging moments of beams.*

6.7. Calculation of Longitudinal Reinforcements Provided for the Beams

In this thesis the beam no. 5 is taken as an example and longitudinal reinforcements are calculated for this beam no.5 and shown in below table 6.7.

Table 6.7 *Calculated longitudinal reinforcements*

The size of the beam = 300 X 450

Effective depth d= 450-25-(25/2) =421.5mmφ

Effective cover d1= 25+(25/2)

Using sp 16 table D we get the value of Mu,lim/bd2

For fe 415 and fck 20 the value of Mu,lim/bd2= 2.76

Mu,lim = 2.76 X b X d

Mu,lim=140.88 kN-m

Hogging Moment : Lets consider the beam 5, the value of bending moment at 5 is

M=203.745kN-m

M>Mu,lim, hence it is doubly reinforced.

We get d1/d = 0.1,

Now,M/bd^2 = 203.745/300 X 421.5^2

M/bd^2 =3.99

Using, Sp16 table no. 50 for doubly reinforced section

The percentage of reinforcements at top and bottom are PT=1.337 and PB=0.401 Eq I

Sagging Moment : for beam no. 5 is M1= 97.074 kN-m

M1<Mu,lim , hence singly reinforced.

For fe 415 and fck 20 the value of PT is 0.96

$M1/bd^2$=1.90

From table 2 , SP 16:1980 ,we get PB= 0.602 PT 0.96 Eq II

From Equation one and two we get,the maximum values are taken and the reinforcement is found

Required Ast (top) =1654.53 mm2 , Ast(bottom)= 744.975 mm2

The Provided reinforcements are

Top, Ast = 1884.9 mm2, 6 bars @20mmφ and bottom, Ast=804.25, 4bars @16mmφ .

Moment capacities of the beams are calculated from the reinforcements provided

6.8. Calculation of Moment Capacities of Beams

6.8.1. Hogging Moment Capacity

The hogging moment capacity for beam no. 5 is calculated and shown in below table 6.8.1 and similarly other moment capacities are also calculated like this.

Table 6.8.1. *Hogging moment capacity*

Mu,lim = 0.36 X (Xu,max/d)[1-0.42(Xu,max / d)]bd^2fck = 140.3 kN m

Steel corresponding to this moment, Ast1= (0.48 X 0.36 X fck X bd)/0.87fy = 1184.54 mm2

Considering the beam 5, Available Ast2= Ast-Ast1

=1884.95-1184.54 = 700.41 mm2

Additional moment capacity due to available compression steel

M2= Asc X fsc(d-d1)/106 =106.46kN-m

(Asc X fsc)/(0.87 X fy) = 786.3mm2

But available Ast2 = 700.41

Hence flexural moment contribution for 700.41mm2 is ,

M2= 0.87 X 415 X 700.41 X (412.5-(25+25/2))/106 = 94.83 kN-m

Total hogging moment capacity,

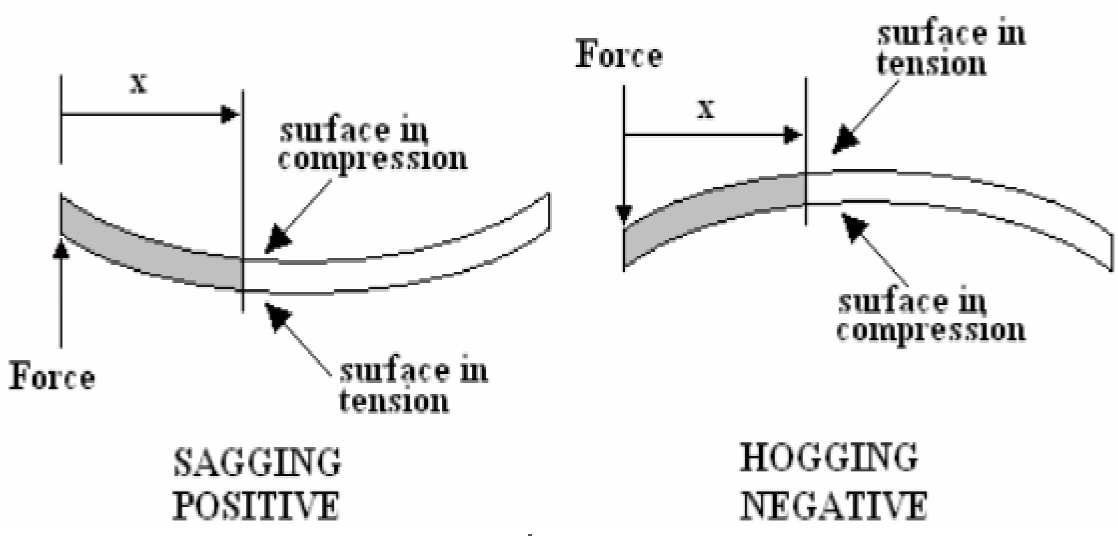
M = MU,lim+M2 = 233.75 kN-m

If the value of Ast <Ast1,

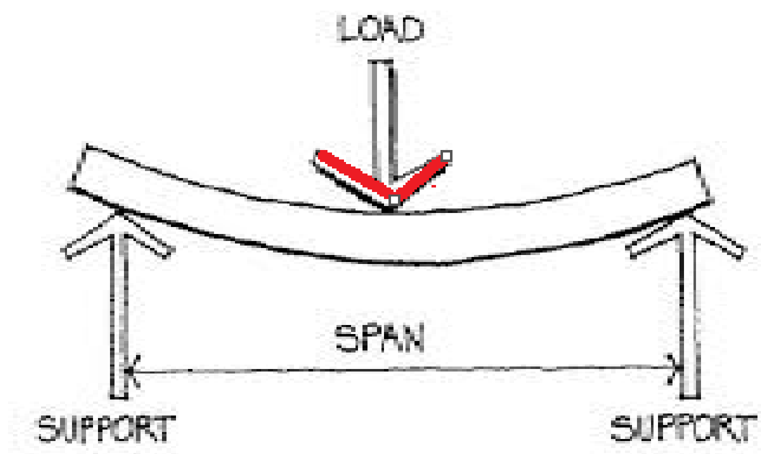
$M = 0.87fy(Ast/bd)[1-(1.005fck/fy)(Ast/bd)]bd^2$

6.8.2. Sagging Moment Capacity

The sagging action of the beam near supports will cause the monolithically constructed slab which act as the flange of T-beam, contributing additional compressive force, thus increasing the flexural capacity as shown in fig 6.8.2. (a) and (b). The numerical value of the bending moment will be the same but in this case the anti-clockwise moment produces sagging and the clockwise moment produces hogging.



(a)



((b) *Sagging actions in beam*

Fig.6.8.2. *Hogging and Sagging in beams*

Table 6.8.2. *Sagging moment capacity*

The sagging action of the beam near supports will cause the monolithically constructed slab to act as the flange of T-beam, contributing additional compressive force, thus increasing the flexural capacity.

Ast=804.25mm2

Bf =lo/6 + bw+6Df =1556.67mm

Xu = (0.87 X fy X Ast)/(0.36 X fck X bf)=25.9<120

Mu= 0.87 X 415 X Ast X d[1-(Ast X fy/bf X d X fck)]/106 = 116.7kN-m

6.9. Plotted Hogging and Sagging Capacities of Beams

The hogging and sagging capacities of beams are calculated manually but in this thesis, beam no.5 is taken as example. Similarly all other capacities of beams are calculated and shown in below fig.6.9. The flexural capacities of the beams under sagging and hogging condition for the provided reinforcements are calculated.

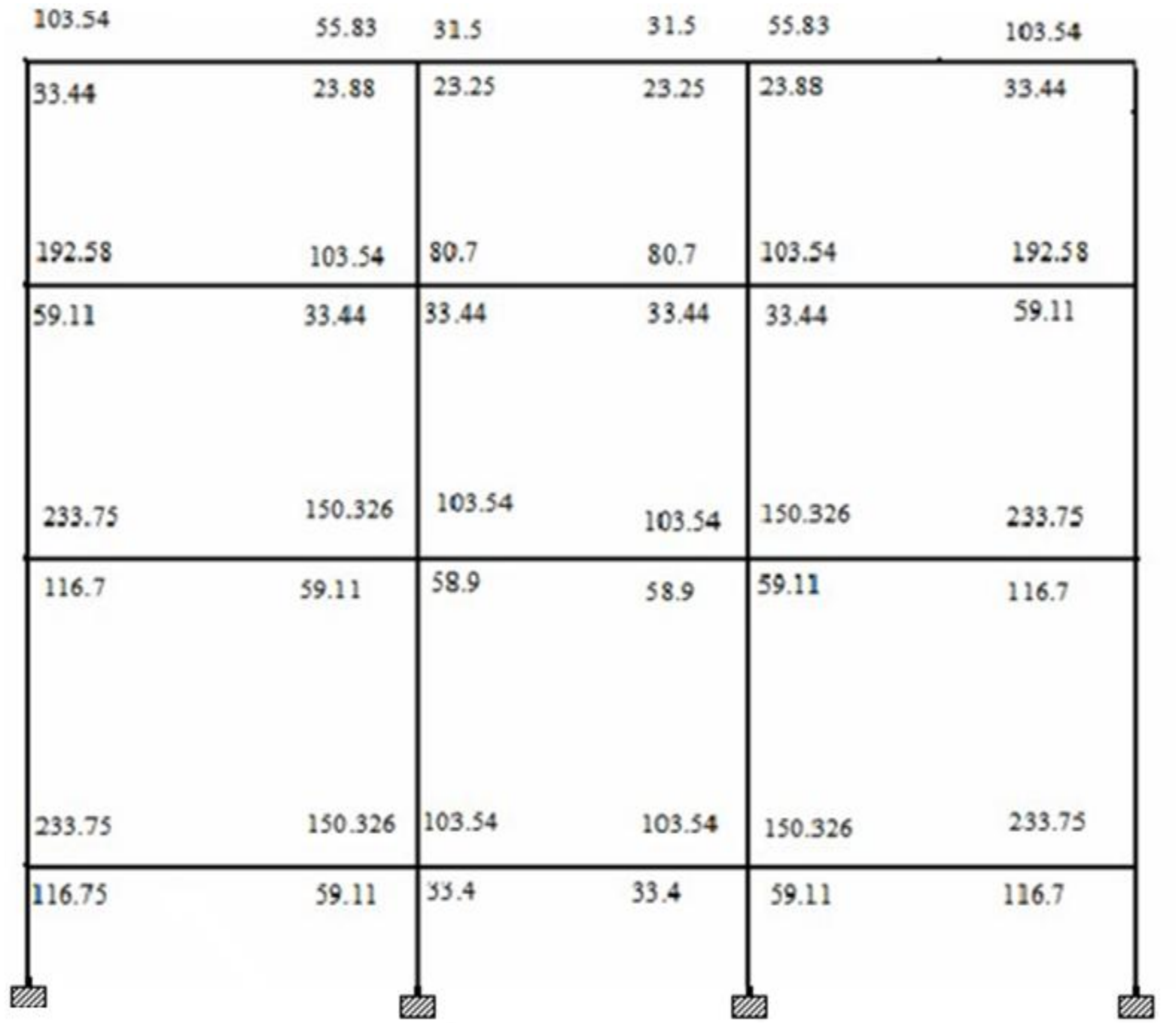


Fig.6.9.*Hogging and Sagging moment capacities of beams*

6.10. Plastic Hinge Mechanism in Reinforced Concrete Beams

To eliminate the possibility of the column sway mechanism during the earthquake, it is essential that the plastic hinges should be formed in the beams is shown in fig.6.10. This is achieved after capacity verification of columns with capacity of beams at every joint of the frame. The amount, by which the design moments of columns at a joint are to be magnified, is achieved by the magnification factor determination at that particular joint.

- Two forms of plastic hinge develop in beams subjected to seismic actions , with the type of plastic hinge depending upon the relative magnitudes of the seismic and gravity loads which act. Where the gravity loadings dominates, as illustrated in below fig.6.11 (a) as the structures sways backwards and forwards negative moment plastic hinges develop in the beam at the column faces and positive moment plastic hinges in the span of the beam. With each inelastic displacement the vertical deflection of the beam increases and the inelastic rotations sustained by the plastic hinges increase in magnitude. As each of these zones sustain inelastic rotations in one direction only, they are referred to as uni-directional plastic hinges. The load deflection characteristics of structures which for uni-directional plastic hinges.

- The situation where the seismic actions dominates as shown in fig (b). In this case as the structure sways backwards and forwards , negative and positive moment plastic hinges form in the beam at the column faces, with the direction of the rotation in each of these reversing with the direction of motion. These are referred to as reversing plastic hinges.

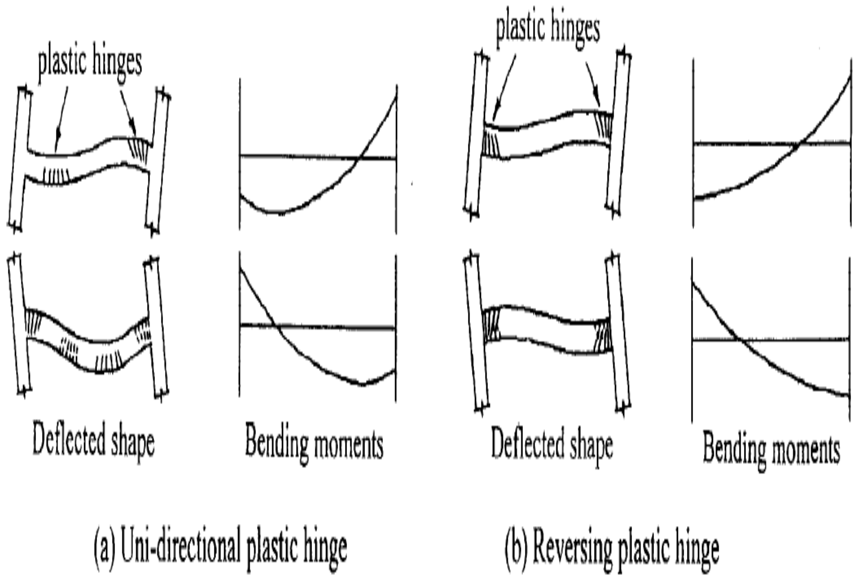


Fig.6.10. *Formation of unidirectional and reversing plastic hinges*

The behavior of the reversing plastic hinges is in sharp contrast to that of uni-directional plastic hinges. With reversed inelastic cyclic loading either the top or the bottom reinforcement yields in tension in one half cycle and it does not completely yield back in compression in the next half cycle. The flexural cracks remain open adding significantly to the elongation of the plastic hinges. The failure of the cracks to close in the compression zone, arises from the dislocation of the aggregate particles at the crack faces and from the truss like action associated with the shear resisting mechanism. The elongation that develops in reversing plastic hinges before strength degradation is typically of the order of 2 to 4 percent of the member depth. In addition significant shear deformation also occurs. This reduces the stiffness at low load levels , causing the characteristics pinched load deflection relationship.

6.11. Determination of Moment Magnification Factor

The sum of the resisting moments of the columns, taking into account the action of axial forces should be greater than the sum of resisting moments of all adjacent beams for each seismic action.

$$|M^{O}_{R1}| + |M^{U}_{R1}| >= \lambda_{Rd} |M^{l}_{R1}| + |M^{r}_{R1}|$$

$$|M^{O}_{R2}| + |M^{U}_{R2}| >= \lambda_{Rd} |M^{l}_{R2}| + |M^{r}_{R2}|$$

Where λ Rd is the factor which takes into account the variability of the yield stress fy and the probability of strain hardening effects in the reinforcement or is known as the over strength factor. It is taken according to EC8 for seismic ductility class high. Therefore, the capacity based design is satisfied if the columns are designed for the following moments:

$$M_{s1,cd} = \alpha_{cd\,1} M_{s1}$$

$$M_{s2,cd} = \alpha_{cd2} M_{s2}$$

$$\text{Where } \alpha_{cd} = \lambda_{Rd} [|M^{l}_{R}| + |M^{r}_{R}| / |M^{O}_{R}| + |M^{U}_{R}|]$$

MlR1,2, MrR1,2, MOR1,2,MUR1,2 are the resisting moments of the left and right beams and design moments of the over and under columns at joint in seismic directions 1 and2. α cd isthe moment magnification factor and Ms,cd is the magnified moment of the column at that joint. If the sum of column moments is greater than that of the beams, there is no need to magnify the column moments. The magnification factor in such case is taken as unity.

6.11.1. Moment Magnification Factor at all Joints

In case of zone 4 the over strength factor is taken as 1.35 and shown in below table 6.11.If I >II then, the moment magnification factor is taken as unity 1.

Table 6.11. *Moment Magnification factors*

Joint no.	Seismic Direction	Sum of resisting moments of top and bottom at a joint (I)	Sum of resisting moments of right and left of beams with over strength factor 1.35 (II)	Check for I >II	Moment magnifiction factor II/I
5,8	1	203.543	128.5*1.35 = 173.475	Ok	1
	2	203.543	315.56	not ok	1.55
6,7	1	145.513	248.02	not ok	1.71
	2	145.513	219.58	not ok	1.51
9,12	1	197.194	157.545	Ok	1
	2	197.194	315.56	not ok	1.6
10,11	1	155.23	282.45	not ok	1.82
	2	155.23	219.58	not ok	1.4
13,16	1	164.425	79.8	Ok	1
	2	164.425	259.98	not ok	1.58
14,15	1	117.78	184.87	not ok	1.57
	2	117.78	154.035	not ok	1.31
17,20	1	90.071	45.144	Ok	1
	2	90.071	139.8	not ok	1.55
18,19	1	62.259	74.76	Ok	1.2
	2	62.259	106.76	Ok	1.7

After obtaining the magnification factors, the flexural strengths are to be increased accordingly at every joint and the maximum revised moment from top and bottom is to be considered for design and the axial load obtained from analysis.

6.12. Determination of Longitudinal Reinforcements in Columns

The longitudinal reinforcements in columns are calculated below and shown in below fig.6.12.

Table 6.12. *Longitudinal reinforcements in columns*

Columns size = 300x530mm
Effective cover d1= 40 +(20/2)=50 mm (d1/D = 0.1)
Effective depth d =530-50=480mm
Considering the columns 1&4 ,
Axial force Pu= 464.210KN
Maximum moment Mu= 208.96 KN-m
PU/fckbD = 0.146
MU/fckbD^2= 0.124
From chart 44 of SP 16:1980 (d1/D=0.1 , fy=415)
We get, p/fck = 0.07
Thus Ast = 2226mm^2, provide 8no.s of 20mmφ bars. In this way the vertical reinforcements are calculated

6.13. Capacity Design for Shear in Beams

The design shear forces in beams are corresponding to the equilibrium condition of the beam under the appropriate gravity load and to end resisting moments corresponding to the actual reinforcement provided, further multiplied by a factor γ Rd. This factor compensates the partial safety factor applied to yield strength of steel and to account the strain hardening effects. Generally this value is taken as 1.25. The shear force is in both the directions is determined by the following equation.

$$V_{A,S1} = wl/2 - \gamma_{Rd} (M_{AR}+M^1_{BR})/l$$

$$V_{B,S1}= wl/2+ \gamma_{Rd} (M_{AR}+M^1_{BR})/l$$

$$V_{A,S2} = wl/2 + \gamma_{Rd} (M^1_{AR}+M_{BR})/l$$

$$V_{B,S2} = wl/2 - \gamma_{Rd} (M^1_{AR}+M_{BR})/l$$

Where MAR,MBR,M1AR,M1BR are the actual resisting moments at hinges and γ Rd is the amplification factor , w comprises of the dead and live load. The reinforcement is determined for the maximum of the above four at a particular joint, given in below table 6.13.

Table 6.13. *Maximum shear force in beams*

BEAM NO.	MAXIMUM SHEAR FORCE
5,7	95.88
6	86.975
12 , 14	95.88
13	100.8
19,21	77.72
20	74.76
26.28	43.125
27	34.82

For the above Table 6.13. the following reinforcements are provided :

Provide a nominal reinforcement of 2 legged, 8mmφ @250mmc/c in all the beams.

Provide special confinement reinforcement at the joint for a length of 2d = 2 X 421.5 ≈830mm.

Provide 8mmφ @100mm c/c.

6.14. Determination of Shear Force in Columns

The capacity design shear forces are evaluated by considering the equilibrium of column under the actual resisting moments at the ends and is shown in below fig. 6.15. It is given by: V= λ

Rd (MC,Rd + MD,Rd)/l , where MC,Rd and MD,Rd are the flexural capacities of the end sections, l is the clear height of the column and λ Rd = 1.35.

Table 6.14. *Shear capacity of columns in the frame*

COLUMN NO.	CAPACITY BASED SHEAR
1,4	141.05
2,3	67.44
8,11	150.96
9,10	125.31
15,18	148.387
16,17	216.69
22,25	112.51
23,24	64.276

For the above Table 6.14. provide a nominal reinforcement of 8mm ϕ 2 legged stirrups @ 300mm c/c for columns.

Special Confining Reinforcement

According to IS 13920 : 1993 DUCTILE DETAILING OF REINFORCED CONCRETE STRUCTURES SUBJECTED TO SEISMIC FORCES — CODE OF PRACTICE

This requirement shall be met with, unless a larger amount of transverse reinforcement isrequired from shear strength considerations. Special confining reinforcement shall be provided over a length *lo* from each joint face, towards mid span, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces . The length '*lo*' shall not be less than (a) larger lateral dimension of the member at the section where yielding occurs, (b) 1/6 of clear span of the member, and (c) 450 mm.

In this thesis, the above data is provided over a length of Lo towards mid span of column.

Lo > D

Lo > *l/6* whichever is greater = 600 mm

Lo > 450mm

The spacing should not exceed 100mm and should be greater than ¼ of minimum

dimension. S> 75mm and <100mm.

Minimum area of cross section of hoop reinforcement,

A=0.18 X S X h X (fck/fy) X (Ag/Ak-1) =0.18 X 75 X 220 X (20/415) X (530 X 300)/(480 X 240)-1)

= 54.54 mm2.

Use 10mm ϕ bars at a spacing of 90mm c/c.

CHAPTER 7

CONCLUSIONS

Soft storey is not a good practice but today many building structure having parking or commercial areas in their first stories, may suffer major structural damages and collapsed during major earthquakes. The behavior of soft storey during earthquakes must be controlled if building with soft storey is designed earthquake resistant using codal provisions. IS I893:2002, provides following remedies for soft storey:

If building with a flexible storey such as the ground storey consisting of open spaces for parking that is stilt buildings, special arrangements needs to be made to increase the lateral strength and stiffness of the soft / open storey. Dynamic analysis of building is carried out including the strength and stiffness effects of infills and inelastic deformations in the members designed accordingly. Alternatively, the following design criteria are to be adopted after carrying out the earthquake analysis, neglecting the effect of infill walls in other storeys:

a) The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads.
b) Besides the columns designed and detailed for the calculated storey shears and moments , shear walls placed symmetrically in both directions of the building as far away from the centre of the building as feasible: to be designed exclusively for 1.5 times the lateral storey shear calculated as before.

7.1. Conclusions

Case 1

The building with continued shear wall up to foundation performs better during an earthquake. But the building with discontinued shear wall which makes a soft storey, may fail during an earthquake. The soft storey phenomenon shows that the flexibility of first storey results in extreme deflections which in turn, lead to concentration of forces at the second storey and in connections due to large plastic deformations. The following can be concluded from the present work:

1. The building with continuous shear wall have frequency for mode 1 as 0.312 Cycle/sec and for mode 6 is 1.480 cycle/sec and the building with discontinuous shear wall have frequency for mode 1 as 0.308 cycle/sec and for mode 6 as 1.395.

2. The time period of building with continuous shear wall in which there is no discontinuity in load path for mode 1 is 3.203 seconds and for mode 6 is 0.675 seconds. On the other hand the building with discontinuous shear wall for mode 1 is 4.24 seconds and for mode 6 is 0.916 seconds.

3. The base shear of building with continuous shear wall, Total SRSS Shear is 2006.40 KN and the building with discontinued shear wall, Total SRSS Shear is 1881.17 KN. The seismic weight of building with shear wall is more because the shear wall goes all the way down to the ground instead of discontinuous shear wall.

4. Peak storey shear for 1st and 8th floor for building with continuous shear wall are 2006.40 KN and 1130.65 KN . On the other hand the building with discontinued shear wall, the peak storey shear for 1^{st} floor and 8^{th} floor are 1881.17 KN and 1061.10 KN.

5. The inter storey drift in case of building with continuous shear wall, the value of drift for all load combinations is 0.0813 in second storey. On the other hand the building with discontinuous shear wall the value of drift in second storey is 2.73.

6. The member forces in case of building with continued shear wall for member 1

Axial force	Shear force	Torsion	Moment
216.09	144	5.15	441.62

The building with discontinuous shear wall the member forces are for member 1 have following values

Axial force	Shear force	Torsion	Moment
395.71	200	7.24	500.92

7 From the above conclusions, it is concluded that the building in which the shear wall goes all the way down to the ground and there is no discontinuity in the load path performs good during earthquake because the member forces, time period inter storey drift are less as compared to the building with discontinued shear wall which makes the building soft during an earthquake.

Case 2

1. Capacity based design is done which deals with strong column weak beam concept and on the basis of this concept an example is taken of G+3 storey and it is designed.

2. The flexural capacities are calculated and the flexural capacities of columns are greater as compared to beams.

Joint	Seismic direction	Flexural capacity of column	Flexural capacity of beam
5,8	1,2	203.543	173.475

3. Required Ast (top) =1654.53 mm2 , Ast(bottom)= 744.975 mm2 and the Provided reinforcements are Top, Ast = 1884.9 mm2, 6 bars @20mmφ and bottom, Ast=804.25, 4bars @16mmφ **.**

4. The shear capcities of columns and beams are increased in order to avoid the shear modes of failure

5. The conventional design methods for earthquake resisting structures although this method is little costlier but is more effective in resisting the earthquake forces.

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APPENDIX A

The analysis of the frame is done by STAAD PRO-V8i. The results are for the calculation of maximum hogging and sagging moments are as follows and the maximum values in all combinations are considered:

Beam	L/C	Node	Fx kN	Fy kN	Fz kN	Mx kNm	My kNm	Mz kNm
1	5 CASE1	1	421.606	-12.993	0	0	0	-17.224
		5	-421.606	12.993	0	0	0	-34.747
	6 CASE2	1	224.9	43.067	0	0	0	143.069
		5	-224.9	-43.067	0	0	0	29.198
	7 CASE3	1	449.254	-63.82	0	0	0	-170.438
		5	-449.254	63.82	0	0	0	-84.843
	8 CASE4	1	183.769	56.915	0	0	0	182.92
		5	-183.769	-56.915	0	0	0	44.741
	9 CASE5	1	464.21	-76.694	0	0	0	-208.964
		5	-464.21	76.694	0	0	0	-97.811
	10 CASE6	1	54.069	60.879	0	0	0	188.176
		5	-54.069	-60.879	0	0	0	55.342
	11 CASE7	1	334.51	-72.729	0	0	0	-203.708
		5	-334.51	72.729	0	0	0	-87.209

	5							
2	CASE1	2	518.158	3.081	0	0	0	4.067
			-					
		6	518.158	-3.081	0	0	0	8.256
	6							
	CASE2	2	409.868	24.026	0	0	0	47.642
			-					
		6	409.868	-24.026	0	0	0	48.46
	7							
	CASE3	2	419.601	-19.087	0	0	0	-41.118
			-					
		6	419.601	19.087	0	0	0	-35.231
	8							
	CASE4	2	402.847	29.23	0	0	0	58.493
			-					
		6	402.847	-29.23	0	0	0	58.425
	9							
	CASE5	2	415.013	-24.661	0	0	0	-52.457

	5							
3	CASE1	3	518.159	-3.081	0	0	0	-4.067
			-					
		7	518.159	3.081	0	0	0	-8.256
	6							
	CASE2	3	419.601	19.087	0	0	0	41.118
			-					
		7	419.601	-19.087	0	0	0	35.231
	7							
	CASE3	3	409.868	-24.026	0	0	0	-47.642
			-					
		7	409.868	24.026	0	0	0	-48.46
	8							
	CASE4	3	415.013	24.661	0	0	0	52.457
			-					
		7	415.013	-24.661	0	0	0	46.189
	9							
	CASE5	3	402.847	-29.23	0	0	0	-58.493
			-					
		7	402.847	29.23	0	0	0	-58.425
	10							
	CASE6	3	251.545	25.573	0	0	0	53.66
			-					
		7	251.545	-25.573	0	0	0	48.632
	11							
	CASE7	3	239.379	-28.318	0	0	0	-57.29

	5							
4	CASE1	4	421.606	12.993	0	0	0	17.224
			-					
		8	421.606	-12.993	0	0	0	34.747
	6							
	CASE2	4	449.254	63.82	0	0	0	170.438
			-					
		8	449.254	-63.82	0	0	0	84.843
	7							-
	CASE3	4	224.9	-43.067	0	0	0	143.069

		8	-224.9	43.067	0	0	0	-29.198
	8 CASE4	4	464.21	76.694	0	0	0	208.964
		8	-464.21	-76.694	0	0	0	97.811
	9 CASE5	4	183.769	-56.915	0	0	0	-182.92
		8	-183.769	56.915	0	0	0	-44.741
	10 CASE6	4	334.51	72.729	0	0	0	203.708
		8	-334.51	-72.729	0	0	0	87.209
	11 CASE7	4	54.069	-60.879	0	0	0	-188.176
		8	-54.069	60.879	0	0	0	-55.342

5	5 CASE1	5	-17.065	117.755	0	0	0	87.572
		6	17.065	107.185	0	0	0	-63.262
	6 CASE2	5	-24.965	56.426	0	0	0	-39.687
		6	24.965	123.526	0	0	0	-114.642
	7 CASE3	5	4.904	131.912	0	0	0	179.576
		6	-4.904	48.04	0	0	0	13.33
	8 CASE4	5	-27.209	42.542	0	0	0	-70.336
		6	27.209	128.992	0	0	0	-128.499
	9 CASE5	5	10.128	136.899	0	0	0	203.742
		6	-10.128	34.635	0	0	0	31.465
	10 CASE6	5	-21.982	6.636	0	0	0	-97.074
		6	21.982	96.284	0	0	0	-109.116
	11 CASE7	5	15.355	100.993	0	0	0	177.005
		6	-15.355	1.927	0	0	0	50.849

6	5 CASE1	6	-12.201	43.384	0	0	0	41.217
		7	12.201	43.384	0	0	0	-41.217
	6 CASE2	6	-6.125	0.485	0	0	0	-6.376
		7	6.125	68.929	0	0	0	-72.335
	7 CASE3	6	-6.125	68.929	0	0	0	72.335
		7	6.125	0.485	0	0	0	6.376
	8 CASE4	6	-4.942	-6.294	0	0	0	-17.015
		7	4.942	79.261	0	0	0	-81.374
	9 CASE5	6	-4.942	79.261	0	0	0	81.374
		7	4.942	-6.294	0	0	0	17.015
	10 CASE6	6	-1.147	-20.887	0	0	0	-29.884
		7	1.147	64.668	0	0	0	-68.505
	11 CASE7	6	-1.147	64.668	0	0	0	68.505
		7	1.147	-20.887	0	0	0	29.884

	5							
7	CASE1	7	-17.065	107.185	0	0	0	63.262
		8	17.065	117.755	0	0	0	-87.572
	6							
	CASE2	7	4.904	48.04	0	0	0	-13.329
								-
		8	-4.904	131.912	0	0	0	179.576
	7							
	CASE3	7	-24.965	123.526	0	0	0	114.642
		8	24.965	56.426	0	0	0	39.687
	8							
	CASE4	7	10.128	34.635	0	0	0	-31.465
								-
		8	-10.128	136.899	0	0	0	203.742
	9							
	CASE5	7	-27.209	128.992	0	0	0	128.499
		8	27.209	42.542	0	0	0	70.336
	10							
	CASE6	7	15.355	1.927	0	0	0	-50.849
								-
		8	-15.355	100.993	0	0	0	177.005
	11							
	CASE7	7	-21.982	96.284	0	0	0	109.116
		8	21.982	6.636	0	0	0	97.074

	5							
8	CASE1	5	303.851	-30.057	0	0	0	-52.825
			-					
		9	303.851	30.057	0	0	0	-47.867
	6							
	CASE2	5	168.474	10.901	0	0	0	10.489
			-					
		9	168.474	-10.901	0	0	0	26.031
	7							
	CASE3	5	317.342	-58.916	0	0	0	-94.733
			-					-
		9	317.342	58.916	0	0	0	102.636
	8							
	CASE4	5	141.227	20.706	0	0	0	25.595
		9	-	-20.706	0	0	0	43.771

			141.227				
	9 CASE5	5	327.311	-66.566	0	0	0	-105.932
		9	-327.311	66.566	0	0	0	-117.063
	10 CASE6	5	47.433	29.897	0	0	0	41.732
		9	-47.433	-29.897	0	0	0	58.425
	11 CASE7	5	233.517	-57.374	0	0	0	-89.795
		9	-233.517	57.374	0	0	0	-102.409
9	5 CASE1	6	367.589	7.945	0	0	0	13.789
		10	-367.589	-7.945	0	0	0	12.827
	6 CASE2	6	285.858	42.866	0	0	0	72.557
		10	-285.858	-42.866	0	0	0	71.044
	7 CASE3	6	302.632	-30.116	0	0	0	-50.433
		10	-302.632	30.116	0	0	0	-50.457
	8 CASE4	6	280.149	51.497	0	0	0	87.088
		10	-280.149	-51.497	0	0	0	85.427
	9 CASE5	6	301.117	-39.731	0	0	0	-66.649
		10	-301.117	39.731	0	0	0	-66.45
	10 CASE6	6	163.983	49.153	0	0	0	83.016
		10	-163.983	-49.153	0	0	0	81.648
	11 CASE7	6	184.95	-42.075	0	0	0	-70.722

10	5 CASE1	7	367.589	-7.945	0	0	0	-13.789
		11	-367.589	7.945	0	0	0	-12.827
	6 CASE2	7	302.632	30.116	0	0	0	50.433
		11	-302.632	-30.116	0	0	0	50.457
	7 CASE3	7	285.858	-42.866	0	0	0	-72.557
		11	-285.858	42.866	0	0	0	-71.044
	8 CASE4	7	301.117	39.731	0	0	0	66.649
		11	-301.117	-39.731	0	0	0	66.45
	9 CASE5	7	280.149	-51.497	0	0	0	-87.088
		11	-280.149	51.497	0	0	0	-85.427
	10 CASE6	7	184.95	42.075	0	0	0	70.722
		11	-184.95	-42.075	0	0	0	70.229
	11 CASE7	7	163.983	-49.153	0	0	0	-83.016
		11	-163.983	49.153	0	0	0	-81.648

11	CASE1	8	303.851	30.057	0	0	0	52.825
		12	-303.851	-30.057	0	0	0	47.867
	6 CASE2	8	317.342	58.916	0	0	0	94.733
		12	-317.342	-58.916	0	0	0	102.636
	7 CASE3	8	168.474	-10.901	0	0	0	-10.489
		12	-168.474	10.901	0	0	0	-26.031
	8 CASE4	8	327.311	66.566	0	0	0	105.932
		12	-327.311	-66.566	0	0	0	117.063
	9 CASE5	8	141.227	-20.706	0	0	0	-25.595
		12	-141.227	20.706	0	0	0	-43.771
	10 CASE6	8	233.517	57.374	0	0	0	89.795
		12	-233.517	-57.374	0	0	0	102.409
	11 CASE7	8	47.433	-29.897	0	0	0	-41.732
		12	-47.433	29.897	0	0	0	-58.425
12	5 CASE1	9	1.96	119.648	0	0	0	94.214
		10	-1.96	105.292	0	0	0	-61.196
	6 CASE2	9	20.754	60.406	0	0	0	-25.21
		10	-20.754	119.546	0	0	0	-110.811
	7 CASE3	9	6.594	130.889	0	0	0	175.49
		10	-6.594	49.063	0	0	0	12.708
	8 CASE4	9	25.563	47.134	0	0	0	-53.681
		10	-25.563	124.4	0	0	0	-

								124.029
	9 CASE5	9	7.862	135.238	0	0	0	197.195
		10	-7.862	36.296	0	0	0	30.37
	10 CASE6	9	24.931	10.625	0	0	0	-82.499
		10	-24.931	92.296	0	0	0	-105.345
	11 CASE7	9	7.23	98.728	0	0	0	168.376
		10	-7.23	4.193	0	0	0	49.054
13	- CASE1	10	1.123	43.384	0	0	0	36.485
		11	-1.123	43.384	0	0	0	-36.485
	6 CASE2	10	13.014	-6.882	0	0	0	-18.625
		11	-13.014	76.296	0	0	0	-77.029
	7 CASE3	10	13.014	76.296	0	0	0	77.029
		11	-13.014	-6.882	0	0	0	18.625
	8 CASE4	10	16.055	-15.502	0	0	0	-31.197
		11	-16.055	88.47	0	0	0	-88.37
	9 CASE5	10	16.055	88.47	0	0	0	88.37
		11	-16.055	-15.502	0	0	0	31.197
	10 CASE6	10	15.691	-30.096	0	0	0	-42.625
		11	-15.691	73.876	0	0	0	-76.943
	11 CASE7	10	15.691	73.876	0	0	0	76.943
		11	-15.691	-30.096	0	0	0	42.625

14	5 CASE1	11	1.96	105.292	0	0	0	61.196
		12	-1.96	119.648	0	0	0	-94.214
	6 CASE2	11	6.594	49.063	0	0	0	-12.708
		12	-6.594	130.889	0	0	0	-175.49
	7 CASE3	11	20.754	119.546	0	0	0	110.811
		12	-20.754	60.406	0	0	0	25.21
	8 CASE4	11	7.862	36.296	0	0	0	-30.37
		12	-7.862	135.238	0	0	0	-197.195
	9 CASE5	11	25.563	124.4	0	0	0	124.029
		12	-25.563	47.134	0	0	0	53.681
	10 CASE6	11	7.23	4.193	0	0	0	-49.054
		12	-7.23	98.728	0	0	0	-168.376
	11 CASE7	11	24.931	92.296	0	0	0	105.345
		12	-24.931	10.625	0	0	0	82.499
15	5 CASE1	9	184.203	-28.097	0	0	0	-46.347
		13	-184.203	28.097	0	0	0	-47.779
	6 CASE2	9	108.068	7.655	0	0	0	-0.821
		13	-108.068	-7.655	0	0	0	26.466
	7 CASE3	9	186.453	-52.323	0	0	0	-72.854
		13	-186.453	52.323	0	0	0	-102.426
	8 CASE4	9	94.092	16.269	0	0	0	9.91
		13	-94.092	-16.269	0	0	0	44.591
	9 CASE5	9	192.074	-58.703	0	0	0	-80.132
		13	-192.074	58.703	0	0	0	-116.525

16	5 CASE1	10	218.914	7.108	0	0	0	11.885
		14	- 218.914	-7.108	0	0	0	11.927
	6 CASE2	10	173.194	35.126	0	0	0	58.392
		14	- 173.194	-35.126	0	0	0	59.28
	7 CASE3	10	177.273	-23.696	0	0	0	-39.28
		14	- 177.273	23.696	0	0	0	-40.101
	8 CASE4	10	171.252	41.989	0	0	0	69.799
		14	- 171.252	-41.989	0	0	0	70.864
	9 CASE5	10	176.351	-31.538	0	0	0	-52.291
		14	- 176.351	31.538	0	0	0	-53.363
	10 CASE6	10	101.782	39.913	0	0	0	66.322
		14	- 101.782	-39.913	0	0	0	67.388

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17	5 CASE1	11	218.914	-7.108	0	0	0	-11.885
		15	- 218.914	7.108	0	0	0	-11.927
	6 CASE2	11	177.273	23.696	0	0	0	39.28
		15	- 177.273	-23.696	0	0	0	40.101
	7 CASE3	11	173.194	-35.126	0	0	0	-58.392
		15	- 173.194	35.126	0	0	0	-59.28
	8 CASE4	11	176.351	31.538	0	0	0	52.291
		15	- 176.351	-31.538	0	0	0	53.363
	9 CASE5	11	171.252	-41.989	0	0	0	-69.799
		15	- 171.252	41.989	0	0	0	-70.864
	10 CASE6	11	106.881	33.614	0	0	0	55.768
		15	- 106.881	-33.614	0	0	0	56.839