# SEISMIC EVALUATION OF REINFORCED CONCRETE BUILDING WITH SOFT STOREY

**A DISSERTATION**

***Submitted in partial fulfilment of the requirements for the award of the degree***

***of***

MASTER OF TECHNOLOGY

***in***

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

***By***

**V.V.S. SURYA KUMAR DADI**



DEPARTMENT OF EARTHQUAKE ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE-247 667 (INDIA)

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## CANDIDATE'S DECLARATION

I here by certify that the work which is being presented in the dissertation entitled "**SEISMIC EVALUATION OF REINFORCED CONCRETE BUILDING WITH SOFT STOREY**" in the partial fulfillment of the requirement for the award of the degree of **MASTER OF TECHNOLOGY** in **EARTHQUAKE ENGINEERING** with specialization in **STRUCTURAL DYNAMICS**, submitted to the Department of Earthquake Engineering, I I T Roorkee, is an authentic record of my own work carried out from July 2003 to June 2004 under the supervision of Dr. S K Thakkar, Professor and Dr. Pankaj Agarwal, Lecturer, Department of Earthquake Engineering, I I T Roorkee, India.

The matter embodied in this dissertation has not been submitted by me for the award of any other degree or diploma.

Date: 30/6/04
Place: Roorkee

**(V. V. S. SURYA KUMAR DADI)**

## CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

S. K. Thakkar

**Dr. S. K. Thakkar**
Professor,
Department of Earthquake Engineering,
Indian Institute of Technology, Roorkee.

Pankaj Agarwal 30/6/04

**Dr. Pankaj Agarwal**
Lecturer,
Department of Earthquake Engineering,
Indian Institute of Technology, Roorkee.

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Place: Roorkee

Date: 25th June, 2004

**(V V S SURYA KUMAR DADI)**

# ABSTRACT

A G+10 RC building frame with one and two bay has been taken for the study of soft storey problem, analyzed and designed using the five different approaches, viz., (i) *IS 1893(Part 1):2002* Code criteria, (ii) Capacity Based Design, (iii) Energy Based Design, (iv) Equal Displacement Based Design, (v) Direct Displacement Based Design.

Analysis has been performed without considering the stiffness of infills in first method. Capacity based design has been carried out using the results obtained from the previous analysis of the frame for further revision of capacity. In energy based design, multiplication factors has been determined by assuming that sections of all columns are kept same in all floors for uniform structure as those from the initial analysis and infills are added for analysis of rigid structures. Equal and direct displacement based designs has been carried out with an assumed spectral velocity. The design of members has been carried out using SP-16.

The design carried out by IS 1893(Part 1):2002 design criteria yielded that the soft storey can resist up to 76% and 69% of the base shear of the maximum considered earthquake with out any reduction factor (i.e., R=1) for one and two bay frames respectively. The observations from the results of capacity based design method are (i) the moment magnification factors are (a) nearly equal to 1.0 at the joints of top and bottom storeys, (b) maximum at the joints of intermediate storeys, (c) nearly equal at all the exterior joints of 2-bay and 1-bay frames, and (d) higher near the interior joints compared to exterior joints of 2-bay frame. In energy based design, (i) the multiplying factors for the design of soft storey elements of 1-bay and 2-bay framed buildings are 1.825 and 1.928 respectively. (ii) soft storey can resist 55 % and 46% of the base shear of maximum considered earthquake with reduction factor of R=1 for 1–bay and 2-bay frames respectively. From equal displacement based design, the design base shears are found to be 52% of the design base shear of the design base earthquake with R = 5 for both one and two bay frames. Direct displacement based design, the design base shears are found to be 79% and 84% of the design base shear of the design base earthquake with R = 5 for one and two bay frames respectively.

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# Chapter 1

# INTRODUCTION

## 1.1 GENERAL

As per *IS 1893 (Part1):2002*: 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 storey above.

A soft storey is the result of either due to open ground storey for the purpose of car parking, reception lobbies, or commercial uses by increasing the height of ground storey or an abrupt change of stiffness with no masonry infill walls in the ground storey. Due to such irregular configuration the strength and stiffness of first storey will be less than the upper stories.

The absence of the infill in the ground storey gives the building a configuration of an inverted pendulum, with a large mass at the top of relatively flexible columns. So during earthquake the ground storey is subjected to the enormous storey shear, and deflection tends to concentrate in the first storey. Plastic hinges will form at the ends of the columns and this will transform the soft storey into a mechanism. Column hinges will be subjected to large plastic hinge rotation, and member ductility demand in column. Each hinge must dissipate huge amount of energy, due to this the strength of column degrades and the column will be unable to support gravity loads, P- $\Delta$ also increase the instability. Ultimately the whole building will collapse.

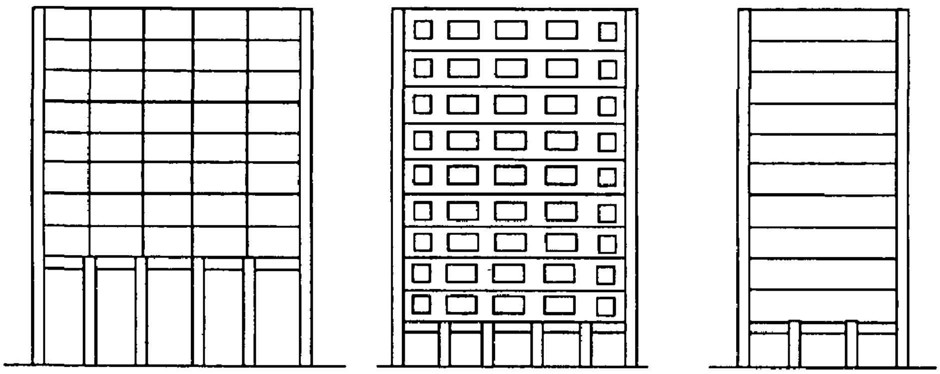
### 1.1.1 Conditions for Soft Storey

*Farzad Naeim (1984)* stated that, the soft storey problem results from the following conditions

1. A first floor structure, which is significantly higher than upper floors, resulting in less stiffness and more deflection in the first floor relative to the upper floors as shown in Fig. 1.1(a). The condition becomes worse as the relative height of the first floor increases, the number of floors above the first increases, and the stiffness of the upper floor increases.
2. An abrupt change of stiffness of second floor (though the floor heights remain approximately equal) as shown in Fig. 1.1(b). This is caused primarily by the material choice, the use , for example, of heavy pre-cast concrete or masonry elements, above

an open frame structure. This condition may often occur as a result of remodeling of older commercial buildings, in order to introduce storefront or hotel lobbies.

3. The use of a discontinuous shear wall, in which shear forces are resisted by walls, do not continue to the foundations but stop at the second floor level as shown in Fig. 1.1(c).
4. The discontinuous load paths created by a change of vertical and horizontal structure at the second floor to provide a more open first floor. This is often done for programmatic reasons, in order to reduce the number of columns at first (or basement) floors to permit large spaces for car parking.



**Flexible first floor** (a) | **Change of stiffness above first floor** (b) | **Discontinuous shear wall** (c)

**Fig. 1.1 Vertical irregularities in buildings**

## 1.2 OBJECTIVES

- To review the literature, covering the various soft storey design procedures and the possible solutions of the soft storey sway mechanism according to the past studies.
- To compare the design of a G+10 storey building with soft stories by different approaches as
  1. *IS 1893 (Part 1):2002* : Criteria for earthquake resistant design of structures
  2. Capacity Based Design of Structures
  3. Energy Based Design
  4. Displacement Based Design

## 1.3 SCOPE OF THE WORK

It is known that the design approach for soft storey problem is not yet complete from all perspectives for an effective design output. But the various methods that are proposed to be applicable, to solve this problem has drawbacks in some aspects of design or in detailing. The member ductility demand requirement for an open storey column members are very high, it is not possible to satisfy the total ductility requirement from the design point of view. It is still confusing for a structural engineer to opt efficient procedure for solving the soft storey problem; various approaches are available to solve the soft storey problem in literature. The codal procedures that are specified for a soft storey problem is also incomplete as there is not even a single procedure that is specified except the multiplication factor that is given for the resisting element's design forces. In present thesis for analytical study, an eleven storeyed one bay RC plane frame (G+10) building with open ground storey.

Applying *IS 1893 (Part 1):2002*, the frame has been analyzed with out considering infill stiffness and designed considering the gravity and earthquake loading for the various load combinations. Similarly it is also analyzed incorporating the infill stiffness by modeling the infills with plate elements and designed. The design criteria according to *IS 1893 (Part 1):2002* has been applied to the soft storey elements such as multiplying the design forces by 2.5.

Applying the capacity based design for a soft storey problem, the same structure is taken and analyzed for gravity and earthquake forces and preliminary design is done for the load combinations given in IS *1893 (Part 1):2002*, analysis and design process is repeated until the achievement of column interaction ration around 0.9-1 for all columns, by revising the column sections in every iteration. The actual capacities of the beams and columns for the actually provided steel is calculated, the capacity design rule is applied considering the actual capacities. Then the magnification factors for the column moments were obtained. The column capacities were revised using the magnification factor, and columns are designed for the new moment capacity requirement keeping the interaction ration 0.9-1.This design procedure provides weak beams and strong columns.

Applying the Energy Based Design approach to obtain the multiplication factor for the design forces of the resisting elements of the soft storey, the same structure is taken and analyzed without the consideration of infill stiffness, here the earthquake forces are taken from the equivalent static load method using the empirical formula for time period given is *IS 1893 (Part 1):2002*. Similarly the same structure is analyzed with consideration of infill stiffness. Multiplying factor is obtained from the ratio of plastic energies of the both structures i.e. uniform structure and rigid structure for the same earthquake. Then the elements of the soft storey were designed for the revised forces by the multiplication factor.

To apply the Displacement Based Design Procedure, an eleven storeyed one bay RC plane frame (G+10) building with open ground storey with height of ground storey more than upper storey is considered. With an assumed spectral velocity the stiffness of the structure is obtained and then the forces coming on to the open ground storey (soft storey) were obtained. Elements of the soft storey were designed accordingly.

## 1.4 ORGANIZATION OF THE THESIS

This thesis organized into six chapters; Chapter-1 introduces the work undertaken for research. Chapter-2 presents the literature review on analytical investigations carried out by the past researchers on RC building with soft storey, mainly, (i) *IS 1893 (Part 1):2002* code criteria for soft storey design. (ii) Capacity Based Design concept. (iii) Energy Based Design concept. (iv) Displacement Based Design Concept. Chapter-3 provides the preliminary data required for the dynamic analysis and design of the two frames (i.e., one bay and two bays of G+10 building with soft storey). Chapter-4 presents the analysis and design results of the (G+10) storeyed plane frame with open ground/soft storey by various proposed methods. Chapter -5 presents the discussions and comparisons of the results obtained from the four methods. Chapter -6 provides the summary and conclusions of the present study.

# Chapter 2

# LITERATURE REVIEW

## 2.1 INTRODUCTION

Open ground storey is a typical feature in the modern multistory constructions in India and other countries. Functional use and aesthetic requirements of the structure compel the designer to go for an open ground storey or open storey with out the provision of infill walls in the moment resisting framed structures, and are safe in the non seismic areas where designed and subjected to gravity loads. But such features are highly undesirable in buildings situated in seismically active areas. The various earthquakes have illustrated the potential hazards associated with buildings having open ground storey. This review of literature includes review of research (analytical and experimental) conducting for knowing the failure behavior of buildings with soft storey and possible or probable design solutions to over come soft storey collapse.

## 2.2 REVIEW OF RESEARCH

*Fintel, M. Khan (1969)* suggested that soft storey concept for multistory structures can be used as shock-absorbing system. The concept is based on controlling the lateral forces (acceleration) that will occur in the structure during an earthquake. This is achieved by designing a shock absorbing soft storey with a bilinear force displacement characteristic, which forces all the inelastic deformation (or undesirable effect) due to high intensity earthquake motions to this soft storey only. Above the soft storey the structure will be designed to wind only and will remain with in the elastic range during an earthquake. A part of the overturning moments caused by the earthquake distortions will be resisted by the stability walls in the soft storey. Elastomeric layers (e.g. neoprene) are used to accommodate the large distortion and separate the wall from the slab above. The force deformation characteristics of the ground storey can be controlled most conveniently by designing the columns to yield at their ends once a predetermined force level is reached. To insure hinging at the column at the column ends rather than in the connecting beams, the section at these points may be reduced by peripheral notches. The total overturning moment $M = \sum P\Delta + Vh$, is resisted by both the columns and stability walls. The stability walls resist the overturning moments caused by horizontal forces transmitted by Elastomeric link. Thus the system minimizes the oscillations during an earthquake by increasing flexibility and time period, so the response of the structure decreases thus reducing damages while providing sufficient strength and rigidity for wind loading. And the structure has to be designed for reduced earthquake forces

*Chopra. A K (1973)* carried out studies related to dynamic, bi-linear response behavior of a series of eight storey shear buildings subjected to simulated earthquake excitation. The specific objective of the investigation is to determine under what conditions a yielding first storey can adequately protect the upper storey from significant yielding. Two classes of buildings are considered: Stiff (5 sec period) and flexible (2.0 sec period), and the basic parameters considered in the yielding first storeys are the yield force level and the bi-linear stiffness. The results demonstrate that a very low yield force level and an essentially perfectly plastic yielding mechanism are required in the first storey to provide effective protection to the superstructure. Moreover, the required displacement capacity of such an effective first storey mechanism is found to be very large.

The above two studies show that due to presence of open first storey (soft storey), the time period of the building lengthens so reduces the force level. But many damage studies show that soft storey is highly vulnerable during earthquake shaking. So many emphasis given for protection of soft storey failure. And various researches have been done for possible or probable solution following by experimental and analytical studies.

*Cassis., J. H et al, (1996)* studied the nonlinear response of reinforced concrete buildings with irregularities in elevation. Two cases of irregularities have been considered (1) Walls interrupted in height and (2) Buildings having a soft first storey. The buildings were first designed using the Chilean code NCh 433, of 72 for earthquake resistant buildings, and applying a nonlinear-elastic dynamic analysis by spectral mode superposition. The strength and displacement capacity of those buildings were obtained by a nonlinear program Drain-2DX. A 5% damping ratio with respect to the critical was used for calculation of the nonlinear response in the dynamic analysis case. They have observed that in building (1) the distribution of the shear and overturning moment in the vertical substructures present great variations in the zone of irregularities. The earthquake response of this irregular building was good because some walls spanned all the height. Thus inter storey drift may be controlled and so the damage, on the other hand, ductility demands kept with in an acceptable range values. Building (2) large values of inter storey drift are concentrated between the soft first storey and second storey. In this kind of building it should be remarked that there is not a desirable collapse mechanism. In effect, it shows brittle failure mode characteristics by the failure of the lateral reinforcement in hoped columns under compression. Due to the high level of axial load, longitudinal steel may also fail in tension. If increasing the lateral or longitudinal steel prevents these failure modes, a flexural ductile mode failure can be reached but it would demand large deformation, impossible to be accepted in real structures. The foregoing features of failure modes in soft storey buildings lead to a rejection of them unless adequate change in the structural pattern may be done, as is the case when some structural walls are extended to the first storey, thus providing enough stiffness and strength to that soft and weak storey.

*Chen.D.G. et al, (1996)* studied the rational stiffness ratio of the second to the first storey, to avoid deformation concentration for the 8-storey composite masonry wall buildings supported by frame shear structure at the first storey. A 1:4 scale 8-storey model composite wall building had been tested on earthquake simulating shaking table to understand the seismic performance of such buildings. Dynamic analysis and reliability of a typical 8-storey composite masonry building supported by frame shear structure at the first storey are conducted. They have concluded that the buildings supporting on frame-shear structure at the first storey with composite masonry wall well behaved seismically, if the stiffness ratio $k_2/k_1$ for the intensities 7 and 8 respectively are of 1.6 to 1.2 are proposed for design purpose.. The percentage of vertical load sustained by the supporting frame beam has also been studied by experimental work and nonlinear FEM. It is shown by the test and analysis that due to the arch action and confined columns of composite wall, only about 30% of the vertical load is transmitted to the supporting beam. A 60% of the total vertical load from the upper part of the building uniformly distributed on the supporting beam, is proposed to design to the frame beam.

*Arlekar, J.N et al, (1997)* conducted an analytical study by taking an example of the reinforced concrete moment resisting frame building, with open first storey and unreinforced brick infill walls in the upper storey. The building is kept symmetric in both orthogonal directions in plan to avoid torsional response under pure lateral force and the building founded on medium strength soil through isolated footing under the column. The linear elastic analysis is performed using ETABS analysis package. After parametric study he concluded that the drift and strength demands in the first storey columns are very large for buildings with soft ground storeys. He has suggested possible solution on the basis of his study that, provide stiffer columns in the first storey or to provide concrete service core in the building. The soil flexibility needs to be examined carefully before finalizing the analytical model.

*Fardis. M.N. et al, (1999)* proposed to modify the capacity design rule at beam-column joints, in such a way that the capacity of the column of the moment infilled storey is added to that of the beams rather than to the capacity of the column of the less- infilled (or open)storey. A three- storey two-way infilled RC test frame was designed on the basis of this concept, for pseudo dynamic testing at the ELSA reaction wall facility in ISpra(1), in two configurations, each one with absence of infills from a different storey. Pre-test nonlinear dynamic analysis showing satisfactory response of the so-designed frame in comparison to *EC8 (2002)* rule version, are verified by the test results. Parametric inelastic analyses for different infill-frame relative strengths verify the proposed rule.

*Yong Lu (2002)* presented a comparative study on nonlinear behavior of reinforced concrete (RC ) multistory structures on the basis of measured response of four six storey, three bay frame structures, namely a regular bare frame, a discontinuous-column frame, a partially masonry infilled frame and a wall- frame system. The structure was designed for similar seismic requirements in accordance with the *EC8 (2002)* and their 1:5.5 scaled models were subjected to similar earthquake Simulation tests. The models were mounted on the earthquake simulator and provide unidirectional excitation. Experimental observation and numerical analysis show that in the infilled stories, an increase of the storey shear strength and stiffness by 60% was attributable to the masonry walls up to an inter storey drift of 3%. The prevention of a soft storey mechanism to occur in the open first storey of the frame requires a smoothened over strength distribution taking into account the infill walls in the adjacent stories, and for this purpose, more accurate estimation of the resistance of the infilled stories under cyclic loading is necessary.

## 2.3 POSSIBLE DESIGN METHODOLOGIES FOR SOFT STOREY DESIGN

The problems arising from soft storey sway mechanism have no proper solutions till date, but possible solutions according to past studies have been mentioned below.

### 2.3.1 IS 1893 (Part 1):2002-Criteria for Earthquake Resistant Design of Structures

*Clasuse 7.10* of *IS 1893 (Part 1):2002* provided the guidelines for earthquake resistant design of reinforced concrete buildings with soft storey and the same is reproduced here.

I. In case of buildings with a flexible storey, such as the ground storey consisting of open spaces for parking that is Stilt buildings, special arrangement needs to be made to increase the lateral strength and stiffness of the soft/open storey.

II. Dynamic analysis of building is carried out including the strength and stiffness effects of infill walls and inelastic deformations in the members, particularly those in the soft storey, and the members designed accordingly.

III. 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 column and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads specified in the other relevant clauses; or,

   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 force calculated as before.

### 2.3.2 Capacity Based Design of Soft Storey

Prediction with accuracy the characteristics of the ground motion due to a large earthquake is impossible, so it is also not possible to estimate with accuracy the response of the R.C structure to this earthquake. These two weak points in the calculation of the action effects render necessary a more reliable approach to the problem, which would ensure the existence of adequate strength and ductility in the structure. However, it is possible to provide the structure with the features that will ensure the most desirable behavior. In terms of ductility, energy dissipation, damage or failure, this means that the sequence in the breakdown of the chain of resistance of the structure will follow a desirable hierarchy. In order to ensure a certain sequence in the failure mechanism of the resistance chain, the resistance of every link should be known. This knowledge should not be based on assumptions of disputable reliability, but on the calculated strength of structural elements which will be subjected to very large deformations (due to formation of plastic hinges) during a catastrophic earthquake.

Although the nature of the design actions is probabilistic, the ability to have a deterministic allocation of strength and ductility in the structural elements provides an effective tool for ensuring a successful response and prevention of collapse during a catastrophic earthquake. Such a response can be achieved if the successive regions of energy dissipation are rationally chosen and secured through a proper design procedure, so that the predecided energy dissipation mechanism would hold throughout the seismic action. This design concept can be included in a procedure which is called the Capacity Design Procedure.

According to the capacity based design procedure, the structural elements which are designated to dissipate the seismic energy are reinforced accordingly, while other members with adequate reserve strength are provided so that it is ensured that the chosen dissipating mechanism is preserved during the seismic cyclic deformation of the structure, without serious reduction of strength in the critical regions. This means that the action effects which have resulted from the analysis serve only as a guide and they are properly modified in order to accommodate the capacity design of the structure. Of course this modification is made in a way that the cost increase is kept with in acceptable limits. It is evident that this modification should also be a function of the selected design ductility class as will be explained below.

#### 2.3.2.1 Design criteria influencing the design action effects

The local resistance criteria and the capacity design criteria nfluence the determination of the design action effects. All the others refer to dimensioning and detailing of the R.C structural elements.

The design criteria influencing the design action effects are, in detail, the following:

1. All critical regions of the structure must exhibit resistance adequately higher than the action effects produced in these regions under the seismic design situation.
2. Brittle or other undesirable failure modes, i.e. (a) shear failure of the structural elements, (b) failure of beam-column joints, (c) yielding of foundations, or yielding of any other element intended to remain elastic) must be excluded. This can be ensured if the design action effects of purposely selected regions are

derived from equilibrium conditions when flexural plastic hinges with their possible over strengths have occurred in adjacent areas.

3. Extensive distribution of plastic hinges, avoiding their concentration in any single storey ('soft storey' mechanism) is ensured if the formation of plastic hinges at both ends of at least some columns on the same storey is prevented. This can be achieved if- with sufficient reliability –it is ensured that the plastic hinges develop only in beams and not in columns, except for the unavoidable formation of plastic hinges at the base of the building as shown in Fig. 2.1.

The implementation of these criteria for the determination of the design action effects of the various structural elements of a structure is given below.

### 2.3.2.2 Capacity design procedure for beams

The design values of the bending moments of beams for all ductility classes are obtained from the analysis of the structure for the seismic loading combinations. However, according to all relevant codes beams need an additional reinforcement at their support, compression reinforcement equal to 50% of the corresponding tension reinforcement, in order to ensure an adequate ductility level. Based on the capacity design concept these reinforcement bars are appropriately anchored in concrete, So that they can operate as tension reinforcement in case of moment reversal. Therefore, the moment resistance envelope of the beam is considerably improved at low cost (the cost of anchorages of the compression reinforcement) no matter what the values of the design action effects which have been derived from the analysis. This means that the beam, as it is designed, can carry much larger moment fluctuations generated by an earthquake than the design action moments. However, in order to ensure this behavior, the structural elements has to be secured against premature shear failure, because, it is well known shear failure does not present ductile mode. Therefore , the design shear, at least for DC 'H'(Ductility Class High) should not be that resulting from the analysis but the shear corresponding to the equilibrium of the beam under the appropriate gravity load and a rational adverse combination of the actual bending resistances of the cross-sections.

$$V_{A,S1} = \frac{wl}{2} + \gamma_{Rd} \frac{M_{AR} + M'_{BR}}{l}$$

$$V_{A,S2} = \frac{wl}{2} - \gamma_{Rd} \frac{M_{BR} + M'_{AR}}{l}$$

$$V_{B,S1} = -\frac{wl}{2} + \gamma_{Rd} \frac{M_{AR} + M'_{BR}}{l}$$

$$V_{A,S1} = -\frac{wl}{2} - \gamma_{Rd}\frac{M_{BR} + M'_{AR}}{l}$$

Where $M_{AR}$, $M'_{AR}$, $M_{BR}$, $M'_{BR}$ are the actual resisting moments at the hinges accounting for the actual area of the reinforcing steel(all positive) and $\gamma_{Rd}$ the amplification factor taking into account the reduced probability that all end cross sections exhibit simultaneously the same over strength. This $\gamma_{Rd}$-factor also counter balances the partial safety factor $\gamma_s$ of steel chosen for the fundamental load combination and covers the hardening effects as well. In the absence of more reliable data, $\gamma_{Rd}$ may be taken as

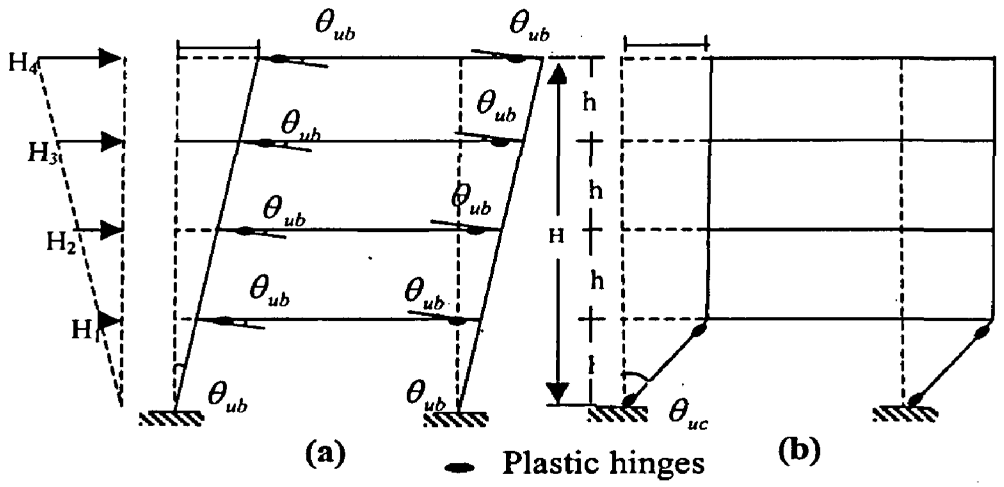
$\gamma_{Rd} = 1.25$

#### 2.3.2.3 Capacity design procedure for columns

##### 2.3.2.3.1 Bending

It has been already stressed that the formation of plastic hinges in the columns during an earthquake should be avoided, in order to make sure that the seismic energy is dissipated by the beams only. The reasons for this requirement are the following:

1) Due to axial compression, columns have less available ductility than beams. On the other hand, for the same displacement of the frame, that is for the same ductility expressed in terms of displacements, much large plastic column rotations are required than beam rotations. Therefore, for the same frame ductility, a larger column ductility expressed in rotation is required for the creation of a beam failure mechanism.




$$\theta_{ub,req} = \frac{\delta_u}{H} \qquad \theta_{uc,req} = \frac{\delta_u}{h} \; for\ same\delta,\ \theta_{uc,req} >> \theta_{ub,req}$$

**Fig. 2.1 Failure mechanism of a frame:**
**(a) beam mechanism (b) storey mechanism**

$\theta^{avail}{}_{UC} << \theta^{avail}{}_{Ub}$, for the same $\delta_u$ (i.e the same $\mu_{req}$)

$$\theta_{UC}{}^{required} = \frac{\delta_U}{h} >> \theta_{Ub}{}^{required} = \frac{\delta_U}{H}$$

2) While beam failure exhibits extended cracking only in the tension zones due to the yielding of the reinforcement, column failure mode successively presents spalling of concrete, breaking of the ties, crushing of the concrete core and buckling of the longitudinal reinforcement bars. This process leads to the creation of a collapse mechanism due to the inability of the columns to carry the axial gravity loads after their failure. Therefore, avoiding column failure is much more crucial for the overall safety of the structure than avoiding beam failure.

3) The formation of plastic hinges in the columns leads to significant inter-storey drifts, so that the relevant second-order effects may cause the collapse of the structure.

4) In order to decrease the probability of plastic hinge formation in the columns, frames must be designed to have '***strong columns and weak beams***'. This

concept is realized in the requirements of *EC8 (2002)* and other relevant codes stating that the sum of the resisting moments of the columns, taking into account the action of axial load, should be greater than the sum of the resisting moments of all adjacent beams for each (positive or negative) direction of the seismic action, as shown in Fig. 2.2 that is

5) $\left|M^{o}{}_{R1}\right| + \left|M^{u}{}_{R1}\right| \geq \gamma_{Rd} \left|M^{l}{}_{R1}\right| + \left|M^{r}{}_{R1}\right|$

6) $\left|M^{o}{}_{R2}\right| + \left|M^{u}{}_{R2}\right| \geq \gamma_{Rd} \left|M^{l}{}_{R2}\right| + \left|M^{r}{}_{R2}\right|$

7) Where $\gamma_{Rd}$ is a factor which takes into account the variability of the yield stress fy and the probability of strain hardening effects in the reinforcement (over strength factor).

8) Therefore, the capacity design is satisfied if the columns are designed for the following moments:

$M_{S1,CD} = \alpha_{CD,1} M_{S1}$,

$M_{S2,CD} = \alpha_{CD,2} M_{S2}$

Where $\alpha_{CD,1} = \gamma_{Rd} \dfrac{\left|M^{l}{}_{R1}\right| + \left|M^{r}{}_{R1}\right|}{\left|M^{o}{}_{S1}\right| + \left|M^{u}{}_{S1}\right|}$

$\alpha_{CD,2} = \gamma_{Rd} \dfrac{\left|M^{l}{}_{R2}\right| + \left|M^{r}{}_{R2}\right|}{\left|M^{o}{}_{S2}\right| + \left|M^{u}{}_{S2}\right|}$

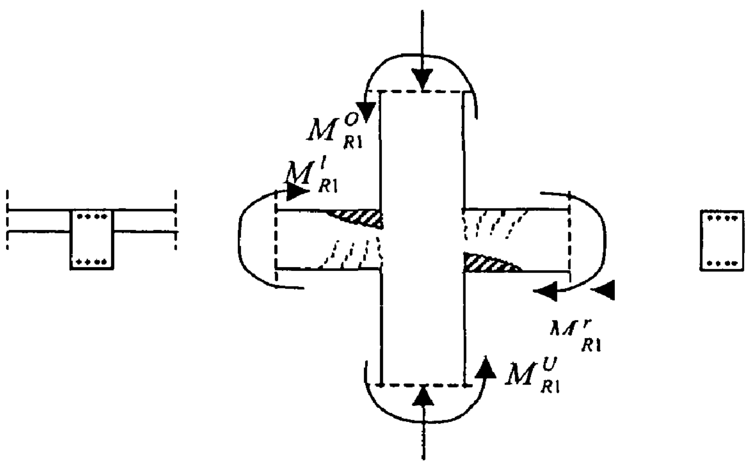



**Fig. 2.2(a) End moment capacity of beams at a joint in seismic action direction 1**

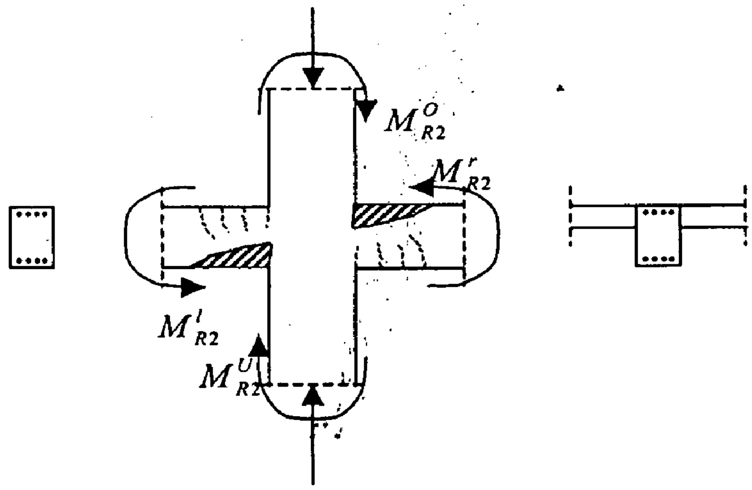



**Fig. 2.2(b) End moment capacity of beams at a joint in seismic action direction 2**

*EC8 (2002)* allows a relaxation of the above capacity design criterion whenever the probability of full reversal of beam end-moments is relatively low. The following cases are also exempted from the requirement of the above procedure:

- In single or two storey buildings and in the top storey of multistory buildings:
- In one-quarter of the column of each storey in plane frames with four or more columns

The design bending moments for DC 'H' are determined according to the above described capacity design criterion with $\gamma_{Rd} = 1.35$

For DC 'M' the design bending moments are determined according to the same procedure, with $\gamma_{Rd} = 1.20$

Finally, for DC 'L' the design bending moments are determined from the analysis of the structure for the seismic load combination without any application of the capacity design criterion. The magnification factor $\alpha_{CD}$ takes rather higher values.

#### 2.3.2.3.2 Shear

Shear forces according to the capacity design criterion and following the rationale developed for the beams are determined by considering the equilibrium of the column under the actual resisting moments at its ends, as follows, see Fig. 2.3

$$V_{Sd,CD} = \gamma_n \frac{M_{DRd} + M_{CRd}}{l_c}.$$

where $\gamma_n$ accounts for the lower probability of all failure modes accepted for the columns, even if their ends exhibit flexural plastification. Practically, $\gamma_n$ may take the values of $\gamma_{Rd}$ used in each case.

The design shear forces for DC 'H' are determined according to the capacity design criterion developed above with $\gamma_{Rd} = 1.35$

For DC 'M' shear forces are determined according to the same procedure with $\gamma_{Rd} = 1.20$

Finally, for DC 'L' the design action shear forces are determined by the analysis of the structure for the seismic load combination without any application of the capacity design consideration

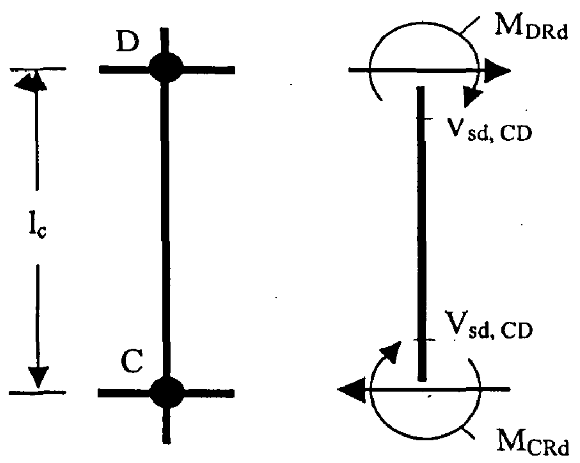



**Fig. 2.3 Capacity design values of shear forces acting on columns**

#### 2.3.2.3.3 Seismic behavior of columns

A fundamental principle of capacity design is that in R.C buildings plastic hinge formation in columns should be avoided. To achieve this, column design moments are derived from equilibrium conditions at beam column joints, taking into account the actual resisting moments of beams framing into the joint, However, there are a number of reasons why the capacity design included in EC8 (2002) cannot achieve this goal: these reasons are discussed in the following section.

Uncertainties regarding the capacity design of columns

1) Whenever the degree of inelasticity at the beam ends is high (typically this would be the case with DC 'H' beams), the longitudinal bars enter strain hardening range and this may cause an increase in beam strength between 10 and 25% depending on the steel characteristics and the ductility factor attained.
2) In calculating actual strengths of beams, reinforcing bars in slabs integrally built with the beams are either completely neglected or taken into account considering an effective slab width in tension that is clearly smaller than that observed in relevant tests. The corresponding increase in the actual beam strength may range from 10% to 30%.
3) The flexural strength of a column varies considerably with the axial load level. During a strong earthquake motion the axial load in a column is continuously changing due to the combined effect of overturning moments and vertical acceleration of the motion: this effect is more pronounced in columns at the perimeter of the building. The range of variation of axial load may be wider than that predicted by the analysis for the design actions, particularly when the vertical motion is significant. Therefore, at certain stages of the seismic response, the strength of a column may be substantially lower than that taken into account in the capacity design.
4) Analysis of the inelastic response of multi-storey R.C buildings subjected to earthquake excitation, have shown that the point of contraflexure in columns shifts considerably during the excitation. Leading to a distribution of bending moments substantially different from that resulting from the code-prescribed analysis (especially when the later is an equivalent static one). In addition to differences between static and dynamic response (influence of higher modes). The shift of the contraflexure point is caused by the formation of hinges in beams adjacent to columns and even by extensive cracking in parts of the column, as all these factors alter the stiffness of the beam-column subassemblage, hence the moment distribution. Therefore, ensuring that the sum of column moments at a joint exceeds the sum of the corresponding beam moments does not necessarily mean that the moment in each single column always remains lower than the corresponding flexural strength. It is not uncommon that in the course of seismic loading a plastic hinge forms in the column below a certain joint, while the

column above the joint remains in the elastic range, as it is subjected to significantly lower moments.

5) The direction of propagation of seismic waves does not in general coincide with a principal axis of the building (if indeed such an axis exists), and this combined with the effect of eccentricities in plan leads to a biaxial stress state in columns (particularly the corner ones). Checking the relative strength of beams and columns at a joint separately in each direction (allowed by most codes, including *EC8 (2002)*), does not necessarily ensure that a column has adequate capacity to resist an arbitrary biaxial loading history, especially when all beams framing into the joint (in two or more directions) form a plastic hinge

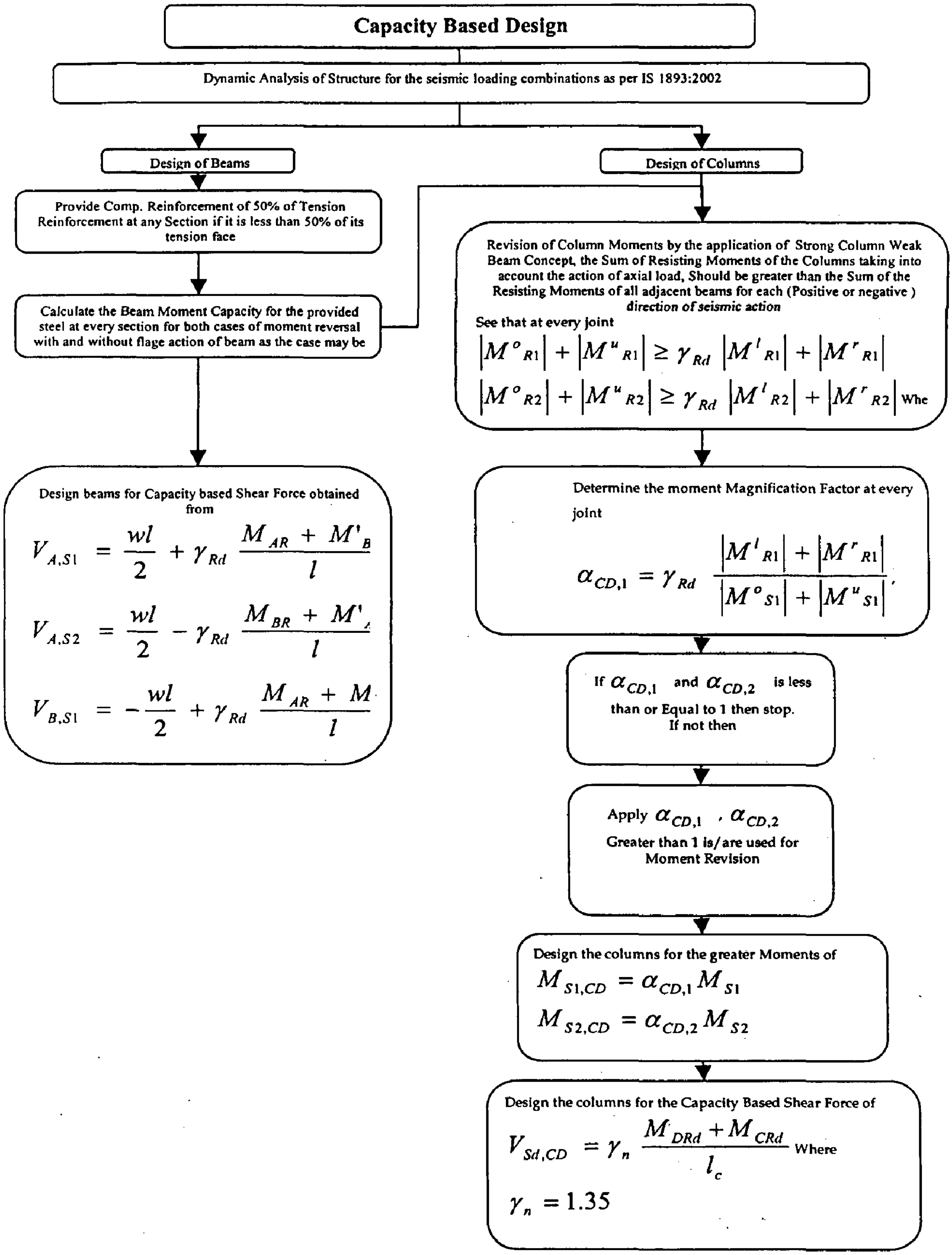



**Fig. 2.4 Step by step procedure in capacity based design**

### 2.3.3 A Simplified Energy Approach *[Adrian S Scarlat (1997)]*

Generally, the vertical irregularities lead to important local stress concentrations accompanied by large plastic deformations, but the soft stories display an additional and more dangerous feature: most of the energy developed during the seismic attack is dissipated by the vertical resisting elements of the soft story (usually columns). Plastic hinges set on and they may transform the soft story into a mechanism, in which case collapse is unavoidable. Moreover, the drift of the soft story is usually very large, thus entailing a significant p - $\delta$ *effect*, i.e. an additional threat to the stability of the structure. The soft stories represent a very dangerous type of vertical irregularity and deserve special consideration.

*Adrain S. Scarlat (1997)* proposed an energy approach in which equivalent static forces are to be taken into account in the design of resisting elements of soft stories. It is based on interpolation between two extreme situations: Uniform structures and rigid structures supported by a soft story. The solution is based on quantifying the plastic energy absorbed during the seismic motion, but its effective computations are to be performed in the elastic range only.

It is presumed that the provision for soft stories should be formulated by taking into account the following criteria;

a) Performing the structural analysis of buildings with soft stories by the usual, static lateral procedure, and multiplying the seismic stresses in the resisting elements of the soft story and the adjacent stories by a given factor c.
b) Determining the multiplying factor "c" by evaluating, even approximately, the plastic energy dissipated by the resisting elements of the soft stories during the earthquake.

According to *Housner*'s proposal, to evaluate the maximum total energy ($E_t$) absorbed by a structure in the form:

$$E_t = {MS^2}_{v,\xi}/2$$

Where M denotes the total mass of the building and $S_{v,\xi}$,the velocity spectrum for a damping ratio $\xi$ spectral velocity with a damping ratio $\xi$

*Housner* assumed that the total energy is constant for a given type of earthquake and a given damping ratio, basing this assumption on the shape of the elastic velocity spectra determined for several earthquakes, in the usual range of rigidities (fundamental periods).

In the case of inelastic velocity spectra we can admit that this assumption remains essentially valid.

The total energy $E_t$, absorbed by the structural elements is made up by two components: elastic energy ($E_e$) and plastic energy ($E_p$):

$$E_t = E_e + E_p$$

According to *Housner*, *the energy input is the same when parts of the structure are stressed beyond the elastic limit as it would be if the structure behaved elastically*, assuming that *the inelastic deformations do not have a major effect on the stiffness characteristics of the structure*

It is emphasized that the results of the elasto-plastic time history analysis performed by *Clough (1970)* on multistory frames and by *Derecho et al. (1978)* on reinforced concrete shear walls have confirmed this basic assumption.

*Therefore two structures **A** and **B**, having the same mass and subjected to identical seismic forces are considered, multiply the forces acting upon the structure **B** by a constant (the 'multiplying factor") so that the elastic energies of both the structures will be identical. Since the total energies of both structures are the same, it is concluded (according to Housner's assumption) that the plastic energies of both the structures will be also identical.*

Hence a procedure for quantifying the soft effect, based on the computation of the plastic energy according to *Housner's* assumption is proposed for two *extreme models*, and to compute accordingly the multiplication factor $c_o$. The final multiplying factor for the given structure is obtained by interpolation between the factors 1 (minimum) and $c_o$ (maximum). In most of the cases, the soft storey will be at the ground floor as shown in Fig. 2.5. The proposed formula can be applied also for any intermediate story (i.e., by computing the stiffness ratio K1/K2).

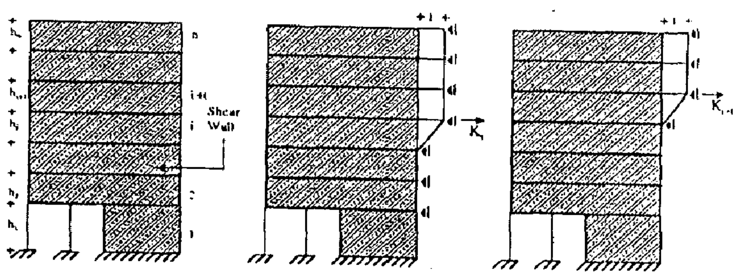



**Fig. 2.5 Definition of lateral stiffness $K_i$**

#### 2.3.3.1 COMPUTATION OF MULTIPLYING FACTOR

a) A given structure R with a soft story at ground floor, subjected to the lateral forces F.

b) An auxiliary uniform structure U as shown in Fig. 2.6, obtained from the structure R by considering that at each story only columns exist and the sum of the rigidities (I/H) of the columns at each story is equal to the sum of the rigidities of the columns at the ground floor of structure R. In order to simplify the computations, the beams are assumed as rigid. The structure U is subjected to the same lateral forces F as the real structure R. So, the structure U is perfectly uniform and "soft story effect" is nil. Therefore, the multiplying factor $c_0$ is equal to 1

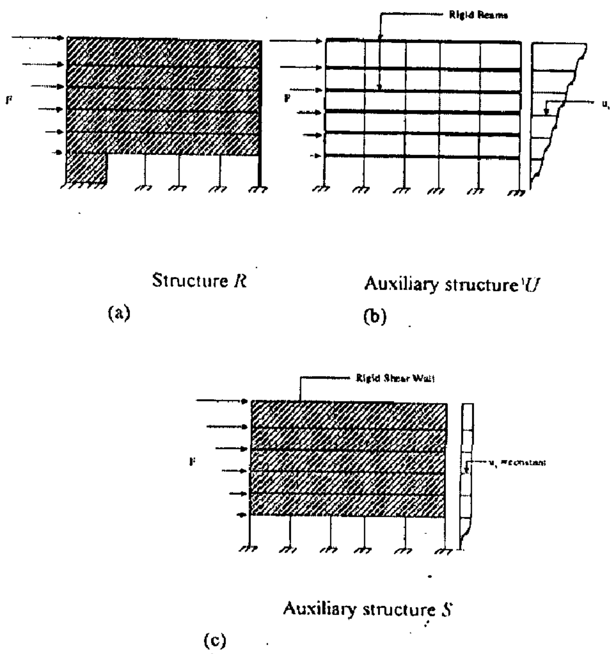



**Fig. 2.6 Computation of multiplying factor $c_0$ *[Adrian S Scarlat (1997)]***

c) An auxiliary structure S as shown in Fig. 2.6 where all the stories consist of rigid shear walls, except the ground floor, where only columns exist; we shall consider that the columns are identical to the columns of the structure U. The auxiliary structure S is subjected to the lateral forces F multiplied by the factor $c_o$. Since this structure represents an extreme case of soft story, the multiplying factor $c_o$ can be taken as maximum *($c_o$ = $c_{max}$)*. The auxiliary structure S is subjected to the lateral loads F multiplied by the factor $c_o$.

*The maximum factor $c_0$ by imposing the condition that the plastic energies of both auxiliary structures U and S be equal, According to the above-mentioned property, it can be imposed that the equality of elastic energies and this will also imply the equality of the plastic energies.*

The elastic energies of the structure U and S are: $E_{e,U} = \frac{1}{2}\sum_{u} Fu$; $\quad E_{e,S} = \frac{c_0}{2}\sum_{S} Fu$

The factor $c_0$ affects the forces F and the corresponding elastic deformations, too. Therefore the elastic energy of the structure S includes the factor $c_0{}^2$.

By equating $E_{e,U}$ and $E_{e,S}$ one can obtain the expression of the maximum multiplying factor $c_0 = \left[\frac{\sum_{u} Fu}{\sum_{S} Fu}\right]^{1/2}$. The multiplying factor $c$ of the given structure R lies between the minimum $c = 1$ and the maximum $c = c_0$. $c$ can be determined by interpolation, as a function of the stiffness ratio as shown in Fig. 2.7 $K_1/K_2$ i.e., $c = c_0 - (c_0 - 1) \times K_1/K_2$

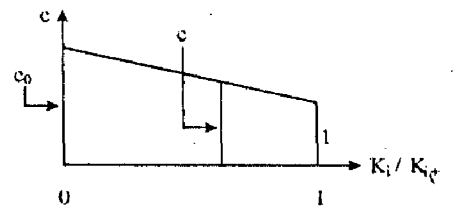



**Fig. 2.7 Computation of multiplying factor** *c [Adrian S Scarlat (1997)]*

### 2.3.4 Displacement Based Design Methods

Seismic design of reinforced concrete building was taken until now using the traditional approach i.e. strength based design. The design focus is the idealized elastic behavior region described in Fig. 2.8 ($F < F_{max}$).The strength based design approach has worked well, especially given the limited scientific basis, but prescriptions often restrict the accomplishment of functional and aesthetic design objectives unnecessarily. The codified design version of the strength-based process and objectives are described in Fig. 2.8. The area of design interest is confined to the presumably sub-yield behavior region $F \leq F_o$, The deformation likely to be experienced in the structure is understood to be in the region of $\Delta_u$ ..Displacement based design approaches start by identifying an objective system displacement ($\Delta_u$) and ductility ($\mu$). Then they proceed to establish the system strength and stiffness necessary to the safe attainment of these objectives.

Design Methodologies based on displacement have been proposed as a part of performance based design. This is logical since the performance in a ductile structure shown in Fig. 2.8 can only be evaluated based on estimates of deformation ($\Delta_u$) and ductility ($\mu$). They then proceed to establish the system strength and stiffness necessary to the safe attainment of these objectives.

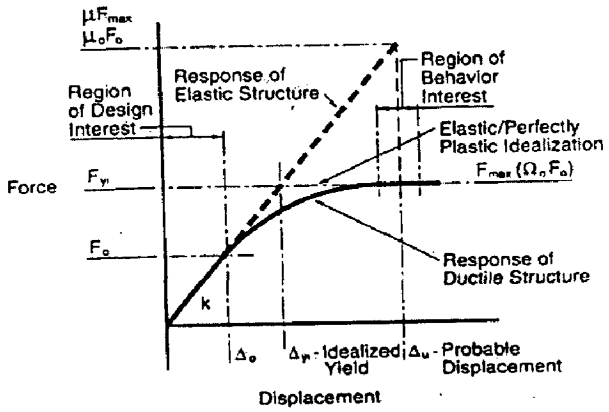



**Fig. 2.8 Force displacement relationship of ductile structure** *[Englekirk (2003)]*

The equal-displacement approach follows the Newmark-Hall proposition described in Fig. 2.9 the basic proposition is that the displacement response of a ductile structure can be developed from estimates of the response of an otherwise equivalent elastic structure.

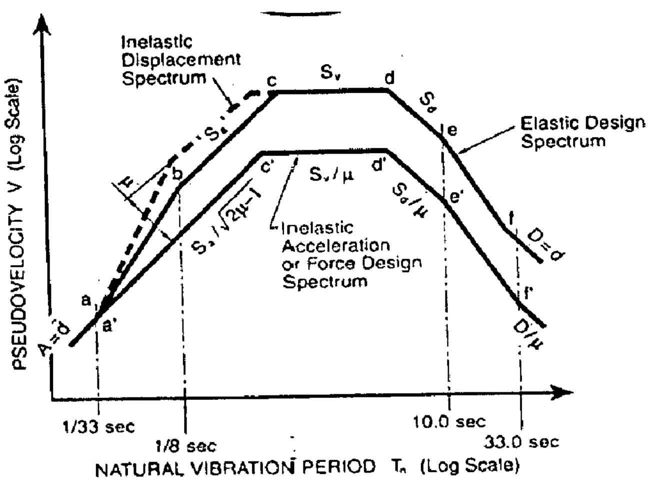



**Fig. 2.9 Construction of the design inelastic spectrum** *[Englekirk (2003)]*

**Assumptions**

1. The first simplifying assumption will be to assume that the structure will have a fundamental period that places it in the velocity- constant region as shown in Fig. 2.10 this establishes equivalence between the peak deformations of the elastic and inelastic structures as shown in Fig. 2.8.

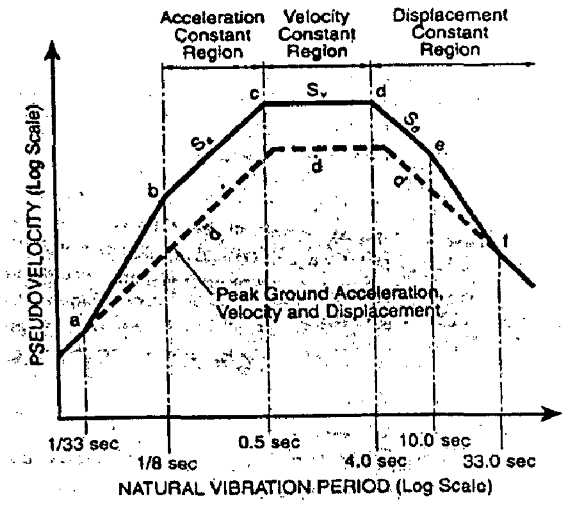



**Fig. 2.10. Elastic response spectra** *[Englekirk (2003)]*

2. Second, for conceptual design purposes, the behavior of the system will be presumed to be elastic/perfectly plastic, and the behavior characteristics of the frame components will be assumed identical

### 2.3.4.1 Equal displacement based design

Let a single degree of freedom system with mass m, and the storey height $h_x$.

*Step 1: Establish the Objective Drift Limit.*

If we assume Drift Objective of 2%, then the Objective Level of Drift is

$$\Delta_u = 0.02 \times h_X$$

*Step 2: Determine the Objective Natural Frequency $\omega_n$ Based on a Criterion of Spectral Velocity*

$$\omega_n = \frac{S_v}{\Delta_u}$$

*Step 3: Determine the Objective Stiffness*

$$K = \omega^2 m$$

*Step 4: Size the Frame Components:*

The relation between force and displacement for the structure with its support conditions is taken i.e. fixed base frame or hinged base frame. The lateral stiffness of the frame will be determined by proportioning the beam and column strengths and from them static condensation method is used for the elimination of rotational degrees of freedom present in the global stiffness matrix of frame The stiffness will be in the form as given in the equation below with a constant term $\alpha$ that is constant for individual frames according to their proportioning..

$$K = \frac{\alpha EI}{L^3}$$

Keeping the width of column constant we can obtain the depth of the beam

*Step 5: Determine the Strength Required of the Beam and Column:*

Taking the assumed system ductility factor ($\mu$) or the ratio of idealized yield displacement ($\Delta_{yi}$) to ultimate displacement ($\Delta_u$). Accordingly, this will produce an estimate of the ultimate strength required of the system ($F_{max}$). The idealized elastic frame displacement is

$$\Delta_{yi} = \frac{\Delta_u}{\mu}$$

and the required ultimate strength, expressed as a mechanism shear force imposed on the frame, is $F_{\max} = \Delta_{yi} \times K$ .

*Step 6: Consider the Impact of PΔ Forces:*

PΔ forces are created at a displacement of $\Delta_u$

$$F_{p\Delta} = \frac{\left(\sum W\right)\Delta_u}{h_x}$$

Therefore the objective level of strength is ($F_{\max} + F_{p\Delta}$).

### 2.3.4.2 Direct displacement based design

*Step 1: Establish the Objective Drift Limit.*

If we assume Drift Objective of 2%, then the Objective Level of Drift is

$$\Delta_u = 0.02 \times h_X$$

*Step 2: Revise the Design Spectral Velocity to Reflect the Level of Provided Structural Damping*

$$\zeta_{eq} = \frac{\sqrt{\mu} - 1}{\pi\sqrt{\mu}}, \dot{d}_{\max} = \frac{S_v}{3.38 - 0.67 \ln 5}, \hat{\zeta}_{eq} = \zeta + \zeta_{eq}, S_v = (3.38 - 0.67 \ln \hat{\zeta}_{eq})\dot{d}_{\max}$$

*Step 3: Determine the Objective Natural Frequency $\omega_{n,ductile}$ of the Ductile Structure Based on a Criterion of Spectral Velocity* $\omega_n = \dfrac{S_v}{\Delta_u}$

*Step 4: Determine the Objective Natural Frequency $\omega_{n,elastic}$ of the Elastic Structure*

$$\omega_{n,elastic} = \omega_{n,ductile}\sqrt{\mu}$$

*Step 5: Determine the Stiffness required of the elastic structure.* $K = \omega_n^{\ 2} m$

*Step 6: Proceed to Develop the Stiffness Required of the components*:

The relation between force and displacement for the structure with its support conditions is taken i.e. fixed base frame or hinged base frame. The lateral stiffness of the frame will be determined by proportioning the beam and column strengths and from them static condensation method is used for the elimination of rotational degrees of freedom present in the global stiffness matrix of the frame The stiffness will be in the form as given in the equation below with a constant term $\alpha$ that is constant for individual frames according to their strength proportioning. $K = \dfrac{\alpha\, EI}{L^3}$

Keeping the width of column constant we can obtain the depth of the beam

*Step 7: Determine the Strength Required of the Column:*

Taking the assumed system ductility factor ($\mu$) or the ratio of idealized yield displacement ($\Delta_{yi}$) to ultimate displacement ($\Delta_u$). Accordingly, this will produce an estimate of the ultimate strength required of the system ($F_{max}$). The idealized elastic frame displacement is $\Delta_{yi} = \dfrac{\Delta_u}{\mu}$

and the required ultimate strength, expressed as a mechanism shear force imposed on the frame, is $F_{max} = \Delta_{yi} \times K$

*Step 8: Consider the Impact of $P\Delta$ Forces:*

$P\Delta$ forces are created at a displacement of $\Delta_u$ $F_{p\Delta} = \dfrac{\left(\sum W\right)\Delta_u}{h_x}$

Therefore the objective level of strength is ($F_{max} + F_{p\Delta}$).

### 2.3.5 Design soft storey with restricted ductility *[Paulay et al., (1992)]*

As we know due to soft storey mechanism plastic hinge form in column these column hinges will now be subjected to large plastic rotation and hence large member ductility demands. In this case of low-rise buildings such as one or two storey frame buildings, we can provide full ductility demand. But in case of high rise buildings the member ductility demand is more than low-rise buildings. In such situation, the detailing for full; ductility is found to be difficult and considered to be too costly. So we reduce the ductility demand (or design the compound of the frame with restricted ductility) by adoption of larger seismic design forces

### 2.3.6 General Solution

(1) Other approach to solve this problem is by increasing the stiffness of the first storey such that the first storey is at least 50% as stiff as the second storey. The possible schemes to achieve the above are follows *[Arlekar et al., (1997)]*

To provide stiffer columns in the first storey than second storey.

To provide RC elevator cores: Due to their large size, the RC elevator cores offer much larger stiffness compared to other and results in resist larger seismic forces.

(2) Practical solution to this problem lies in either providing light weight easily collapsible partition or by isolating the stiff non structural partitions from frame so that the stiffness, strength and frame work of the structure are uniform from one floor to another *[Khanna et al., (1997)]*.

Following are the design solutions for soft storey recommended by *[Earthquake Hazard News Letter, 2000, W5]*

(3) Design the soft storey frame to with stand seismic load elastically. This means avoiding the need for frame ductility by designing for loads about four to six times larger. Even then this approach can't guarantee safety in the event of an earthquake intensity exceeding the code design level.

- Provide an alternative structural system such as reinforced concrete shearwall else where in the building plan. If the wall is designed to resist the shear forces, bending and overturning moments from the building lateral loads due to its greater stiffness it will attract the earthquake loads to itself and protect the soft storey frame damages.

- Physically separate infill walls above the soft storey with continuous gap so no storey is significantly stronger than other and also light weight material or hollow construction

(4) Introducing bracing that provides very high lateral strength and stiffness. The bracing members may be provided in X- shape in selected bays of the building maintaining its symmetry.

(5) To increase the size of columns in the open storey or change the design of first storey columns or add the column at first for enhanced stiffness *[Farzad Naeim (1984)]*

(6) To design the columns for large shear strength for avoiding shear failure of columns at first storey.

If the columns of first storey were designed on the basis of stress criteria and storey deflection then the first storey building do not necessarily have soft storey. But failure results from unfavorable reasons, such as torsion, excessive mass on the upper floor, P-delta effect and lack of ductility in the bottom storey.

Chapter 3

# DYNAMIC ANALYSIS AND DESIGN OF BUILDING PLANE FRAME

## 3.1 INTRODUCTION

Earthquake resistant design of a structure is the design of the building to the probable lateral loads that would act on the structure in an event of a seismic action. The seismic force that is coming on to a structure depends on the seismic mass, lateral storey stiffness, damping of the structure and the founding soil strata. Therefore to determine the lateral forces it is required to perform dynamic analysis of the building either by the time history or by the response spectrum method of analysis. According to the *IS 1893 (Part 1):2002*, the seismic forces can be found out by the code specified design response spectra.

In the present dissertation work it is intended to study the various design methods available or applicable to a soft storey design solution, hence two plane frame structures one of single bay and the other of Two-bay with eleven storeys i.e. (G+10) storey is considered for analysis and design as shown in Fig. 3.1. The ground storey of the two plane frames is kept open so that possible soft storey configuration occurs which is intended to study. The ground floor height is also kept higher than other stories.

The two plane frames were modeled in STAAD Pro 2001 soft ware package for analysis and design. Preliminary data pertaining to the frames is given in Table 3.1. The node, column and beam numbers modeled in STAAD are as shown in the Fig. 3.2. The gravity loads (imposed and dead loads) on to the structure is taken as per *IS 875 (Part 1 & 2):1987*. Loading calculation was given in appendix-A is shown in Figs. 3.3 to 3.5 for both one-bay and Two-bay frames. For response spectrum analysis Seismic Weight is calculated as per *IS 1893 (Part 1):2002* and applied as nodal loads at the nodes of tributary floor area surrounding the node. The infills that are present in the upper floors were considered for seismic weight and gravity load calculation only, but its stiffness not considered in the dynamic analysis.

Dynamic analysis results for one-bay frame are presented in Tables 3.2 to 3.3 and Tables 3.4 to 3.5 for two-bay frames. Preliminary design is carried out for the maximum action affects of all load combinations specified in *IS 1893 (Part 1):2002* with initially assumed trial sections for members. Then the analysis and design is carried out in iterations by changing the analysis and design sections at every stage with the criteria of getting all the column designs for an interaction ratio of around 0.9 to 1.0, so that the design sections become economical and also the material stresses can be utilized effectively.

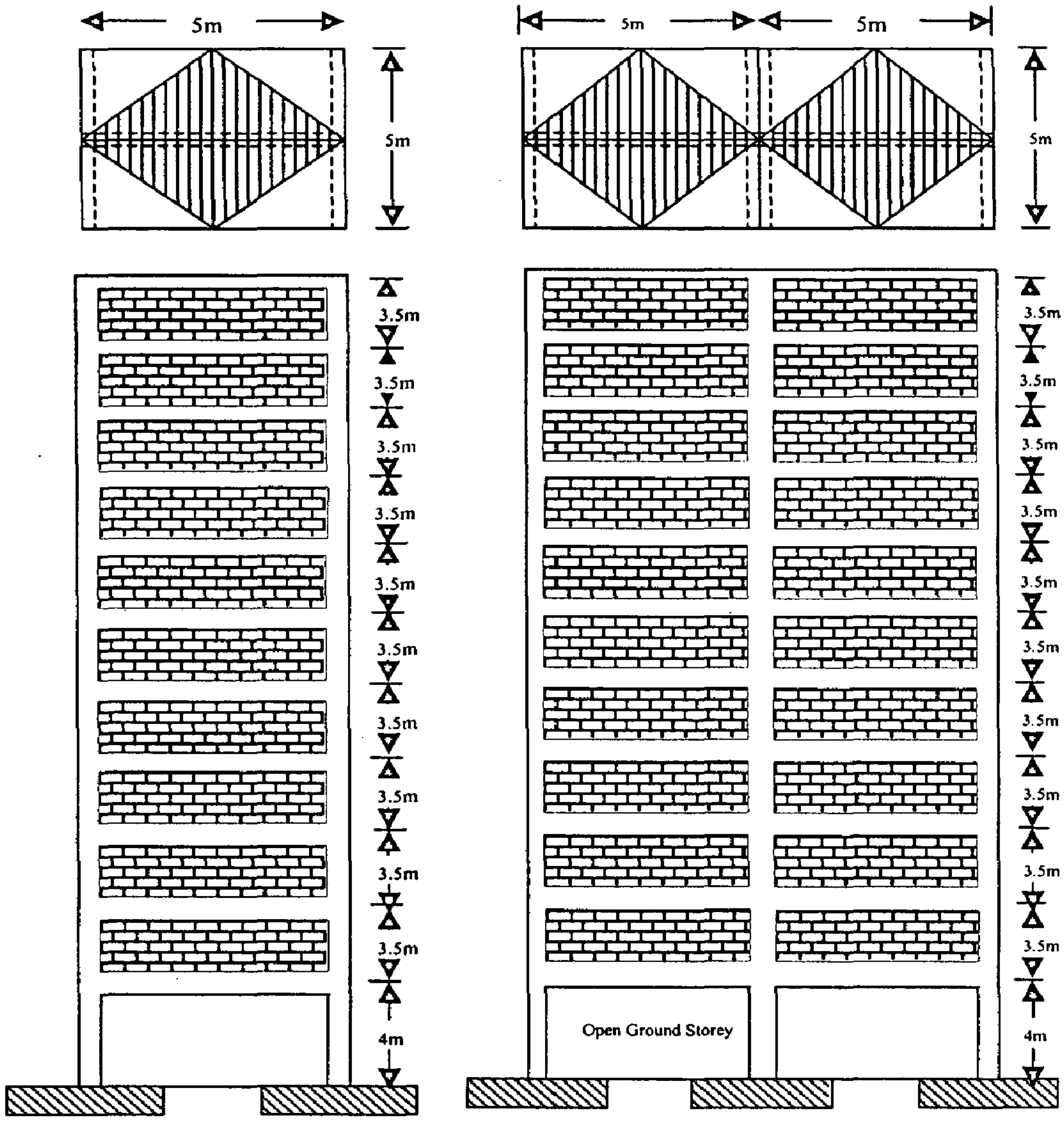



**Fig. 3.1 Plan and elevation of G+10 storeyed plane frame building's one-bay and two-bay frames with soft storey**

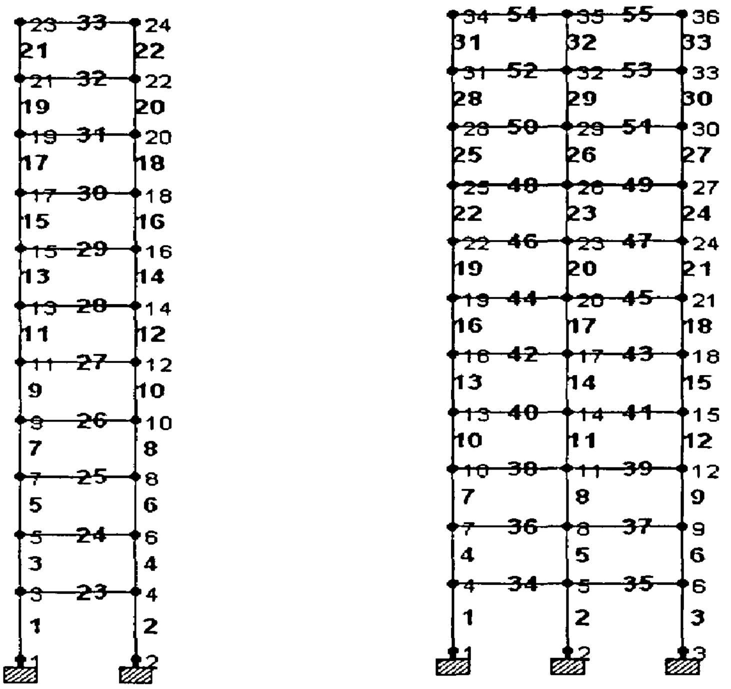
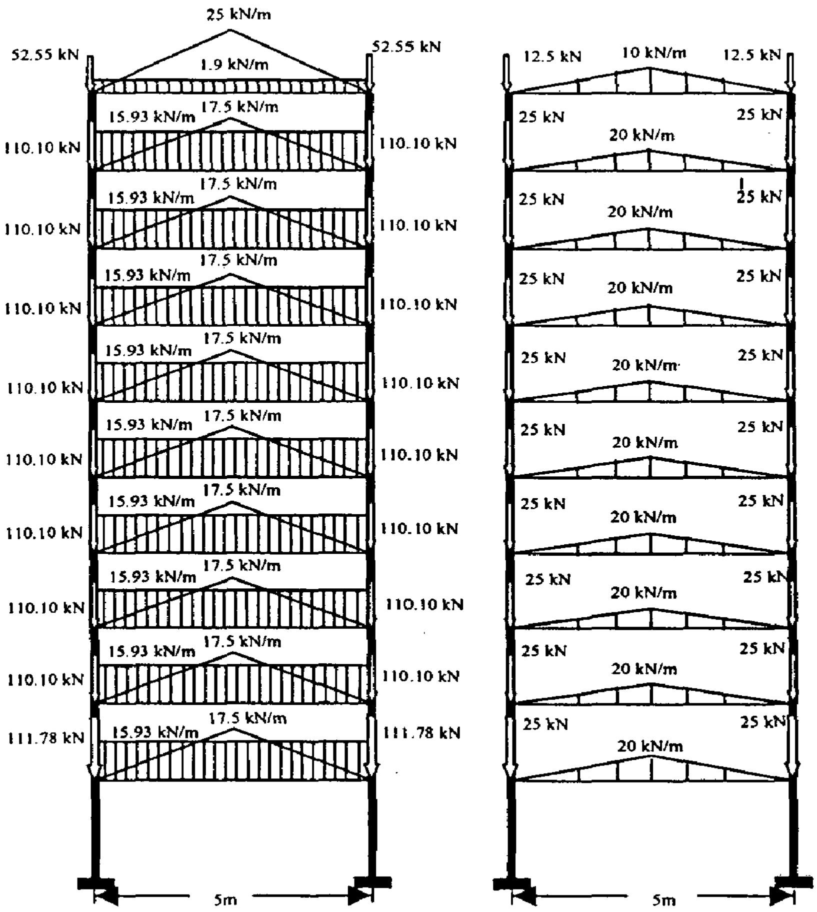


**Fig. 3.2 Beam numbers and node numbers of (G+10) storeyed one-bay and two-bay plane frame STAAD models**




(a) (b)

**Fig. 3.3 (a) Dead load on one-bay plane frame (b) Live load on two-bay plane frame**

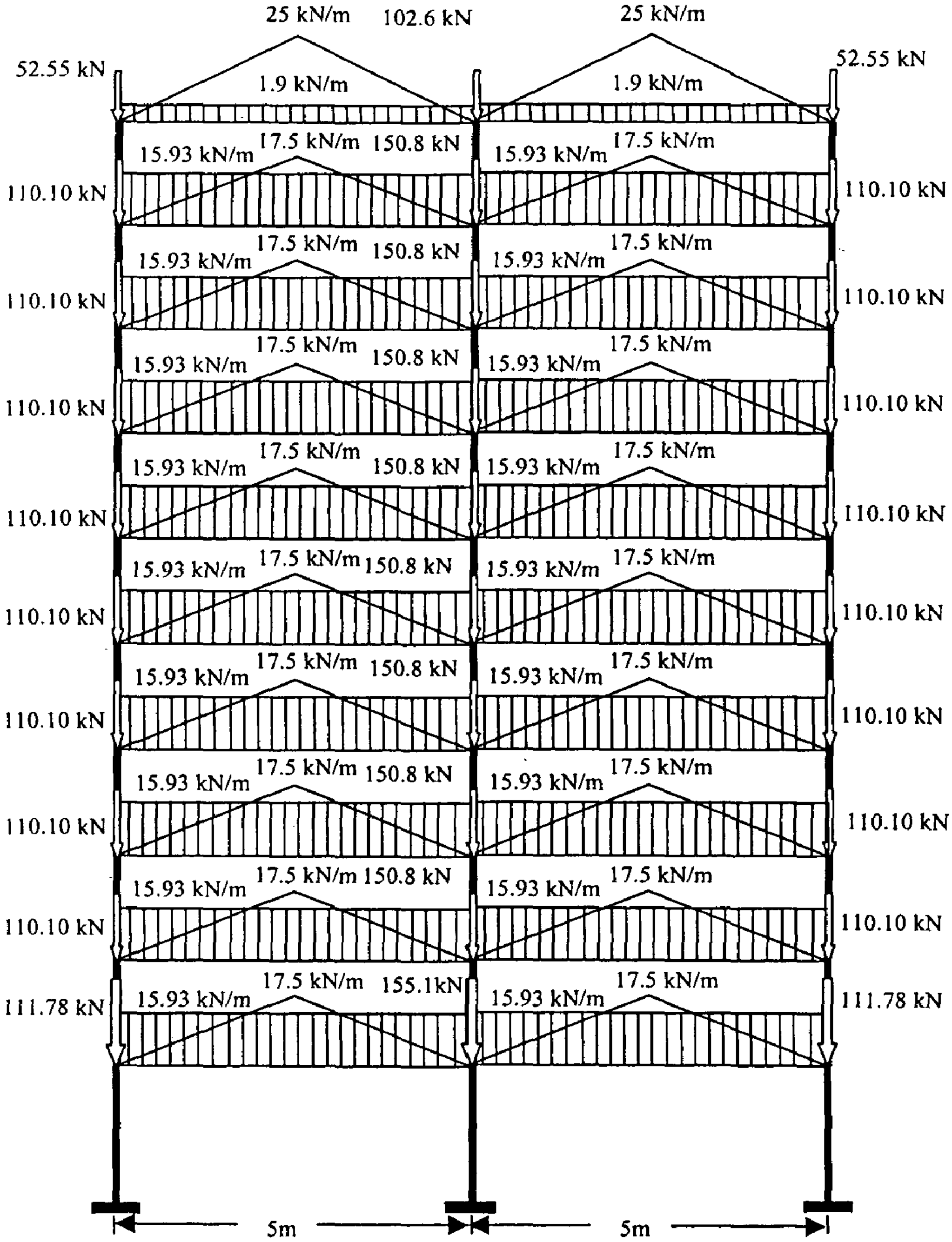



**Fig. 3.4. Dead load details on G+10 storeyed two-bay plane frame with open storey**

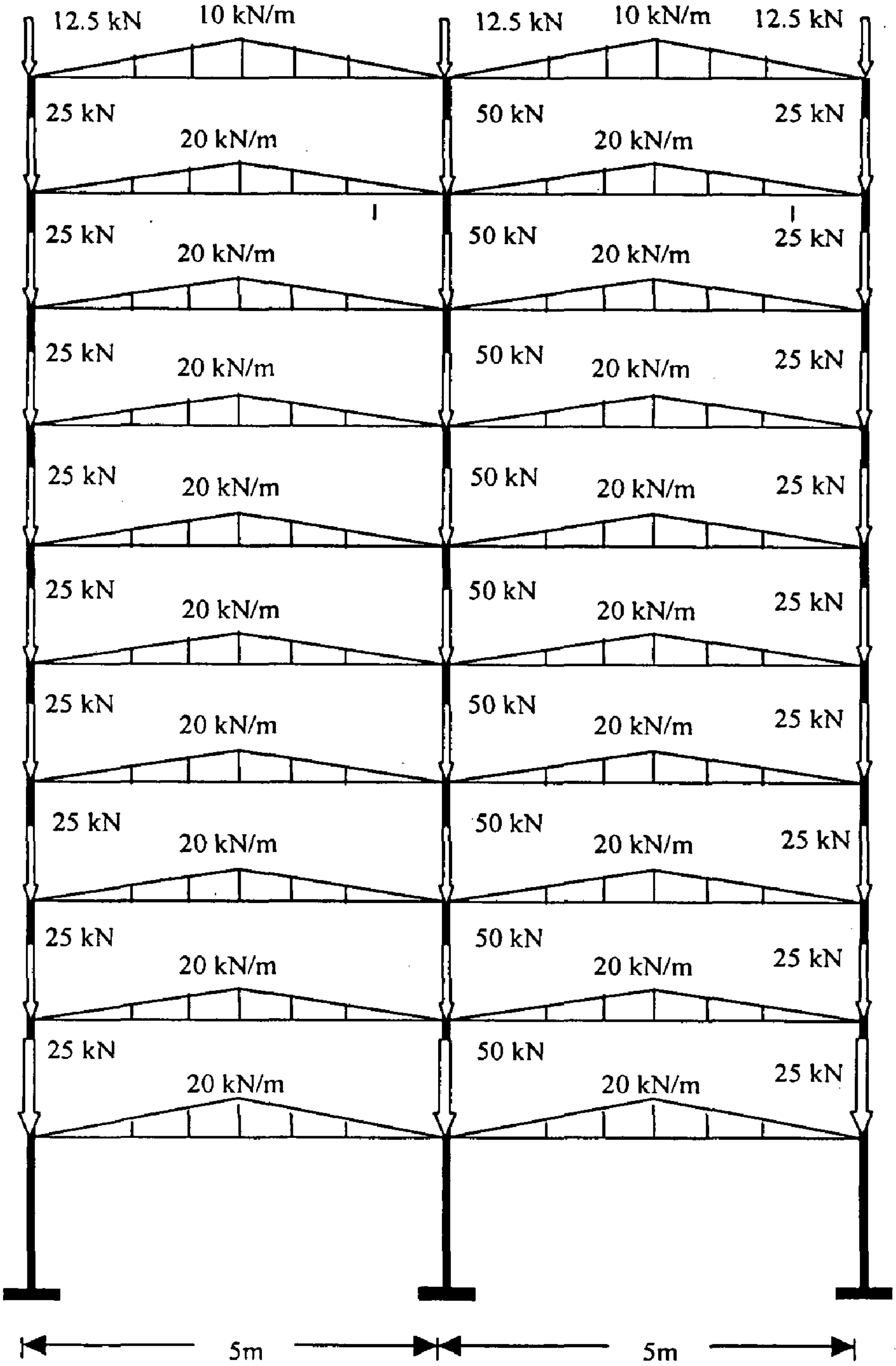



**Fig. 3.5. Live load details on G+10 storeyed two-bay plane frame with soft storey**

**Table 3.1 Preliminary data of G+10 plane frame building with an open ground storey**

| | | |
|---|---|---|
| 1 | Type of structure | Eleven storeyed rigid jointed plane frame |
| 2 | Zone | V |
| 3 | Layout | As shown in Fig 3.1 |
| 4 | Number of Storey | (G+10) with open ground storey |
| 5 | Live load at roof | 2 kN/m2 |
| 6 | Live load at floor | 4 kN/m2 |
| 7 | Terrace water proofing(TWF) | 1.5 kN/m2 |
| 8 | Floor finish | 0.5 kN/m2 |
| 9 | Materials | M20 concrete and Fe415 steel |
| 10 | Unit weight of RCC | 25 kN/m2 |
| 11 | Unit weight of masonry | 20 kN/m2 |
| 12 | Modulus of Elasticity of Concrete | 2.2360679 x 107 kN/m2 |
| 13 | Modulus of Elasticity of Masonry | 1.38 x 107kN/m2 |
| 14 | Bay width of plane frame | 5 m |
| 15 | Total height of building | 39 m |
| 16 | Height of ground storey | 4 m |
| 17 | Height of 1st storey till 11 th storey | 3.5 m |

## 3.2 DYNAMIC ANALYSIS OF SOFT STOREY PLANE FRAME BUILDING

According to *IS 1893(Part 1):2002* Dynamic analysis may be performed either by the Time History Method or by the Response Spectrum Method. However in either method, the design base shear ( $V_B$ ) shall be compared with a base shear ( $\overline{V_B}$ ) calculated using $T_a$, ( an approximate fundamental natural period of vibration in seconds, of a moment-resisting frame building without brick infill panels, $T_a = 0.075h^{0.75}$ for RC frame building with out infills. Where

$h$ = Height of building, in m. This excludes the basement storeys, where basement walls are connected with the ground floor deck or fitted between the building columns. But, it includes the basement storeys, when they are not so connected.) Where $V_B$ is less than $\overline{V_B}$, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $\overline{V_B}/V_B$.

### 3.2.1 Response Spectrum Method

Response Spectrum method of analysis shall be performed using the design spectrum specified in 6.4.2. of *IS 1893 (Part 1):2002.*

Hence the dynamic analysis of the G+10 storeyed plane frame with open ground storey is carried out using STAAD Pro. Software by Response Spectrum Method, for the code specified design spectrum, the results of the Dynamic analysis are presented in Table 3.2

for one bay frame and Table 3.4 for two bay frame respectively. The beam and column element action effects from the dynamic analysis are given in Table 3.3 for one bay frame and in Table 3.5 for two bay frame. The design of the beams and columns were done for the Maximum action effects from the load combinations specified in *clause 6.3.1.2 IS 1893 (Part 1):2002, Partial safety factors for the limit state design of reinforced concrete and pre-stressed concrete structures.*

**Table 3.2 Dynamic properties of building (one bay frame)**

| | | |
|---|---|---|
| Dynami Analysis using STAAD Pro 2001 | Seismic Weight in kN | 4200.5 |
| | Time Period for Staad model in sec | 3.45 |
| | Total Base Shear in kN | 39.22 |
| Fundamental Natural period Estimated by Emperical Expression($T_a$=0.075 $h^{0.75}$) | Seismic Weight in kN | 4200.5 |
| | Time Period for Staad model in sec | 1.17 |
| | Total Base Shear in kN | 129.18 |
| Response Revision Factor According to Clause 7.8.2 of IS 1893:2002 | $\bar{V}_B / V_B$ | 3.29 |
| Revised Dynamic Analysis using STAAD Pro 2001 after applying Response Revision Factor | Seismic Weight in kN | 4200.5 |
| | Time Period for Staad model in sec | 3.454 |
| | Total Base Shear in kN | 129.18 |

**Table 3.3 Analysis results from STAAD Pro.2001 (one bay frame)**

| Storey No | Beam size (mm x mm) | Moment at left and right end of Beam Hogg/Sagg (kNm) | Reinforcement for left and right end of Beam Hogg/Sagg ($mm^2$) | Max Shear at end of beam (kN) | Column Size (mm x mm) | Axial Load ( kN) | Biaxial Bending Moments on Column Mx, My in kNm |
|---|---|---|---|---|---|---|---|
| 11 | 350 x 450 | 110/32 | 1130.97/565.48 | 78 | 350 x 500 | 157 | 109 - 4 |
| 10 | 350 x 450 | 187/59 | 1809.56/904.78 | 138 | 350 x 500 | 457 | 134 - 10 |
| 9 | 350 x 450 | 232/104 | 2148/1130.97 | 154 | 350 x 500 | 776 | 146 - 16 |
| 8 | 350 x 450 | 264/136 | 2412/1407.43 | 166 | 350 x 500 | 1107 | 150 - 22 |
| 7 | 350 x 450 | 287/160 | 2613.6/1696.4 | 176 | 350 x 500 | 1448 | 160 - 29 |
| 6 | 350 x 450 | 310/183 | 3015.93/1884.96 | 185 | 350 x 500 | 1797 | 170 - 36 |
| 5 | 350 x 450 | 330/202 | 3015.95/2035.75 | 193 | 350 x 500 | 2154 | 175 - 43 |
| 4 | 350 x 450 | 343/216 | 3455.75/2199.11 | 198 | 350 x 500 | 2518 | 176 - 50 |
| 3 | 350 x 450 | 350/222 | 3455.75/2412.74 | 201 | 375 x 500 | 2884 | 208 - 58 |
| 2 | 350 x 450 | 334/204 | 3141.59/2148.85 | 194 | 500 x 550 | 3242 | 287 - 77 |
| 1 | 350 x 450 | 226/136 | 2412.74/1470.27 | 166 | 550 x 600 | 3604.5 | 395 - 94 |

**Table 3.4 Dynamic properties of building (two bay frame)**

| | | |
|---|---|---|
| Dynami Analysis using STAAD Pro 2001 | Seismic Weight in kN | 7115.9 |
| | Time Period for Staad model in sec | 3.20 |
| | Total Base Shear in kN | 70.78 |
| Fundamental Natural period Estimated by Emperical Expression($T_a=0.075\ h^{0.75}$) | Seismic Weight in kN | 7115.9 |
| | Time Period for model in sec | 1.17 |
| | Total Base Shear in kN | 218.45 |
| Response Revision Factor According to Clause 7.8.2 of IS 1893:2002 | $\overline{V_B}/V_B$ | 3.09 |
| Revised Dynamic Analysis using STAAD Pro 2001 after applying Response Revision Factor | Seismic Weight in kN | 7115.9 |
| | Time Period for Staad model in sec | 3.20 |
| | Total Base Shear in kN | 218.45 |

Table 3.5 Analysis results from STAAD Pro.2001 for beams (two bay frame)

| Storey No | Beam No | Beam Size (mmxmm) | Moment at left end Hogg/Sagg (kNm) | Moment at right end Hogg/Sagg (kNm) | Reinforcement at left end Hogg/Sagg ($mm^2$) | Reinforcement at right end Hogg/Sagg ($mm^2$) | Max Shear at end of beam (kN) |
|---|---|---|---|---|---|---|---|
| 11 | 54,55 | 300x400 | 109.9/9.51 | 80.8/22.78 | 1099.56/339.3 | 706.86/339.3 | 81.68 |
| 10 | 52,53 | 300x400 | 190.9/32.1 | 151.19/51.6 | 1806.42/785.4 | 1413.72/471.24 | 142.13 |
| 9 | 50,51 | 300x400 | 229/75.68 | 194.12/90.62 | 2148.85/1099.56 | 1809.56/863.94 | 154.01 |
| 8 | 48,49 | 300x400 | 257.7/103.56 | 219.6/114.6 | 2412.74/1357.17 | 2010.62/1130.97 | 162.36 |
| 7 | 46,47 | 300x400 | 275.7/124.31 | 242.86/133.4 | 2513.27/1570.8 | 2199.11/1256.64 | 168.89 |
| 6 | 44,45 | 300x400 | 292.8/144.8 | 265.4/152.7 | 2814.87/1696.46 | 2412.74/1470.26 | 176.03 |
| 5 | 42,43 | 300x400 | 308.3/163.2 | 285/169 | 3015.93/1884.96 | 2613.8/1884.96 | 182.21 |
| 4 | 40,41 | 300x400 | 319.39/176.5 | 299.4/181.5 | 3015.93/2010.62 | 2814.87/1809.56 | 186.84 |
| 3 | 38,39 | 300x400 | 328.41/184.37 | 307.2/189 | 3015.93/2035.75 | 2814.87/1922.65 | 90.11 |
| 2 | 36,37 | 300x400 | 319.54/176.25 | 301.62/180.41 | 3015.93/2010.62 | 2814.87/1809.56 | 186.48 |
| 1 | 34,35 | 300x400 | 264.2/129.4 | 257.9/128.4 | 2412.74/1470.26 | 2412.74/1470.26 | 164.62 |

Table 3.6 Analysis results of STAAD Pro for columns (two bay frame)

| Storey ID No. | Column ID No. | Size (mm x mm) | Axial load Pu-kN | Mx Mux-kNm | My Muy-kNm |
|---|---|---|---|---|---|
| 11 | 31,33 | 350x450 | 156.5 | 109.94 | 3.13 |
| 11 | 32 | 350x500 | 150.8 | 89.08 | 3.02 |
| 10 | 28,30 | 350x450 | 457.7 | 126.29 | 9.15 |
| 10 | 29 | 350x500 | 388.7 | 137.53 | 7.77 |
| 9 | 25,27 | 350x450 | 774.2 | 134.28 | 15.48 |
| 9 | 26 | 350x500 | 1046.6 | 164.27 | 20.93 |
| 8 | 22,24 | 350x500 | 1101.1 | 151.55 | 22.02 |
| 8 | 23 | 350x500 | 1445.6 | 165.86 | 28.91 |
| 7 | 19,21 | 350x500 | 1435.1 | 147.2 | 28.7 |
| 7 | 20 | 450x500 | 1846.4 | 204.87 | 40.62 |
| 6 | 16,18 | 350x500 | 1776.3 | 162.87 | 35.53 |
| 6 | 17 | 450x500 | 3031.3 | 71.74 | 66.69 |
| 5 | 13,15 | 350x500 | 2123.6 | 162.37 | 42.47 |
| 5 | 14 | 450x500 | 3580.4 | 84.74 | 78.77 |
| 4 | 10,12 | 350x500 | 2475.6 | 165.91 | 49.51 |
| 4 | 11 | 500x500 | 4131.5 | 97.78 | 97.78 |
| 3 | 7,9 | 400x550 | 2830.9 | 171.45 | 57.56 |
| 3 | 8 | 500x550 | 4682.8 | 118.63 | 110.83 |
| 2 | 4,6 | 500x550 | 3182.5 | 248.21 | 75.31 |
| 2 | 5 | 500x600 | 5235.8 | 141.37 | 123.91 |
| 1 | 1,3 | 500x600 | 3514 | 440.69 | 86.7 |
| 1 | 2 | 550x600 | 5801.5 | 162.44 | 152.77 |

Table 3.7 Specifications of equivalent diagonal strut and its lateral stiffness (one-bay)

| Storey | Length(Ld) m | Width (W) m | Area (Ad) m2 | Thickness (t) m | Lateral Stiffness kN/m |
|---|---|---|---|---|---|
| 1 | 0.00 | 0.00 | 0.000 | 0 | 0 |
| 2 | 5.39 | 0.95 | 0.220 | 0.23 | 382232.55 |
| 3 | 5.39 | 0.94 | 0.217 | 0.23 | 377305.86 |
| 4 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 5 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 6 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 7 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 8 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 9 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 10 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |
| 11 | 5.44 | 0.93 | 0.215 | 0.23 | 373124.87 |

Table 3.8 Summary of lateral storey stiffness of infill and columns (one-bay)

| Storey | Size of Column ( mm x mm ) | Stiffness of columns ( kN/m) | Stiffness of Infill ( kN/m) | Total stiffness (kN/m) |
|---|---|---|---|---|
| 1 | 550 x 600 | 118753.49 | 0 | 118753.49 |
| 2 | 500 x 550 | 131121.39 | 382232.55 | 513353.94 |
| 3 | 375 x 550 | 98341.04 | 377305.86 | 475646.91 |
| 4 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 5 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 6 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 7 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 8 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 9 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 10 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |
| 11 | 350 x 500 | 68959.41 | 373124.87 | 442084.28 |

**Table 3.9 Check for soft storey formation in (G+10) plane frame in all storeys (one bay)**

| Storey | Lateral Storey Stiffness kN/m | Check 1 (Ki/Ki+1) | Check 2 (Ki/(((Ki+1)+(Ki+2)+(Ki+3))/3) | Remarks for Check1 | Remarks for Check2 |
|---|---|---|---|---|---|
| 1 | 118753.49 | 0.23 | 0.25 | Soft storey | Soft storey |
| 2 | 513353.94 | 1.08 | 1.13 | Not a soft Storey | Not a soft Storey |
| 3 | 475646.91 | 1.08 | 1.08 | Not a soft Storey | Not a soft Storey |
| 4 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 5 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 6 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 7 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 8 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 9 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 10 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 11 | 442084.28 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |

Therefore the above calculations show that, the considered eleven storey (G+10) R.C building plane frame was a soft storey building frame at ground storey as the lateral storey stiffness of the ground storey was less than the upper storey by 70% for check-1 and by 80% for check-2.

Similarly the soft storey check is also carried for the designed sections of the G+10 storeyed two bay building plane frame. The specifications of equivalent diagonal strut is given in Table 3.5, summary of stiffness contribution from columns and infill walls of all floors is given in Table 3.6. The soft storey check is given in Table 3.7

**Table 3.10 Specifications of equivalent diagonal strut and its lateral stiffness (two-bay)**

| Storey | Length(Ld) m | Width (W) m | Area (Ad) m2 | Thickness (t) m | Lateral Stiffness kN/m |
|---|---|---|---|---|---|
| 1 | 0.00 | 0.00 | 0.000 | 0 | 160458.58 |
| 2 | 5.42 | 0.86 | 0.199 | 0.23 | 886508.59 |
| 3 | 5.42 | 0.85 | 0.196 | 0.23 | 818887.47 |
| 4 | 5.46 | 0.84 | 0.193 | 0.23 | 772771.25 |
| 5 | 5.46 | 0.84 | 0.193 | 0.23 | 768080.09 |
| 6 | 5.46 | 0.84 | 0.193 | 0.23 | 768080.09 |
| 7 | 5.46 | 0.84 | 0.193 | 0.23 | 768080.09 |
| 8 | 5.46 | 0.84 | 0.193 | 0.23 | 758697.78 |
| 9 | 5.51 | 0.83 | 0.191 | 0.23 | 733957.35 |
| 10 | 5.51 | 0.83 | 0.191 | 0.23 | 733957.35 |
| 11 | 5.51 | 0.83 | 0.191 | 0.23 | 733957.35 |

**Table 3.11 Summary of lateral storey stiffness of infill and columns (two-bay)**

| Storey | Size of Column ( mm x mm ) | Stiffness of columns ( kN/m) | Stiffness of Infill ( kN/m) | Total stiffness (kN/m) |
|---|---|---|---|---|
| 1 | 550x600,500 x 600 | 160458.58 | 0 | 160458.58 |
| 2 | 500x600,500x550 | 205941.78 | 680566.81 | 886508.59 |
| 3 | 500x500,400x550 | 146814.44 | 672073.03 | 818887.47 |
| 4 | 500x500,350x500 | 112587.76 | 660183.49 | 772771.25 |
| 5 | 450x500,350x500 | 107896.60 | 660183.49 | 768080.09 |
| 6 | 450x500,350x500 | 107896.60 | 660183.49 | 768080.09 |
| 7 | 450x500,350x500 | 107896.60 | 660183.49 | 768080.09 |
| 8 | 350x500,350x500 | 98514.29 | 660183.49 | 758697.78 |
| 9 | 350x500,350x450 | 80716.04 | 653241.31 | 733957.35 |
| 10 | 350x500,350x450 | 80716.04 | 653241.31 | 733957.35 |
| 11 | 350x500,350x450 | 80716.04 | 653241.31 | 733957.35 |

**Table 3.12 Check for soft storey formation in (G+10) plane frame in all storeys (two bay)**

| Storey | Total Lateral Storey Stiffness kN/m | Check 1 (Ki/Ki+1) | Check 2 (Ki/(((Ki+1)+(Ki+2)+(Ki+3))/3) | Remarks for Check1 | Remarks for Check2 |
|---|---|---|---|---|---|
| 1 | 160458.58 | 0.18 | 0.19 | Soft storey | Soft storey |
| 2 | 886508.59 | 1.08 | 1.13 | Not a soft Storey | Not a soft Storey |
| 3 | 818887.47 | 1.06 | 1.06 | Not a soft Storey | Not a soft Storey |
| 4 | 772771.25 | 1.01 | 1.01 | Not a soft Storey | Not a soft Storey |
| 5 | 768080.09 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 6 | 768080.09 | 1.00 | 1.02 | Not a soft Storey | Not a soft Storey |
| 7 | 768080.09 | 1.01 | 1.03 | Not a soft Storey | Not a soft Storey |
| 8 | 758697.78 | 1.03 | 1.03 | Not a soft Storey | Not a soft Storey |
| 9 | 733957.35 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 10 | 733957.35 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |
| 11 | 733957.35 | 1.00 | 1.00 | Not a soft Storey | Not a soft Storey |

Therefore the above calculations show that, the considered eleven storey (G+10) two bay RC building plane frame was a soft storey building frame at ground storey as the lateral storey stiffness of the ground storey was less than the upper storey by 70% for check-1 and by 80% for check-2.

# SEISMIC DESIGN APPROACHES FOR SOFT STOREY BUILDING

## 4.1 GENERAL

Design of soft storey is still left open as there is no perfect design concept such that the designed soft storey is safe from the performance point of view in the future seismic events. There are various methods that have been published in literature which will lead to design with some assumptions at every stage. The best design method can be rated only from the observed performance with respect to structural safety and residual strength and economic considerations for applicability of a specific method.

In this dissertation it is intended to compare the various design methods from the design point of view by applying for a same soft storey problem with same loading conditions. The various methods that are taken for applicability are given below.

## 4.2 METHODS OF DESIGN

### 4.2.1 IS 1893 (Part 1):2002 Code Design Criteria

According to the clauses specified in the *IS 1893 (Part 1):2002*, Dynamic Analysis is to be carried out with out taking into account the stiffness of infill-walls and taking its mass participation. Therefore Dynamic analysis is performed using STAAD Pro 2001 software. The loading is taken as per IS 875. Gravity loading and seismic weight is calculated as per *IS 1893 (Part 1):2002*, which is carried out in the preliminary analysis and design in chapter-3.The action effects from the analysis are taken as the max from the load combinations specified in *IS 1893 (Part 1):2002* are given below

(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 designs for the various column members is carriedout by taking the interaction ratio of column subjected to axial load and biaxial bending into consideration according to Clause 39.6 of *IS 456:2000*. To keeping interaction ratio's range around 0.9-1, iterative design and analysis is done until all the columns comes in that range by changing the size of columns at every iteration . The dynamic analysis results of plane frame elements without infills in the upper storeys are presented in Tables 4.1 & 4.3 for one-bay and two-bay frames respectively. The design forces for soft storey elements (i.e., columns and beams) are obtained by applying the *Clause No. 7.10.3 of IS 1893 (Part 1):2002* and are tabulated in Tables 4.2 & 4.4 for one-bay and two bay frames respectively.

**Table 4.1 Action effects on soft storey elements from the analysis (one bay frame)**

| Storey No | Beam size (mm x mm) | Soft Storey Beam Hogg/Sagg Moment at left and right end of Beam(kNm) | Reinforcement for Hogg/Sagg Moment at left and right end of Beam($mm^2$) | Max Shear at end of beam (kN) | Column Size (mm x mm) | Axial Load (kN) | Biaxial Bending Moments Mx, My (kNm) |
|---|---|---|---|---|---|---|---|
| 1 | 350 x 450 | 226/136 | 2412.74/1470.27 | 166 | 550 x 600 | 3604.5 | 394.91-94 |

**Table 4.2 IS 1893:2002 Design forces for soft storey elements and their design (one-bay)**

| Element of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (500mm x 600mm) | 3604.5 | 987.27 / 237.3 | 4.71 | Section is insufficient, Revised to 600mmx 600mm, Ineraction Ratio - 1.0, |
| Beam no. 23 (350mm x 450mm) | - | Hogging-565 / Sagging- 340 | 2.2 / 1.34 | Section is insufficient, Revised to 350mm x 500mm |

**Table 4.3 Action effects on soft storey beam elements from the analysis (two bay)**

| Storey No | Beam No | Beam Size (mmxmm) | Beam size-Hogging/Sagging Moment at left and right end of Beam(kNm) | Reinforcement for Hogging/Sagging Moment at left and right end of Beam($mm^2$) | Max Shear at end of beam (kN) |
|---|---|---|---|---|---|
| 1 | 34,35 | 300x400 | 264.2/129.4---257.9/128.4 | 2412.74/1470.26--2412.74/1470.26 | 164.62 |

**Table 4.4 Action Effects on soft storey column elements from the analysis (two bay)**

| | | | Analysis Result STAAD | | | Design Result (Manual)using SP-16 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Storey No | Column No | Size | Axial load | Mx | My | Axial load | Mx | My | %steel | int ratio |
| | | (mmxmm) | Pu kN | Mux kNm | MuykNm | Puz kN | Mux1 kNm | Muy1 kNm | %steel | int ratio |
| 1 | 1,3 | 500x600 | 3514 | 440.69 | 86.7 | 5437.755 | 478.8 | 378 | 2.932 | 0.942 |
| 1 | 2 | 550x600 | 5801.5 | 162.44 | 152.77 | 6685.719 | 264 | 239.25 | 3.62 | 0.79 |

**Table 4.5 IS 1893:2002 Design forces for soft storey columns, beams and design (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (500mm x 600mm) | 3514 | 1101.72 / 216.75 | 4.712 | Section is insufficient, Revised to 600mmx 600mm, Ineraction Ratio - 1.0, |
| Column 2 (550mm x 600mm) | 5801.5 | 406.10 / 381.92 | 5 | Ineraction Ratio -0.92 |
| Beam nos. 34,35 (300mm x 400mm) (Exterior End) | - | Hogging-660.5 / Sagging- 323.5 | 2.15 / 1.29 | Section is insufficient, Revised to 350mm x 550mm |
| Beam nos. 34,35 (300mm x 400mm)) (Interior End) | - | Hogging- 644.75 / Sagging- 321.0 | 2.10 / 1.24 | Section is insufficient, Revised to 350mm x 550mm |

### 4.2.2 Capacity Based Design Criteria

To apply Capacity Based design concept according to EC8 to a soft storey problem, the same structure which is taken in the previous method ( IS 1893 (Part 1):2002), The G+10 storeyed RC building Plane frames, one-bay and two-bay's preliminary design forces from dynamic analysis with gravity load combinations are considered for further capacity design application for both the frames. The Capacity Based design concept according to EC-8 is given in chapter-3

The sequential steps that are performed to achieve capacity based design are as follows

1. Design the elements of the structure for the dynamic loading with gravity load combinations (Preliminary Design). Details are shown in Table 4.2
2. To ensure an **adequate ductility** at supports to all beams, revise the compression reinforcement equal to **50%** of the corresponding tension reinforcement. For an earthquake reversal situation, compression and tension comes at every face, so revise accordingly.
3. Determine the **bending capacity** of the beams for the provided reinforcement.

   At any section the max Hogging and Sagging moments from load combinations will be from different load cases and the steel provided will be different, so take into account the actual provided steel in the section considered and determine the capacity of the beam taking into account the tension steel as compressive reinforcement (Doubly Reinforced) in hogging case and compression steel as tension reinforcement with included flange action (T-beam) of slab concrete in sagging case.

4. Revise the **Design Shear** for beams corresponding to the equilibrium of the beam under appropriate gravity load and a rational adverse combination of the actual bending resistances of cross sections.
5. To decrease the probability of plastic hinge formation in the columns, frame is designed to have **"Strong Columns and Weak Beams"**, insisting that the sum of the resisting moments of the columns, taking into account the axial load, should be greater than the sum of the resisting moments of all adjacent resisting beams for each case ( positive or negative ) direction of the seismic action.
6. Determine the **moment magnification factor**; the capacity design will be satisfied if the columns are designed for the moments revised according to the max moment magnification factor (which is greater than 1) times the moment capacity of the column for both directions of earthquake motion at every joint. The greater of the moment revised at both top and bottom joint of every column is taken for final capacity design.
7. Determine the shear forces according to the capacity design criteria by considering the equilibrium of the column under actual resisting moments at its ends.

Applying the capacity based design for the one-bay and two bay frames , the calculated capacities for the provided reinforcement is given in Tables 4.6, 4.9,4.10 respectively and moment magnification factor in Tables 4.7,4.11 respectively. The Revision of moments are given in Table 4.8 for one-bay frame and in Table 4.12 for exterior columns, in Table 4.13 for interior columns of two bay frame The final design of capacity revised design moments is given in Table 4.14. The soft storey capacity of the structures are given in Table 4.15 for one bay frame and in Table 4.16 for two bay frame

**Table 4.6 Design capacities for beams and columns for provided actual reinforcement (one bay)**

| Storey No | Beam size(mmxmm)- Hogging/Sagging Moment capacity of left and right end of Beam(kNm) | Reinforcement for Hogging/Sagging Moment at left and right end of Beam($mm^2$) | Max Shear Capacity at end of beam (kN) | Column Size(mmxmm)- Axial Load Capacity of Column in kN | Biaxial Bending Moment capacity of Column Mx, My in kNm | Percentage of Steel(%) in Column | Interaction Ratio |
|---|---|---|---|---|---|---|---|
| 11 | 350 x 450 - 110/72 | 1130.97/565.485 | 114.7 | 350 x 500-1966.09 | 133 - 80.5 | 0.718 | 0.869 |
| 10 | 350 x 450 - 187/114 | 1809.56/904.78 | 145.125 | 350 x 500-1966.09 | 161 - 100.1 | 0.718 | 0.912 |
| 9 | 350 x 450 - 232/141 | 2148/1130.97 | 162.45 | 350 x 500 -1966.09 | 161 - 102.2 | 0.718 | 0.971 |
| 8 | 350 x 450 - 264/174 | 2412/1407.43 | 153.6 | 350 x 500-2161.63 | 164.5 -102.5 | 1.077 | 0.965 |
| 7 | 350 x 450 - 287/207 | 2613.6/1696.4 | 192.7 | 350 x 500-2552.82 | 192.5 - 119 | 1.790 | 0.845 |
| 6 | 350 x 450 - 310/228 | 3015.93/1884.96 | 208.8 | 350 x 500-2748.25 | 183.75 - 112.9 | 2.154 | 1.000 |
| 5 | 350 x 450 - 330/245 | 3015.95/2035.75 | 212.95 | 350 x 500-3139.34 | 192.5 - 115.5 | 2.872 | 1.000 |
| 4 | 350 x 450 - 343/263 | 3455.75/2199.11 | 230.3 | 350 x 500-3530.64 | 215.2 - 126.5 | 3.590 | 0.867 |
| 3 | 350 x 450 - 350/286 | 3455.75/2412.74 | 236.05 | 375 x 500-3983.98 | 259.3 - 155.9 | 3.310 | 0.818 |
| 2 | 350 x 450 - 334/258 | 3141.59/2148.85 | 219.625 | 500 x 550-5017.13 | 308.55 - 275 | 2.970 | 0.990 |
| 1 | 350 x 450 - 226/181 | 2412.74/1470.27 | 180.95 | 550 x 600-5708.31 | 487 - 442.2 | 2.670 | 0.991 |

**Table 4.7 Determination of moment magnification factors at a joint (one bay)**

| Joint No | Seismic Direction | Sum of Resisting Moments of Top & Bottom Columns at Joint | Sum of Resisting Moments of Left & Right Beams at Joint with an Over Strength Factor of 1.35 | Check for Sum of Column Resisting Moments Greater than Sum of Beam Resisting moments | Moment Magnification Factor αcd |
|---|---|---|---|---|---|
| 23,24 | 1 | (0 + 133) = 133 | 1.35 (0 + 110) = 148.5 | Not OK | 1.1165 |
| | 2 | (0 + 133) = 133 | 1.35 (0 + 72) = 97.2 | OK | 0.73 |
| 21,22 | 1 | (133 + 161) = 294 | 1.35 (0 + 187)= 252.4 | OK | 0.858 |
| | 2 | (133 + 161) = 294 | 1.35 (0 + 113.7) | OK | 0.522 |
| 19,20 | 1 | (161 + 161) = 322 | 1.35 (0 + 232)= 313.2 | OK | 0.973 |
| | 2 | (161 + 161) = 322 | 1.35 (0 + 141) =190.35 | OK | 0.591 |
| 17,18 | 1 | (161 + 164.5) = 325.5 | 1.35 (0 + 264)= 356.4 | Not OK | 1.095 |
| | 2 | (161 + 164.5) = 325.5 | 1.35 (0 +173.6)=234.4 | OK | 0.72 |
| 15,16 | 1 | (164.5 + 192.5) = 357 | 1.35 (0 + 287)= 387.45 | Not OK | 1.085 |
| | 2 | (164.5 + 192.5) = 357 | 1.35 (0 +207)=279.45 | OK | 0.783 |
| 13,14 | 1 | (192.5 + 183.75) = 376.2 | 1.35 (0 + 330)= 445.5 | Not OK | 1.184 |
| | 2 | (192.5 + 183.75) = 376.2 | 1.35 (0 +228.4)=308.3 | OK | 0.819 |
| 11,12 | 1 | (183.75 + 192.5) = 376.2 | 1.35 (0 + 330)= 445.5 | Not OK | 1.184 |
| | 2 | (183.75 + 192.5) = 376.2 | 1.35 (0 +245)=330.75 | OK | 0.879 |
| 9,10 | 1 | (192.5 + 215.25) = 407.75 | 1.35 (0 + 381)= 514.3 | Not OK | 1.261 |
| | 2 | (192.5 + 215.25) = 407.75 | 1.35 (0 +263)=355 | OK | 0.871 |
| 7,8 | 1 | (215.25 + 259.3) = 474.55 | 1.35 (0 + 381)= 514.3 | Not OK | 1.083 |
| | 2 | (215.25 + 259.3) = 474.55 | 1.35 (0 +286.4)=386.64 | OK | 0.815 |
| 5,6 | 1 | (259.3 + 308.55) = 567.85 | 1.35 (0 + 344)= 464.4 | OK | 0.817 |
| | 2 | (259.3 + 308.55) = 567.85 | 1.35 (0 +257.7)=347.89 | OK | 0.613 |
| 3,4 | 1 | (308.5 + 487.0) = 795.63 | 1.35 (0 + 266)= 359.1 | OK | 0.451 |
| | 2 | (308.5 + 487.0) = 795.63 | 1.35 (0 +181)=244.35 | OK | 0.307 |

**Table 4.8 Revision of column moments (one bay frame)**

| Joint No | Column No | Moment Magnification Factor (Greater of Direction 1&2 ) ≥1 | Column Moments | Revised Column Moments for magnification factor at respective joint | Revised Max Moment from Both joints of Column |
|---|---|---|---|---|---|
| 23,24 | - | 1.1165 | - | - | - |
| | 21,22 | | 133 | 148.5 | 148.5 |
| 21,22 | 21,22 | 1 | 133 | 133 | |
| | 19,20 | | 161 | 161 | 161 |
| 19,20 | 19,20 | 1 | 161 | 161 | |
| | 17,18 | | 161 | 161 | 176.28 |
| 17,18 | 17,18 | 1.095 | 161 | 176.28 | |
| | 15,16 | | 164.5 | 180.11 | 180.11 |
| 15,16 | 15,16 | 1.0852 | 164.5 | 178.51 | |
| | 13,14 | | 192.5 | 208.9 | 227.92 |
| 13,14 | 13,14 | 1.184 | 192.5 | 227.92 | |
| | 11,12 | | 183.75 | 217.56 | 217.56 |
| 11,12 | 11,12 | 1.184 | 183.75 | 217.56 | |
| | 9,10 | | 192.5 | 227.92 | 242.82 |
| 9,10 | 9,10 | 1.2614 | 192.5 | 242.82 | |
| | 7,8 | | 215.25 | 271.51 | 271.51 |
| 7,8 | 7,8 | 1.0838 | 215.25 | 233.28 | |
| | 5,6 | | 259.29 | 281.03 | 281.03 |
| 5,6 | 5,6 | 1 | 259.29 | 259.3 | |
| | 3,4 | | 308.55 | 308.55 | 308.55 |
| 3,4 | 3,4 | 1 | 308.55 | 308.55 | |
| | 1,2 | | 487.08 | 487.08 | 487.08 |
| 1,2 | 1,2 | 1 | 487.08 | 487.08 | |
| | - | | - | - | - |

**Table 4.9 Design capacities of beams for actually provided reinforcement (two bay frame)**

| Storey No | Beam No | Beam Size (mmxmm) | Beam size-Hogging/Sagging Moment at left and right end of Beam(kNm) | Reinforcement for Hogging/Sagging Moment at left and right end of Beam($mm^2$) | Max Shear at end of beam (kN) |
|---|---|---|---|---|---|
| 11 | 54,55 | 300x400 | 114.87 / 70.04---80.81 / 45.35 | 1099.56/339.3--706.86/339.3 | 79.80 |
| 10 | 52,53 | 300x400 | 195.99 / 113.58---151.18 / 89.54 | 1806.42/785.4--1413.72/471.24 | 140.58 |
| 9 | 50,51 | 300x400 | 231.48 / 137.27---196.91 / 113.78 | 2148.85/1099.56--1809.56/863.94 | 155.52 |
| 8 | 48,49 | 300x400 | 260.58 / 167.79---220.14 / 141.02 | 2412.74/1357.17--2010.62/1130.97 | 169.60 |
| 7 | 46,47 | 300x400 | 278.21 / 192.64---242.86 / 155.95 | 2513.27/1570.8--2199.11/1256.64 | 177.74 |
| 6 | 44,45 | 300x400 | 298.91 / 207.05---266.59 / 181.00 | 2814.87/1696.46--2412.74/1470.26 | 189.18 |
| 5 | 42,43 | 300x400 | 320.20 / 228.4---289.82 / 228.40 | 3015.93/1884.96--2613.8/1884.96 | 206.35 |
| 4 | 40,41 | 300x400 | 334.39 / 242.44---311.68 / 219.9 | 3015.93/2010.62--2814.87/1809.56 | 207.77 |
| 3 | 38,39 | 300x400 | 336.28 / 245.23---313.06 / 232.63 | 3015.93/2035.75--2814.87/1922.65 | 211.43 |
| 2 | 36,37 | 300x400 | 334.39 / 242.44---311.68 / 219.9 | 3015.93/2010.62--2814.87/1809.56 | 207.77 |
| 1 | 34,35 | 300x400 | 266.6 / 180.99---266.6 / 180.99 | 2412.74/1470.26--2412.74/1470.26 | 181.10 |

**Table 4.10 Design capacities of columns using SP-16(two bay frame)**

| | | | Analysis Result STAAD | | | Design Result (Manual)using SP-16 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Storey No | Column No | Size | Axial load | Mx | My | Axial load | Mx | My | %steel | int ratio |
| | | (mmxmm) | Pu kN | Mux kNm | MuykNm | Puz kN | Mux1 kNm | Muy1 kNm | %steel | int ratio |
| 11 | 31,33 | 350x450 | 156.5 | 109.94 | 3.13 | 2004.292 | 141.75 | 98.91 | 1.197 | 0.807 |
| 11 | 32 | 350x500 | 150.8 | 89.08 | 3.02 | 2161.628 | 148.75 | 94.675 | 1.077 | 0.631 |
| 10 | 28,30 | 350x450 | 457.7 | 126.29 | 9.15 | 2004.292 | 155.925 | 109.935 | 1.197 | 0.876 |
| 10 | 29 | 350x500 | 388.7 | 137.53 | 7.77 | 1966.086 | 155.75 | 99.575 | 0.718 | 0.961 |
| 9 | 25,27 | 350x450 | 774.2 | 134.28 | 15.48 | 2004.292 | 151.6725 | 108.99 | 1.197 | 0.93 |
| 9 | 26 | 350x500 | 1046.6 | 164.27 | 20.93 | 2161.628 | 164.5 | 113.225 | 1.077 | 1 |
| 8 | 22,24 | 350x500 | 1101.1 | 151.55 | 22.02 | 2161.628 | 166.25 | 100.275 | 1.077 | 0.97 |
| 8 | 23 | 350x500 | 1445.6 | 165.86 | 28.91 | 2552.714 | 192.5 | 119 | 1.795 | 0.889 |
| 7 | 19,21 | 350x500 | 1435.1 | 147.2 | 28.7 | 2357.171 | 162.75 | 100.1 | 1.436 | 0.967 |
| 7 | 20 | 450x500 | 1846.4 | 204.87 | 40.62 | 3002.636 | 209.25 | 180.9 | 1.396 | 1 |
| 6 | 16,18 | 350x500 | 1776.3 | 162.87 | 35.53 | 2748.257 | 192.5 | 112.875 | 2.154 | 0.88 |
| 6 | 17 | 450x500 | 3031.3 | 71.74 | 66.69 | 3589.513 | 135 | 117 | 2.23 | 0.61 |
| 5 | 13,15 | 350x500 | 2123.6 | 162.37 | 42.47 | 2943.8 | 175 | 105 | 2.513 | 1 |
| 5 | 14 | 450x500 | 3580.4 | 84.74 | 78.77 | 4176.206 | 150.75 | 131.625 | 3.07 | 0.67 |
| 4 | 10,12 | 350x500 | 2475.6 | 165.91 | 49.51 | 3334.885 | 175 | 117.25 | 3.231 | 1 |
| 4 | 11 | 500x500 | 4131.5 | 97.78 | 97.78 | 4792.334 | 167.5 | 167.5 | 3.27 | 0.68 |
| 3 | 7,9 | 400x550 | 2830.9 | 171.45 | 57.56 | 3606.966 | 193.6 | 132.352 | 2.376 | 0.98 |
| 3 | 8 | 500x550 | 4682.8 | 118.63 | 110.83 | 5353.704 | 204.325 | 180.95 | 3.36 | 0.71 |
| 2 | 4,6 | 500x550 | 3182.5 | 248.21 | 75.31 | 4625.971 | 338.8 | 302.5 | 2.513 | 0.649 |
| 2 | 5 | 500x600 | 5235.8 | 141.37 | 123.91 | 5828.996 | 205.2 | 165 | 3.351 | 1 |
| 1 | 1,3 | 500x600 | 3514 | 440.69 | 86.7 | 5437.755 | 478.8 | 378 | 2.932 | 0.942 |
| 1 | 2 | 550x600 | 5801.5 | 162.44 | 152.77 | 6685.719 | 264 | 239.25 | 3.62 | 0.79 |

Table 4.11 Determination of moment magnification factors at a joint (two bay)

| Joint No | Seismic Direction | Sum of Resisting Moments of Top & Bottom Columns at Joint (1) | Sum of Resisting Moments of Left & Right Beams at Joint with an Over Strength Factor of 1.35,(2) | Check for (1) ≥(2) | Moment Magnification Factor $\alpha cd$ |
|---|---|---|---|---|---|
| 34,36 | 1 | (0+141.75)= 141.75 | 1.35( 0+ 114.87)= 155.07 | Not OK | 1.093 |
| | 2 | (0+141.75)= 141.75 | 1.35( 0+ 70.05)= 94.56 | OK | 0.6671 |
| 35 | 1 | (0+148.75)=148.75 | 1.35(80.81+45.35)=170.32 | Not OK | 1.145 |
| | 2 | (0+148.75)=148.75 | 1.35(80.81+45.35)=170.32 | Not OK | 1.145 |
| 31,33 | 1 | (141.7+155.9)= 297.67 | 1.35( 0+ 196)= 264.59 | OK | 0.888 |
| | 2 | (141.7+155.9)= 297.67 | 1.35( 0+ 113.59)= 153.33 | OK | 0.515 |
| 32 | 1 | (148.7+155.7)=304.5 | 1.35(89.54+151.18)=324.97 | Not OK | 1.067 |
| | 2 | (148.7+155.7)=304.5 | 1.35(89.54+151.18)=324.97 | Not OK | 1.067 |
| 28,30 | 1 | (155.9+151.67)= 307.6 | 1.35( 0+ 231.48)= 312.49 | Not OK | 1.015 |
| | 2 | (155.9+151.67)= 307.6 | 1.35( 0+ 137.27)= 185.31 | OK | 0.602 |
| 29 | 1 | (155.75+164.5)=320.2 | 1.35(113.77+196.9)=419.42 | Not OK | 1.309 |
| | 2 | (155.75+164.5)=320.2 | 1.35(113.77+196.9)=419.42 | Not OK | 1.309 |
| 25,27 | 1 | (151.67+166.2)= 317.9 | 1.35( 0+ 260.58)= 351.78 | Not OK | 1.106 |
| | 2 | (151.67+166.2)= 317.9 | 1.35( 0+ 167.8)= 226.52 | OK | 0.712 |
| 26 | 1 | (164.5+192.5)=357 | 1.35(141.02+220.14)=487.57 | Not OK | 1.365 |
| | 2 | (164.5+192.5)=357 | 1.35(141.02+220.14)=487.57 | Not OK | 1.365 |
| 22,24 | 1 | (166.25+162.75)= 329 | 1.35( 0+ 278.21)= 375.58 | Not OK | 1.141 |
| | 2 | (166.25+162.75)= 329 | 1.35( 0+ 192.64)= 260.06 | OK | 0.79 |
| 23 | 1 | (192.5+209.2)=401.75 | 1.35(155.95+241.92)=537.11 | Not OK | 1.336 |
| | 2 | (192.5+209.2)=401.75 | 1.35(155.95+241.92)=537.11 | Not OK | 1.336 |
| 19,21 | 1 | (162.75+192.5)= 355.2 | 1.35( 0+ 298.91)= 403.53 | Not OK | 1.135 |
| | 2 | (162.75+192.5)= 355.2 | 1.35( 0+ 207.0)= 297.52 | OK | 0.786 |
| 20 | 1 | (209.25+135)=344.25 | 1.35(180.99+266.59)=604.24 | Not OK | 1.755 |
| | 2 | (209.25+135)=344.25 | 1.35(228.4+289.82)=699.59 | Not OK | 1.755 |
| 16,18 | 1 | (192.5+175)= 367.5 | 1.35( 0+ 320.2)= 432.27 | Not OK | 1.176 |
| | 2 | (192.5+175)= 367.5 | 1.35( 0+ 228.4)= 308.34 | OK | 0.839 |
| 17 | 1 | (135+150.75)=285.75 | 1.35(228.4+289.82)=699.59 | Not OK | 2.448 |
| | 2 | (135+150.75)=285.75 | 1.35(228.4+289.82)=699.59 | Not OK | 2.448 |
| 13,15 | 1 | (175+175)= 350 | 1.35( 0+ 334.39)= 451.43 | Not OK | 1.289 |
| | 2 | (175+175)= 350 | 1.35( 0+ 242.44)= 331.06 | OK | 0.935 |
| 14 | 1 | (150+167.5)=318.25 | 1.35(219.9+311.7)=717.66 | Not OK | 2.254 |
| | 2 | (150+167.5)=318.25 | 1.35(219.9+311.7)=717.66 | Not OK | 2.254 |
| 10,12 | 1 | (175+193.6)= 368.6 | 1.35( 0+ 336.3)= 453.98 | Not OK | 1.231 |
| | 2 | (175+193.6)= 368.6 | 1.35( 0+ 245.23)= 331.06 | OK | 0.898 |
| 11 | 1 | (167.5+204.32)=371.8 | 1.35(232.63+313.06)=736.68 | Not OK | 1.981 |
| | 2 | (167.5+204.32)=371.8 | 1.35(232.63+313.06)=736.68 | Not OK | 1.981 |
| 7,9 | 1 | (193.6 +338.8)= 532.4 | 1.35( 0+ 334.39)= 451.43 | OK | 0.848 |
| | 2 | (193.6 +338.8)= 532.4 | 1.35( 0+ 242.44)= 327.29 | OK | 0.614 |
| 8 | 1 | (204.32+205.2)= 409.5 | 1.35(219.9 +311.68)=717.63 | Not OK | 1.752 |
| | 2 | (204.32+205.2)= 409.5 | 1.35(219.9 +311.68)=717.63 | Not OK | 1.752 |
| 4,6 | 1 | (338.8 +478.8)= 817.6 | 1.35( 0+ 266.6)= 359.91 | OK | 0.44 |
| | 2 | (338.8 +478.8)= 817.6 | 1.35( 0+ 181)= 244.336 | OK | 0.298 |
| 5 | 1 | (205.2 + 264.0)= 469.2 | 1.35(180.99 +266.6)=604.246 | Not OK | 1.287 |
| | 2 | (205.2 + 264.0)= 469.2 | 1.35(180.99 +266.6)=604.246 | Not OK | 1.287 |

Table 4.12 Revision of column moments by magnification factors (two bay)

| Exterior Joint No | Column No | Moment Magnification Factor (Greater of the Two Seismic Directions and also it should be, greater than 1, otherwise take 1) | Column Moments | Revised Column Moments for magnification factor at respective joint | Revised Max Moment from Both joints of Column |
|---|---|---|---|---|---|
| 34,36 | - | 1.09 | - | - | - |
| | 31,33 | | 141.8 | 155.1 | 155.1 |
| 31,33 | 31,33 | 1.00 | 141.8 | 141.8 | |
| | 28,30 | | 155.9 | 155.9 | 158.3 |
| 28,30 | 28,30 | 1.02 | 155.9 | 158.3 | |
| | 25,27 | | 151.7 | 153.9 | 167.7 |
| 25,27 | 25,27 | 1.11 | 151.7 | 167.7 | |
| | 22,24 | | 166.3 | 183.9 | 189.7 |
| 22,24 | 22,24 | 1.14 | 166.3 | 189.7 | |
| | 19,21 | | 162.8 | 185.7 | 185.7 |
| 19,21 | 19,21 | 1.14 | 162.8 | 184.7 | |
| | 16,18 | | 192.5 | 218.5 | 226.4 |
| 16,18 | 16,18 | 1.18 | 192.5 | 226.4 | |
| | 13,15 | | 175.0 | 205.8 | 225.6 |
| 13,15 | 13,15 | 1.29 | 175.0 | 225.6 | |
| | 10,12 | | 175.0 | 225.6 | 225.6 |
| 10,12 | 10,12 | 1.23 | 175.0 | 215.4 | |
| | 7,9 | | 193.6 | 238.3 | 238.3 |
| 7,9 | 7,9 | 1.00 | 193.6 | 193.6 | |
| | 4,6 | | 338.8 | 338.8 | 338.8 |
| 4,6 | 4,6 | 1.00 | 338.8 | 338.8 | |
| | 1,3 | | 478.8 | 478.8 | 487.1 |

Table 4.13 Revision of column moments (two bay frame)

| Interior Joint No | Column No | Moment Magnification Factor (Greater of the Two Seismic Directions and also it should be, greater than 1, otherwise take 1) | Column Moments | Revised Column Moments for magnification factor at respective joint | Revised Max Moment from Both joints of Column |
|---|---|---|---|---|---|
| 35 | - | 1.145 | - | | |
| | 32 | | 148.75 | 170.318 | 170.318 |
| 32 | 32 | 1.067 | 148.75 | 158.746 | |
| | 29 | | 155.75 | 166.216 | 202.879 |
| 29 | 29 | 1.302 | 155.75 | 202.879 | |
| | 26 | | 164.5 | 214.227 | 224.542 |
| 26 | 26 | 1.365 | 164.5 | 224.542 | |
| | 23 | | 192.5 | 262.762 | 262.762 |
| 23 | 23 | 1.336 | 192.5 | 257.18 | |
| | 20 | | 209.25 | 279.558 | 367.233 |
| 20 | 20 | 1.755 | 209.25 | 367.233 | |
| | 17 | | 135 | 236.925 | 330.48 |
| 17 | 17 | 2.448 | 135 | 330.48 | |
| | 14 | | 150.75 | 369.03 | 369.03 |
| 14 | 14 | 2.254 | 150.75 | 339.79 | |
| | 11 | | 167.5 | 377.545 | 377.545 |
| 11 | 11 | 1.981 | 167.5 | 331.817 | |
| | 8 | | 204.325 | 404.767 | 404.767 |
| 8 | 8 | 1.752 | 204.325 | 357.977 | |
| | 5 | | 205.2 | 359.51 | 359.51 |
| 5 | 5 | 1.287 | 205.2 | 264.09 | |
| | 2 | | 264 | 339.768 | 339.768 |

**Table 4.14 Revised design capacities of columns using SP-16 (two bay frame)**

| | | Analysis Result with Revised Moments | | | | Final Design (Manual)using SP-16 | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Storey No | Column No | Size | Axial load | Mx | My | Axial load | Mx | My | %steel | int ratio |
| | | (mmxmm) | Pu kN | Mux kNm | Muy kNm | Puz kN | Mux1 kNm | Muy1 kNm | %steel | int ratio |
| 11 | 31,33 | 350x450 | 156.5 | 155.07 | 3.13 | 2199.889 | 168.6825 | 117.6525 | 1.596 | 0.95 |
| 11 | 32 | 350x500 | 150.8 | 170.318 | 3.02 | 2357.171 | 192.5 | 119 | 1.436 | 0.91 |
| 10 | 28,30 | 350x450 | 457.7 | 158.263 | 9.15 | 2199.889 | 185.6925 | 131.355 | 1.596 | 0.92 |
| 10 | 29 | 350x500 | 388.7 | 202.879 | 7.77 | 2357.171 | 210 | 130.725 | 1.436 | 1.00 |
| 9 | 25,27 | 350x450 | 774.2 | 167.749 | 15.48 | 2199.889 | 178.605 | 127.26 | 1.596 | 1.00 |
| 9 | 26 | 350x500 | 1046.6 | 224.542 | 20.93 | 2552.714 | 231 | 141.75 | 1.795 | 1.00 |
| 8 | 22,24 | 350x500 | 1101.1 | 189.691 | 22.02 | 2357.171 | 192.5 | 119 | 1.436 | 1.00 |
| 8 | 23 | 350x500 | 1445.6 | 262.762 | 28.91 | 3139.343 | 280 | 172.9 | 2.872 | 0.99 |
| 7 | 19,21 | 350x500 | 1435.1 | 185.697 | 28.7 | 2552.714 | 192.5 | 119 | 1.795 | 1.00 |
| 7 | 20 | 450x500 | 1846.4 | 367.233 | 40.62 | 4176.36 | 382.5 | 333.45 | 3.072 | 1.00 |
| 6 | 16,18 | 350x500 | 1776.3 | 226.38 | 35.53 | 3139.343 | 239.75 | 147.875 | 2.872 | 1.00 |
| 6 | 17 | 450x500 | 3031.3 | 330.48 | 66.69 | 4763.222 | 344.25 | 301.725 | 3.91 | 1.00 |
| 5 | 13,15 | 350x500 | 2123.6 | 225.575 | 42.47 | 3334.885 | 232.75 | 140.875 | 3.231 | 1.00 |
| 5 | 14 | 450x500 | 3580.4 | 369.03 | 78.77 | 5574.928 | 382.5 | 382.5 | 4.273 | 1.00 |
| 4 | 10,12 | 350x500 | 2475.6 | 225.575 | 49.51 | 3725.971 | 252 | 172.2 | 3.949 | 0.93 |
| 4 | 11 | 500x500 | 4131.5 | 377.545 | 97.78 | 5965.547 | 382.5 | 382.5 | 4.775 | 1.00 |
| 3 | 7,9 | 400x550 | 2830.9 | 238.321 | 57.56 | 3857.585 | 249.26 | 173.888 | 2.742 | 1.00 |
| 3 | 8 | 500x550 | 4682.8 | 404.767 | 110.83 | 6354.965 | 426.525 | 382.14 | 4.533 | 1.00 |
| 2 | 4,6 | 500x550 | 3182.5 | 338.8 | 75.31 | 4821.981 | 369.05 | 330 | 2.742 | 0.93 |
| 2 | 5 | 500x600 | 5235.8 | 359.51 | 123.91 | 6611.479 | 403.2 | 324 | 4.189 | 0.94 |
| 1 | 1,3 | 500x600 | 3514 | 478.8 | 86.7 | 5633.843 | 507.6 | 405 | 3.142 | 0.98 |
| 1 | 2 | 550x600 | 5801.5 | 339.768 | 152.77 | 7076.446 | 368.28 | 332.64 | 3.998 | 1.00 |

**Table 4.15 Capacity based design forces for soft storey columns, beams and design(one bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (500mm x 600mm) | 3604.5 | 487.08 / 94 | 4.71 | Section is insufficient, Revised to 600mmx 600mm, Ineraction Ratio - 1.0, |
| Beam no. 23 (350mm x 450mm) | - | Hogging-565 / Sagging- 340 | 2.2 / 1.34 | - |

**Table 4.16 Capacity based design forces for soft storey columns, beams and design (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (500mm x 600mm) | 3514 | 478.8 / 86.7 | 3.142 | Ineraction Ratio - 0.98 |
| Column 2 (550mm x 600mm) | 5801.5 | 339.768 / 152.77 | 3.99 | Ineraction Ratio -1 |
| Beam nos. 34,35 (300 x 400) (Exterior End) | - | Hogging-266.6 / Sagging- 180.99 | 2.01 / 1.23 | Doubly Reinforced Beam in Both Cases |
| Beam nos. 34,35 (300 x 400) (Interior End) | - | Hogging-266.6 / Sagging- 180.99 | 2.01 / 1.23 | Doubly Reinforced Beam in Both Cases |

### 4.2.3 Energy Based Design

To apply the Energy Based Design to a Soft Storey Building, the (G+10) storeyed building Plane frames one with single-bay and the other with double-bay with an open ground storey satisfying the criteria of soft storey is considered. In this method the soft storey design is done by increasing the lateral forces of the resisting elements of soft storey by a multiplication factor which is obtained by equating the plastic energies of the structure. The concept of Energy based design is discussed in chapter-2.

In the application of this method, for the frames considered to calculate the multiplying factor, it requires two auxiliary structures that is one with extreme soft storey, which is fully open at the ground floor with infills at the upper floors and the other is a uniform structure, that is obtained by keeping the stiffness of the other storey columns equal to the ground storey columns so that there is no soft storey effect. The size of the columns of the ground storey is adopted from the results of the preliminary dynamic analysis that is carried out for each frame in chapter-3.The lateral loads applied statically to the frame members are derived according to the equivalent static force method from *IS 1893 (Part 1):2002*.. The two frames i.e. single bay and double bay G+10 storeyed plane frames are modeled using STAAD Pro. Static analysis of the structure is carried out for the lateral loads using the STAAD pro 2001 software package The storey deflections and the lateral loads applied on the structure were given in Tables 4.17 and 4.18 for one bay and Two bay frames respectively.

After getting the multiplying factor of each frame by equating the plastic energies of the two auxiliary structures of each case, the forces coming according to the preliminary analysis is factored using the multiplying factor of that case. Then the elements of the soft storey are designed for those forces. The design forces of soft storey elements are given in Tables 4.19 & 4.20 for one bay and two bay frames respectively.

Table 4.17 Lateral load distribution and lateral storey displacements of one bay frame

| | | Plane Frame With out | | | Plane Frame With | | |
|---|---|---|---|---|---|---|---|
| Storey No | Column Size | Lateral Load | Lateral displacement | Load times Displacement | Lateral Load | Lateral displacement | |
| | mm x mm | kN | m | | kN | m | |
| 11 | 550 x 600 | 22.966 | 0.0284 | 0.705990215 | 22.966 | 0.0079 | 0.000695 |
| 10 | 550 x 600 | 27.258 | 0.0259 | 0.652229845 | 27.258 | 0.0073 | 0.003163 |
| 9 | 550 x 600 | 22.148 | 0.0232 | 0.513842655 | 22.148 | 0.0067 | 0.008375 |
| 8 | 550 x 600 | 17.568 | 0.0203 | 0.356638291 | 17.568 | 0.0062 | 0.017281 |
| 7 | 550 x 600 | 13.518 | 0.0173 | 0.23386667 | 13.518 | 0.0056 | 0.030835 |
| 6 | 550 x 600 | 9.998 | 0.0143 | 0.142973375 | 9.998 | 0.005 | 0.049991 |
| 5 | 550 x 600 | 7.008 | 0.0114 | 0.079889936 | 7.008 | 0.0044 | 0.075703 |
| 4 | 550 x 600 | 4.548 | 0.0086 | 0.039108996 | 4.548 | 0.0038 | 0.108924 |
| 3 | 550 x 600 | 2.617 | 0.0061 | 0.015964577 | 2.617 | 0.0032 | 0.148394 |
| 2 | 550 x 600 | 1.217 | 0.0038 | 0.00462326 | 1.217 | 0.0026 | 0.198986 |
| 1 | 550 x 600 | 0.347 | 0.002 | 0.000694782 | 0.347 | 0.002 | 0.18143 |
| | | | | **2.745822603** | | | **0.823776** |

Therefore the multiplying factor is obtained as $c_0 = \left[\frac{\sum_u Fu}{\sum_s Fu}\right]^{1/2} = \left[\frac{2.7458}{0.8237}\right]^{1/2} = 1.825$

Table 4.18 Lateral load distribution and lateral storey displacements of two bay frame

| | | Plane Frame With out Infillwalls | | | Plane Frame With Infillwalls | | |
|---|---|---|---|---|---|---|---|
| Storey No | Column Size-ext/int | Lateral Load | Lateral displacement | Load x Displacement | Lateral Load | Lateral displacement | Load x Displacement |
| | mm x mm | kN | m | | kN | m | |
| 11 | 500 x 600/ 550x600 | 40.0351 | 0.0206 | 0.8247 | 40.0351 | 0.0051 | 0.204 |
| 10 | 500 x 600/ 550x600 | 45.7784 | 0.0193 | 0.8835 | 45.7784 | 0.0048 | 0.220 |
| 9 | 500 x 600/ 550x600 | 37.1966 | 0.0177 | 0.6584 | 37.1966 | 0.0046 | 0.171 |
| 8 | 500 x 600/ 550x600 | 29.5049 | 0.0158 | 0.4662 | 29.5049 | 0.0043 | 0.127 |
| 7 | 500 x 600/ 550x600 | 22.7030 | 0.0139 | 0.3156 | 22.7030 | 0.004 | 0.091 |
| 6 | 500 x 600/ 550x600 | 16.7912 | 0.0118 | 0.1981 | 16.7912 | 0.0037 | 0.062 |
| 5 | 500 x 600/ 550x600 | 11.7693 | 0.0097 | 0.1142 | 11.7693 | 0.0034 | 0.040 |
| 4 | 500 x 600/ 550x600 | 7.6373 | 0.0077 | 0.0588 | 7.6373 | 0.003 | 0.023 |
| 3 | 500 x 600/ 550x600 | 4.3953 | 0.0057 | 0.0251 | 4.3953 | 0.0027 | 0.012 |
| 2 | 500 x 600/ 550x600 | 2.0433 | 0.0038 | 0.0078 | 2.0433 | 0.0024 | 0.005 |
| 1 | 500 x 600/ 550x600 | 0.6037 | 0.0021 | 0.0013 | 0.6037 | 0.0021 | 0.001 |
| | | | | **3.5536** | | | **0.9558** |

Therefore the multiplying factor for two bay frame resisting elements is obtained as

$$c_0 = \left[\frac{\sum_u Fu}{\sum_s Fu}\right]^{1/2} = \left[\frac{3.5535}{0.9558}\right]^{1/2} = 1.928$$

Table 4.19 Energy based design forces for soft storey columns, beams and design (one bay)

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (550mm x 600mm) | 3604.5 | 720.71 / 171.55 | 3.808 | Ineraction Ratio - 1.0, |
| Beam no. 23 (350mm x 450mm) | - | Hogging-412.45 / Sagging- 248.20 | 2.07 / 1.28 | - |

Table 4.20 Energy based design forces for soft storey columns, beams and design (two bay)

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (500mm x 600mm) | 3514 | 849.65 / 167.15 | 4.379 | Ineraction Ratio - 1 |
| Column 2 (550mm x 600mm) | 5801.5 | 313.18 / 294.54 | 4.8 | Ineraction Ratio -1 |
| Beam nos. 34,35 (300 x 400) (Exterior End) | - | Hogging-509.396 / Sagging- 249.483 | 2.51 / 1.7 | Doubly Beam Reinforced in Both Cases |
| Beam nos. 34,35 (300 x 400) (Interior End) | - | Hogging-497.23 / Sagging- 247.55 | 2.46 / 1.6 | Doubly Reinforced Beam in Both Cases |

### 4.2.4 Equal Displacement Based Design

#### 4.2.4.1 Equal displacement based design (one bay frame)

To apply the displacement based design concept for the soft storey problem, the (G+10) storeyed single bay building plane frame with open ground is considered, assuming that the ground storey columns will provide lateral support for the tributary building frame weight of 428.185ton.

*Step 1: Establish the objective Drift limit*

A drift objective of two percent has been adopted, considering the storey drift limitation of 0.004 times the storey height in any storey due to the minimum specified design lateral force with partial load factor of 1.0 *(Clause 7.11.1and 7.11.2, IS 1893 (Part 1):2002)* taking a ductility of 5 (R = 5).

i.e., $(0.004h)\times R = (0.004h)\times 5 = 2\%$

Therefore the objective level of drift $\Delta_u = 0.02\times 4 = 0.08$ m

*Step 2: Determine the objective natural frequency $\omega_n$ based on a criteria spectral velocity,* $S_v$ :

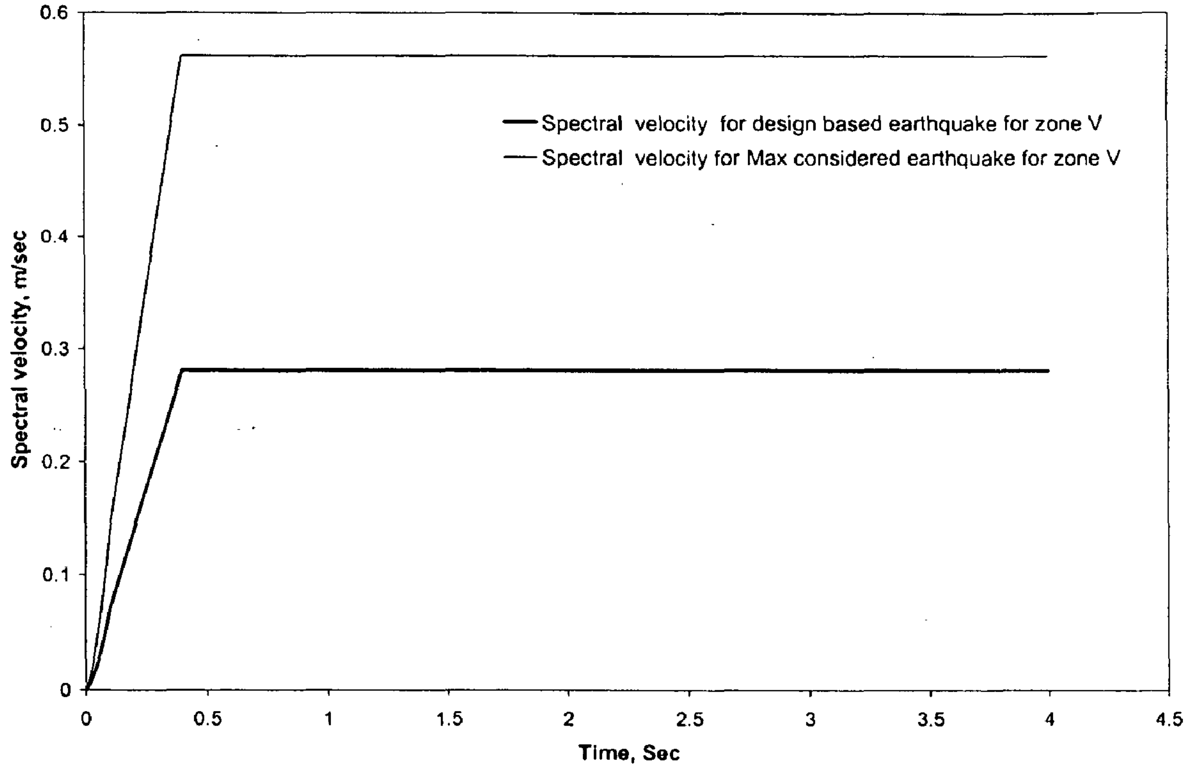



**Fig. 4.1 Spectral velocity from IS 1893 (Part 1) 2002 for rock or hard soil**

Objective natural frequency, $\omega_n = \dfrac{S_v}{\Delta_u} = \dfrac{.281}{0.08} = 3.513 \, {}^{rad}\!/_{\sec}$

*Step 3: Determine the objective stiffness*

$$K = \omega_n^{\ 2} m = 3.513^2 \times 428.185 = 5284.3 \, {}^{kN}\!/_{m}$$

*Step 4: Size the frame components*

From the assumption of fixed base, the lateral stiffness of frame will be calculated .as follows

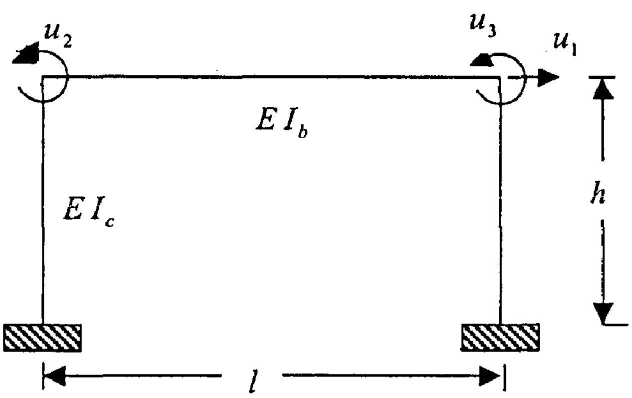



**Fig. 4.2 Lateral stiffness of frame system**

$$K = \begin{bmatrix} 2\left[\frac{12EI_c}{h^3}\right] & \frac{6EI_c}{h^2} & \frac{6EI_c}{h^2} \\ \frac{6EI_c}{h^2} & \left[\frac{4EI_c}{h}+\frac{4EI_b}{l}\right] & \frac{2EI_b}{l} \\ \frac{6EI_c}{h^2} & \frac{2EI_b}{l} & \left[\frac{4EI_c}{h}+\frac{4EI_b}{l}\right] \end{bmatrix}$$

The bay length "$l$" is 5m and storey height "$h$" is 4m

Therefore $\frac{l}{h} = \frac{5}{4} = 1.25h$

Similarly taking the ratio of $\frac{I_b}{I_c} = \frac{1}{1.35}$

Substituting $l = 1.25\ h$ and $I_b = 0.74 I_c$

We get

$$K = \begin{bmatrix} \frac{24EI_c}{h^3} & \frac{6EI_c}{h^2} & \frac{6EI_c}{h^2} \\ \frac{6EI_c}{h^2} & \frac{6.368EI_c}{h} & \frac{1.184EI_c}{h} \\ \frac{6EI_c}{h^2} & \frac{1.184EI_c}{h} & \frac{6.368EI_c}{h} \end{bmatrix}$$

From the equation $f_s = KU$

Where $f_s = Lateralforce$, $K = Lateralstiffness$,

$U = \mathrm{Re}\,lativedisplacement\ associated\ with\ deformations$

$$\begin{bmatrix} \frac{24EI_c}{h^3} & \frac{6EI_c}{h^2} & \frac{6EI_c}{h^2} \\ \frac{6EI_c}{h^2} & \frac{6.368EI_c}{h} & \frac{1.184EI_c}{h} \\ \frac{6EI_c}{h^2} & \frac{1.184EI_c}{h} & \frac{6.368EI_c}{h} \end{bmatrix} \begin{bmatrix} u_1 \\ u_2 \\ u_3 \end{bmatrix} = \begin{bmatrix} f_s \\ 0 \\ 0 \end{bmatrix}$$

from the second and third equations, the joint rotations can be expressed in terms of lateral displacements as follows.

Solving 1 and 2 we get $u_2 = u_3$

Substituting $u_2 = u_3$ in the 2nd equation we get $u_2 = u_3 = -\frac{0.7944u_1}{h}$

Substituting $u_2 = u_3 = -\frac{0.7944u_1}{h}$ in equation 1,

We get

$\frac{14.4672\ EI_c}{h^3}u_1 = f_s$ therefore $\frac{14.4672\ EI_c}{h^3} = \frac{f_s}{u_1} = k$

Lateral stiffness of the frame is $k = \frac{14.4672\ EI_c}{h^3}$

$k = \dfrac{14.47EI_c}{h^3}$, assuming the mass acting as a rigid body above the ground floor columns representing a lumped mass single degree of freedom system.

Taking width of column as 0.6 m

Therefore, $14.47EI_c/h^3 \quad \Rightarrow 5284.3 = \dfrac{14.47 \times 22360 \times 10^3 \times 0.3 \times d^3}{4^3 \times 12}$

$$\Rightarrow d = 0.347m$$

*Step 5: Determine the strength required of the column*

It is assumed that the system ductility factor $(\mu)$or the ratio of idealized yield displacement $(\Delta y_i)$ to ultimate displacement $(\Delta_u)$ is 5.

Accordingly, this will produce an estimate of the ultimate strength required of the system ($F_{max}$). The idealized elastic frame displacement is obtained by, $\dfrac{\Delta_u}{5}$ then required ultimate strength, expressed as a mechanism shear force imposed on the frame, is $F_{max} = \Delta_{yi} \times K = 0.016 \times 5284.304 = 85.54kN$

The objective level of strength $\quad \Omega F_0 \;=\; F_{max}$

$= 85.54$ kN

Therefore $F_0 = F_{Max}/\Omega \;=\; 85.54/1.25 \;=\; 67.68$ kN

We have assumed that the system is elastic/perfectly plastic for design purposes. The required nominal strength of the column,

$$M_c = \frac{F_0}{2}h = \frac{67.68}{2} \times 4 \qquad = 135.27 \text{ kNm}$$

$$M_b = \frac{M_c}{1.35} = \frac{135.27}{1.35} = 100.20\,kNm$$

### 4.2.4.2 Equal displacement based design (two bay frame )

Considering the (G+10) storey with open ground storey plane frame assuming that the ground storey columns will provide lateral support for the tributary building frame weight of 428.185ton.

*Step 1: Establish the objective Drift limit*

A drift objective of two percent has been adopted, considering the storey drift limitation of 0.004 times the storey height in any storey due to the minimum specified design lateral force with partial load factor of 1.0 (Clause 7.11.1 and 7.11.2, IS 1893 (Part 1):2002) taking ductility of 5 (R = 5).

i.e., $(0.004h)\times R = (0.004h)\times 5 = 2\%$

Therefore the objective level of drift $\Delta_u = 0.02\times 4 = 0.08$ m

*Step 2: Determine the objective natural frequency $\omega_n$ based on a criteria spectral velocity,* $S_v$:

Objective natural frequency, $\omega_n = \dfrac{S_v}{\Delta_u} = \dfrac{0.281}{0.08} = 3.513\, {rad}/{sec}$

*Step 3: Determine the objective stiffness*

$K = \omega_n{}^2 m = 3.513^2 \times 725.372 = 8952.57\, {kN}/{m}$

*Step 4: Size the frame components*

From the assumption of fixed base the lateral stiffness of frame will be taken as $k = \dfrac{27.560EI_c}{h^3}$, assuming the mass acting as a rigid body above the ground floor columns representing a lumped mass single degree of freedom system.

Taking width of column as 0.6 m

Therefore, $27.560EI_c/h^3 \quad \Rightarrow 8952.57 = \dfrac{27.560\times 22360\times 10^3\times 0.3\times d^3}{4^3\times 12}$

$$\Rightarrow d = 0.33m$$

*Step 5: Determine the strength required of the column*

For this example it is assumed that the system ductility factor $(\mu)$ or the ratio of idealized yield displacement $(\Delta y_i)$ to ultimate displacement $(\Delta_u)$ is 5.

Accordingly, this will produce an estimate of the ultimate strength required of the system ($F_{max}$). The idealized elastic frame displacement is $\Delta y_i = \dfrac{\Delta_u}{\mu} = \dfrac{0.08}{5} = 0.016m$ then required ultimate strength, expressed as a mechanism shear force imposed on the frame, is

$F_{max} = \Delta_{yi}\times K = 0.016\times 8952.57 = 143.24kN$

The objective level of strength $\Omega F_0$ = $F_{max}$

= 143.24 kN

Therefore $F_0 = F_{Max}/\Omega$ = 143.24/1.25 = 1.14.59 kN

We have assumed that the system is elastic/perfectly plastic for design purposes. The required nominal strength of the Exterior column,

$$M_c = \frac{F_0}{3.728}h = \frac{114.59}{3.728} \times 4 \quad = 122.95 \text{ kN-m}$$

The required nominal strength of the Interior column,

$$M_c = (\frac{F_0}{3.728}h)1.728 = (\frac{114.59}{3.728} \times 4)1.728 \quad = 212.45 \text{ kN-m}$$

### 4.2.5 Direct Displacement Based Design

#### 4.2.5.1 Direct displacement based design (one bay frame)

*Step 1: Establish the Objective Drift Limit.*

If we assume Drift Objective of 2%, then the Objective Level of Drift is

$$\Delta_u = 0.02 \times h_X = 0.02 \times 4 = 0.08m$$

*Step 2: Revise the Design Spectral Velocity to Reflect the Level of Provided Structural Damping*

$$\zeta_{eq} = \frac{\sqrt{\mu}-1}{\pi\sqrt{\mu}} = \frac{\sqrt{5}-1}{\pi\sqrt{5}} = 17.6\,\%$$

$$\dot{d}_{\max} = \frac{S_v}{3.38-0.67\ln 5} = \frac{0.281}{3.38-0.67\ln 5} = 0.122\,m/\sec$$

$$\hat{\zeta}_{eq} = \zeta + \zeta_{eq} = 5 + 17.6 = 22.6$$

$$S_v = (3.38-0.67\ln\hat{\zeta}_{eq})\dot{d}_{\max} = (3.38-0.67\ln 22.6) \times 0.122 = 0.159\,m/\sec$$

*Step 3: Determine the Objective Natural Frequency $\omega_{n,ductile}$ of the Ductile Structure Based on a Criterion of Spectral Velocity*

$$\omega_n = \frac{S_v}{\Delta_u}$$

$$\omega_n = \frac{0.159}{0.080} = 1.99\,rad/\sec$$

*Step 4: Determine the Objective Natural Frequency $\omega_{n,elastic}$ of the Elastic Structure*

$$\omega_{n,elastic} = \omega_{n,ductile}\sqrt{\mu} = 1.99\sqrt{5} = 4.4636\,rad/\sec$$

*Step 5: Determine the Stiffness Required of the Elastic Structure.*

$$K = \omega_n^{\,2} m = 4.4636^2 \times 428.185 = 8531.422\ kN/m$$

*Step 6: Proceed to Develop the Stiffness required of the components:*

The relation between force and displacement for the fixed base frame described in fig. is $k = \frac{14.47 EI_c}{h^3}$

Keeping the width of column constant we can obtain the depth of the column

$$\Rightarrow 8531.422 = \frac{14.47 \times 22360 \times 10^3}{4^3}\left[\frac{0.3 \times d^3}{12}\right] \qquad \Rightarrow d = 0.407\ m$$

*Step 7: Determine the Strength Required of the Column:*

Taking the assumed system ductility factor (μ) or the ratio of idealized yield displacement ($\Delta_{yi}$) to ultimate displacement ($\Delta_u$). Accordingly, this will produce an estimate of the ultimate strength required of the system ($F_{max}$). The idealized elastic frame displacement is

$$\Delta_{yi} = \frac{\Delta_u}{\mu} = \frac{0.08}{5} = 0.016m$$

then required ultimate strength, expressed as a mechanism shear force imposed on the frame, is $F_{max} = \Delta_{yi} \times K$

$$F_{max} = 0.016 \times 8531.422 = 136.502\,kN$$

Therefore the objective level of strength,

$$\Omega F_0 = F_{max}$$

$$= 136.502 \text{ kN}$$

Therefore $F_0 = F_{Max}/\Omega = 136.502/1.25 = 109.22$ kN

Moment,

$$M_c = \frac{F_0}{2} h = \frac{109.22}{2} \times 4 = 218.404 \text{ kN-m}$$

$$M_b = \frac{M_c}{1.35} = \frac{218.404}{1.35} = 161.781\,kNm$$

#### 4.2.5.2 Direct displacement based design(two bay frame)

*Step 1: Establish the Objective Drift Limit.*

If we assume Drift Objective of 2%, then the Objective Level of Drift is

$$\Delta_u = 0.02 \times h_X = 0.02 \times 4 = 0.08m$$

*Step 2: Revise the Design Spectral Velocity to Reflect the Level of Provided Structural Damping*

$$\zeta_{eq} = \frac{\sqrt{\mu}-1}{\pi\sqrt{\mu}} = \frac{\sqrt{5}-1}{\pi\sqrt{5}} = 17.6\,\%$$

$$\dot{d}_{\max} = \frac{S_v}{3.38-0.67\ln 5} = \frac{0.281}{3.38-0.67\ln 5} = 0.122\,m/\sec$$

$$\hat{\zeta}_{eq} = \zeta + \zeta_{eq} = 5 + 17.6 = 22.6$$

$$S_v = (3.38-0.67\ln\hat{\zeta}_{eq})\dot{d}_{\max} = (3.38-0.67\ln 22.6)\times 0.122 = 0.1597\,m/\sec$$

*Step 3: Determine the Objective Natural Frequency $\omega_{n,ductile}$ of the Ductile Structure Based on a Criterion of Spectral Velocity*

$$\omega_n = \frac{S_v}{\Delta_u}$$

$$\omega_n = \frac{0.1597}{0.080} = 1.99\,rad/\sec$$

*Step 4: Determine the Objective Natural Frequency $\omega_{n,elastic}$ of the Elastic Structure*

$$\omega_{n,elastic} = \omega_{n,ductile}\sqrt{\mu} = 1.99\sqrt{5} = 4.4636\,rad/\sec$$

*Step 5: Determine the Stiffness Required of the Elastic Structure.*

$$K = \omega_n^{\,2} m = 4.4636^2 \times 725.372 = 14452.36\,kN/m$$

*Step 6: Proceed to Develop the Stiffness required of the components*:

The relation between force and displacement for the fixed base frame described in fig. is $k = \dfrac{27.560EI_c}{h^3}$

Keeping the width of column constant we can obtain the depth of the column

$$\Rightarrow 14452.36 = \frac{27.560\times 22360\times 10^3}{4^3}\left[\frac{0.3\times d^3}{12}\right] \qquad \Rightarrow d = 0.391\ m$$

*Step 7: Determine the Strength Required of the Column:*

Taking the assumed system ductility factor ($\mu$) or the ratio of idealized yield displacement ($\Delta_{yi}$) to ultimate displacement ($\Delta_u$). Accordingly, this will produce an estimate of the ultimate strength required of the system ($F_{max}$). The idealized elastic frame displacement is

$$\Delta_{yi} = \frac{\Delta_u}{\mu} = \frac{0.08}{5} = 0.016m$$

then the required ultimate strength, expressed as a mechanism shear force imposed on the frame, is $F_{max} = \Delta_{yi} \times K$

$F_{max} = 0.016 \times 14452.36 = 231.237\, kN$

Therefore the objective level of strength,

$\Omega F_0 =$ $F_{max}$

$=$ 231.237 kN

Therefore $F_0 = F_{Max} / \Omega$ = 231.237/1.25 = 184.99 kN

Moment of exterior column

$$M_c = \frac{F_0}{3.728} h = \frac{184.99}{3.728} \times 4 = 198.487 \text{ kN-m}$$

$$M_b = \frac{M_c}{1.35} = \frac{198.487}{1.35} = 147.027\, kNm$$

Moment of interior column

$$M_c = \left(\frac{F_0}{3.728}\right) 1.728 h = \left(\frac{184.99}{3.728}\right) 1.728 \times 4 = 342.984 \text{ kN-m}$$

# Chapter 5

# RESULTS AND DISCUSSION

## 5.1 GENERAL

Five types of design Methods for a soft storey Plane Frame is carried out in order to obtain the design forces of the elements of the soft storey. Those methods are as follows

a. *IS 1893(Part 1):2002* Code criteria
b. Capacity Based Design
c. Energy Based Design
d. Equal Displacement Based Design
e. Direct Displacement Based Design

The design results obtained are as follows

### 5.1.1 *IS 1893(Part 1):2002* Code Criteria Design Results

G+10 Storeyed open ground storey plane frame satisfying the criteria of soft storey structure was initially analyzed for lateral Earthquake Forces by Response Spectrum Method of analysis. Then the Analysis Results were factored by 2.5 times and then the soft storey columns and beams were designed for the factored forces. The lateral loads taken in the analysis were revised according to *IS 1893(Part 1):2002*, as the time period of the STAAD model is high resulting in lesser forces, Hence Response Revision factor is applied to the Design Seismic coefficient in STAAD. Then the forces are equal to the lateral loads according to *IS 1893(Part 1):2002*. Two soft storey Plane frames of (G+10) storey one with single bay and the other with double bay are taken for analysis. The design forces of soft storey elements are presented in Tables 5.1 and 5.2 for one-bay and two bay frames respectively.

**Table 5.1 IS 1893(Part 1):2002 Design forces for soft storey columns and beams and design (one bay)**

| Element of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No. 1,2 (500mm x 600mm) | 3604.5 | 987.275 / 237.3 | 4.71 | Section is insufficient, Revised to 600mmx 600mm, Ineraction Ratio - 1.0, |
| Beam no. 23 (350mm x 450mm) | - | Hogging-565 / Sagging- 340 | 2.2 / 1.34 | Section is insufficient, Revised to 350mm x 500mm |

**Table 5.2 IS 1893 (Part 1):2002 Design forces for soft storey columns, beams and design (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (500mm x 600mm) | 3514 | 1101.72 / 216.75 | 4.712 | Section is insufficient, Revised to 600mmx 600mm, Ineraction Ratio - 1.0, |
| Column 2 (550mm x 600mm) | 5801.5 | 406.10 / 381.92 | 5 | Ineraction Ratio -0.92 |
| Beam nos. 34,35 (300mm x 400mm) (Exterior End) | - | Hogging-660.5 / Sagging- 323.5 | 2.15 / 1.29 | Section is insufficient, Revised to 350mm x 550mm |
| Beam nos. 34,35 (300mm x 400mm)) (Interior End) | - | Hogging- 644.75 / Sagging- 321.0 | 2.10 / 1.24 | Section is insufficient, Revised to 350mm x 550mm |

### 5.1.2 Capacity Based Design Results

The single bay and two bay plane frames that were taken to analysis and designed according to IS 1893:2002 were taken to apply capacity based design concept for the soft storey design solution with a view of strong column and weak beam proportioning. Dynamic Analysis has been carried out taking into account only the mass of infill walls and neglecting its stiffness. The analysis and design is performed through various no of iterations with a view of getting column interaction ratio near around 0.9 to 1.so that the capacity of the column will be exact to the analysis The iterations were performed until all the columns interaction ratio with in the range. Then the capacity revision for column moments were obtained from the moment magnification factor Then the column moments were increased according to the moment magnification factor and then designed the columns. The design forces according to the capacity based design are as shown in the following table 5.3 for one bay frame and in Table 5.4 for two bay frame.

**Table 5.3 Capacity based design forces for soft storey columns, beams and design (one bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (500mm x 600mm) | 3604.5 | 487.08 / 94 | 4.71 | Section is insufficient, Revised to 600mmx 600mm, Ineraction Ratio - 1.0, |
| Beam no. 23 (350mm x 450mm) | - | Hogging-565 / Sagging- 340 | 2.2 / 1.34 | - |

**Table 5.4 Capacity based design forces for soft storey columns, beams and design (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (500mm x 600mm) | 3514 | 478.8 / 86.7 | 3.142 | Ineraction Ratio - 0.98 |
| Column 2 (550mm x 600mm) | 5801.5 | 339.768 / 152.77 | 3.99 | Ineraction Ratio -1 |
| Beam nos. 34,35 (300 x 400) (Exterior End) | - | Hogging-266.6 / Sagging- 180.99 | 2.01 / 1.23 | Doubly Reinforced Beam in Both Cases |
| Beam nos. 34,35 (300 x 400) (Interior End) | - | Hogging-266.6 / Sagging- 180.99 | 2.01 / 1.23 | Doubly Reinforced Beam in Both Cases |

### 5.1.3 Energy Based Design Results

In this method of design the soft storey plane frame structures were initially analyzed and designed for earthquake forces according to IS 1893:2002. with gravity load combinations specified in IS 1893:2002.. Then for every plane frame we have taken a uniform structure with the stiffness of columns of all stories same as the ground storey columns so that there is no soft storey formation in this case and in the second case the same structure is provided with infill walls in the upper floors. The lateral displacements were then obtained for lateral loads acting at various floor levels for both cases the plane frame with uniform stiffness and the in filled frame with open ground storey. Then the multiplying factor for design forces is calculated by equating the plastic energies of the two structures. The design forces obtained from this method are given in the following tables.5.5, 5.6

**Table 5.5 Energy based design forces for soft storey columns, beams and design(one bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (550mm x 600mm) | 3604.5 | 720.71 / 171.55 | 3.808 | Ineraction Ratio - 1.0, |
| Beam no. 23 (350mm x 450mm) | - | Hogging-412.45 / Sagging- 248.20 | 2.07 / 1.28 | - |

**Table 5.6 Energy based design forces for soft storey columns, beams and design (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design biaxial bending Moments Mx / My (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (500mm x 600mm) | 3514 | 849.65 / 167.15 | 4.379 | Ineraction Ratio - 1 |
| Column 2 (550mm x 600mm) | 5801.5 | 313.18 / 294.54 | 4.8 | Ineraction Ratio -1 |
| Beam nos. 34,35 (300 x 400) (Exterior End) | - | Hogging-509.396 / Sagging- 249.483 | 2.51 / 1.7 | Doubly Beam Reinforced in Both Cases |
| Beam nos. 34,35 (300 x 400) (Interior End) | - | Hogging-497.23 / Sagging- 247.55 | 2.46 / 1.6 | Doubly Reinforced Beam in Both Cases |

### 5.1.4 Displacement Based Design Results

In this method the design for a soft storey solution is very simple but its concept is different from other methods of design. The design forces were obtained with an assumed objective drift limit, and the peak spectral velocity of an earthquake. The peak spectral velocity adopted is derived from the design spectra specified in IS 1893:2002. From the spectral velocity and the objective drift the frequency of the system will be calculated and from the frequency and mass, the stiffness required by the structure is obtained. From the stiffness and the assumed idealized elastic displacement at the level of storey columns the forces acting on the column is determined. Two displacement based design concepts have been applied for the soft storey solution of the G+10 storeyed plane frame. The design forces are given in the table 5.7 for one bay and Table 5.8 for two bay frames by Equal Displacement Based approach respectively.

Similarly for the Direct Displacement based design forces are given in Table 5.9 and 5.10 for one bay and two bay frames respectively.

**Table 5.7 Equal displacement based design forces for soft storey columns, beams and design details(one bay)**

| Elements of Soft storey | Design Axial load (kN) | Design bending Moments Mx (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (600mm x 865mm) | 3604.5 | 417.32 | 0.8 | - |
| Beam no. 23 (600mm x 780mm) | - | 309.12 | | - |

**Table 5.8 Equal displacement based design forces for soft storey columns, beams and design details (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design bending Moments Mx (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (600mm x 830mm) | 3577.19 | 379.279 | 0.8 | - |
| Column 2 (600 x 1000) | 5801.5 | 655.394 | 1 | - |
| Beam nos. 34,35 (600 x 750) | - | 280.94 | - | - |
| | | | | |

**Table 5.9 Direct displacement based design forces for soft storey columns, beams and design details (one bay)**

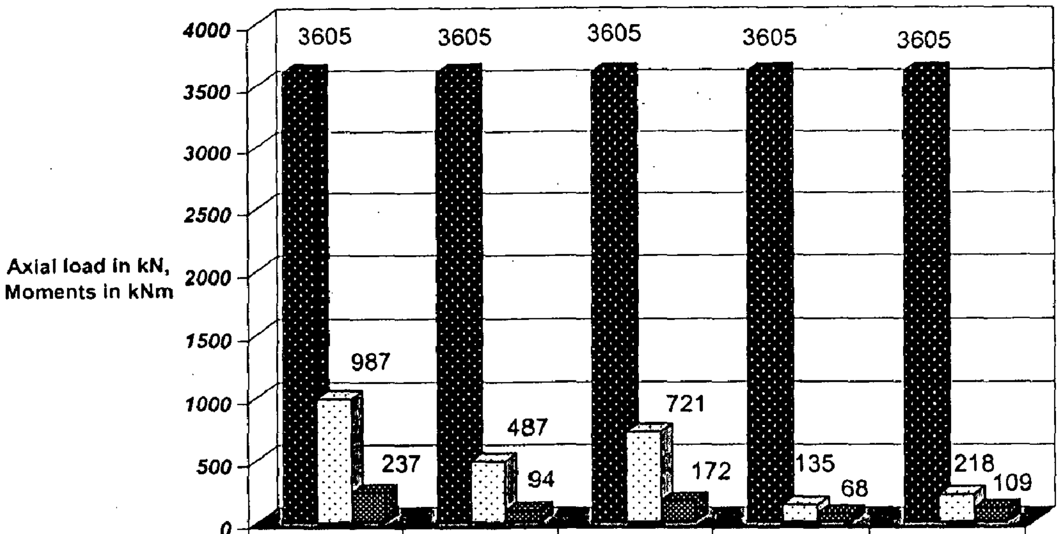
| Elements of Soft storey | Design Axial load (kN) | Design bending Moments Mx (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Column No.1,2 (600mm x1000mm) | 3604.5 | 655.54 | 0.8 | - |
| Beam no. 23 (600mm x905mm) | - | 485.585 | - | - |

**Table 5.10 Direct displacement based design forces for soft storey columns, beams and design details (two bay)**

| Elements of Soft storey | Design Axial load (kN) | Design bending Moments Mx (kNm) | Provided Steel (%) | Remarks |
|---|---|---|---|---|
| Columns 1,3 (600mm x 965mm) | 3577.19 | 595.785 | 0.8 | - |
| Column 2 (600mmx1160mm) | 5801.5 | 1029.517 | 0.8 | - |
| Beam nos. 34,35 (600 x 870) | - | 441.322 | - | - |

## 5.2 Comparison of Design Forces of the Design Methods

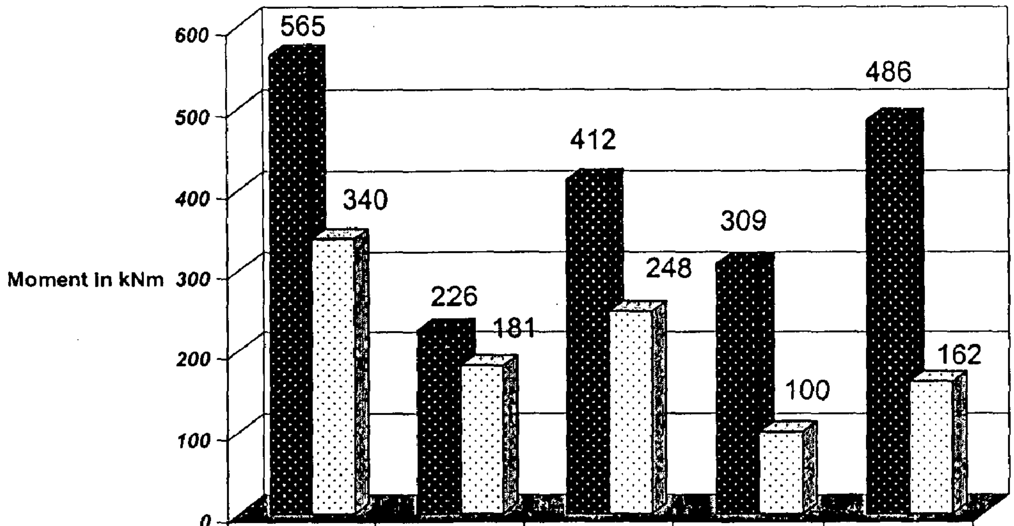
The deign forces and moments for columns and beams are obtained from all five methods are plotted and shown in Figs. 5.1 to 5.6.




| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ■ *Axial load on Column 1,2* | *3605* | *3605* | *3605* | *3605* | *3605* |
| □ *Moment Mx on column 1,2* | *987* | *487* | *721* | *135* | *218* |
| ▩ *Moment My on column 1,2* | *237* | *94* | *172* | *68* | *109* |

**(1) IS 1893(Part 1):2002 Code (2) Capacity Based Design**
**(3) Energy Based Design (4) Equal Displacement Based Design**
**(5) Direct Displacement Based Design**

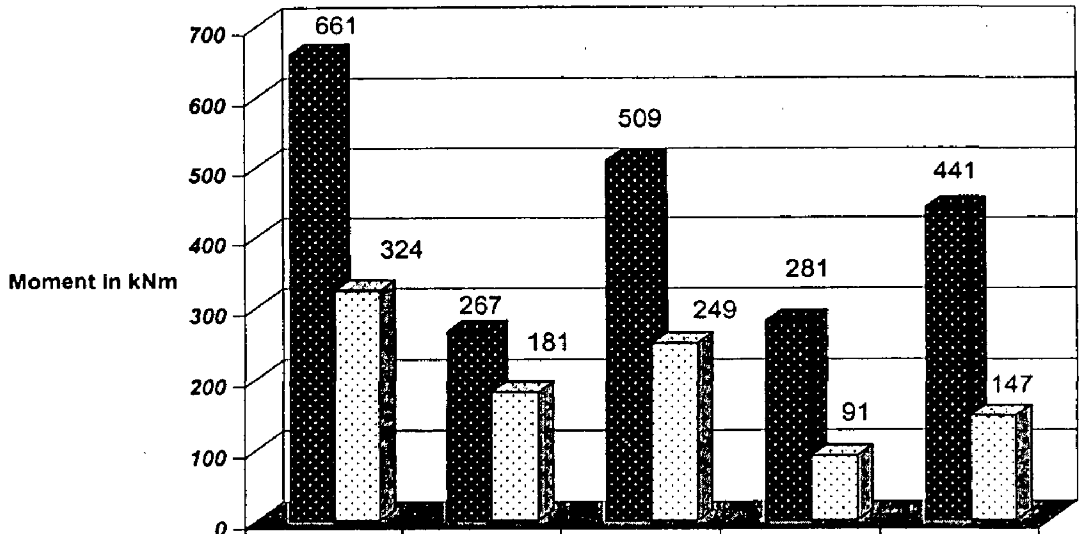
**Fig. 5.1 Comparison of column design forces and moments (one bay)**




| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ■ *Hogging Moment at End of Beam 23* | *565* | *226* | *412* | *309* | *486* |
| □ *Sagging Moment at end of Beam 23* | *340* | *181* | *248* | *100* | *162* |

**(1) IS 1893(Part 1):2002 Code (2) Capacity Based Design**
**(3) Energy Based Design (4) Equal Displacement Based Design**
**(5) Direct Displacement Based Design**

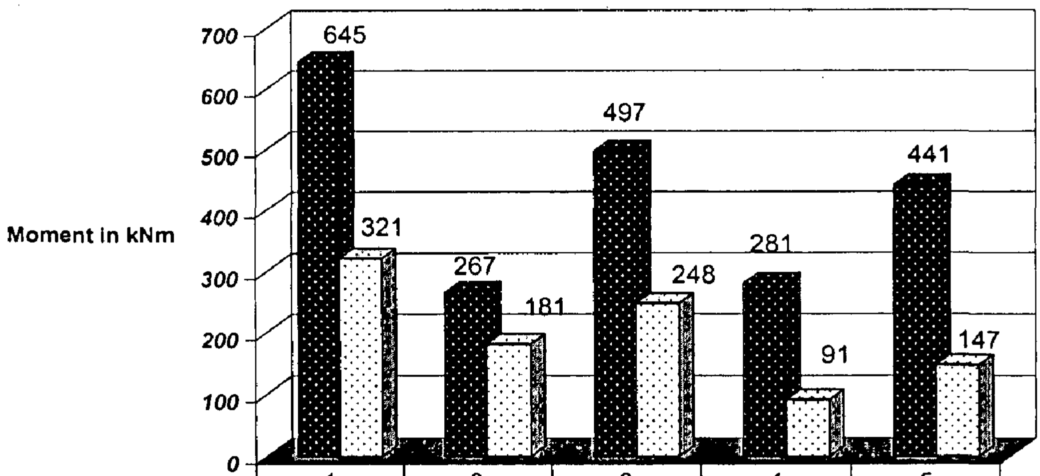
**Fig. 5.2 Comparison of beam design moments (one bay)**




| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ■ *Hogging Moment at Exterior End of Beams 34,35* | *661* | *267* | *509* | *281* | *441* |
| □ *Sagging Moments at Exterior End of Beams 34,35* | *324* | *181* | *249* | *91* | *147* |

**(1) IS 1893(Part1):2002 (2) Capacity Based Design**
**(3) Energy Based Design (4) Equal Displacement Based Design**
**(5) Direct Displacement Based Design**

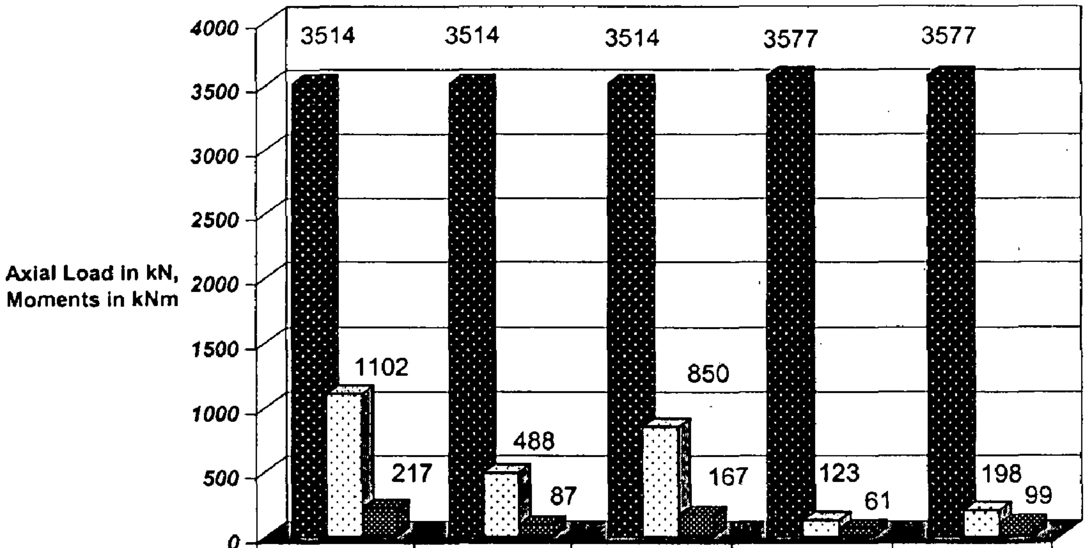
**Fig. 5.3 Comparison of beam design moments at exterior end (two bay)**




| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ■ *Hogging Moment at Interior End of Beams 34,35* | *645* | *267* | *497* | *281* | *441* |
| □ *Sagging Moment at Interior end of Beams 34,35* | *321* | *181* | *248* | *91* | *147* |

**(1 ) IS 1893(Part 1):2002 Code (2) Capacity Based Design**
**(3) Energy Based Design (4) Equal Displacement Based Design (5) Direct Displacement based Design Method**

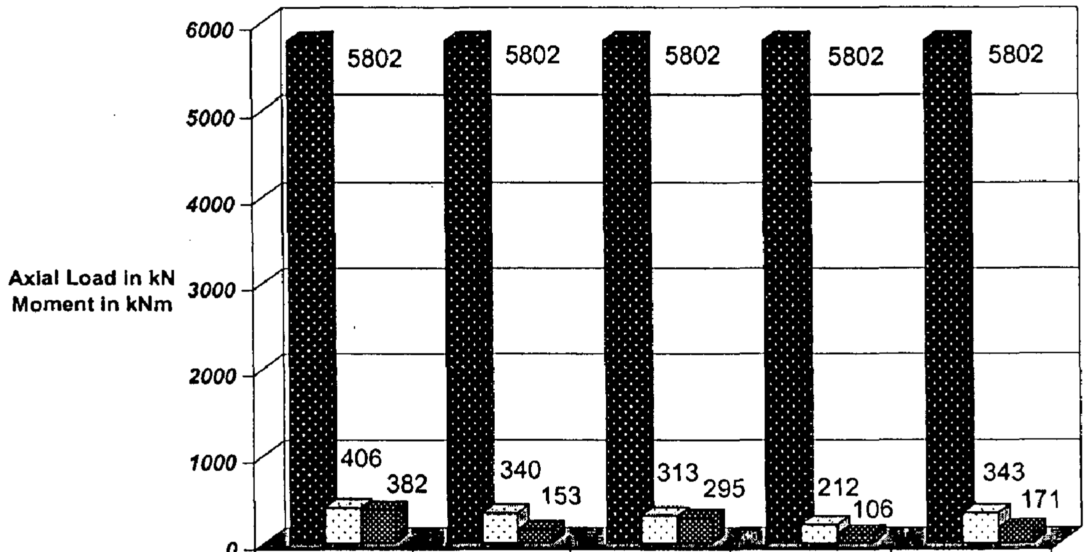
**Fig. 5.4 Comparison of beam design moments at interior end (two bay)**




| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ■ *Axial Load on Column 1,3* | *3514* | *3514* | *3514* | *3577* | *3577* |
| □ *Moment Mx on Column 1,3* | *1102* | *488* | *850* | *123* | *198* |
| ■ *Moment My on Column 1,3* | *217* | *87* | *167* | *61* | *99* |

**(1) IS 1893(Part 1):2002 Code (2) Capacity Based Design**
**(3) Energy Based Design (4) Equal Displacement Based Design**
**(5) Direct Displacement Based Design**

**Fig. 5.5 Comparison of design forces and moments in exterior columns (two bay)**




| | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| ■ *Axial Load on Column* | *5802* | *5802* | *5802* | *5802* | *5802* |
| □ *Moment Mx on Column 2* | *406* | *340* | *313* | *212* | *343* |
| ■ *Moment My on column 2* | *382* | *153* | *295* | *106* | *171* |

**(1) IS 1893(Part 1):2002 Code (2) Capacity Based Design**
**(3) Energy Based Design (4) Equal Displacement Based Design**
**(5) Direct Displacement Based Design**

**Fig. 5.6 Comparison of column design forces and moments in interior end (two bay)**

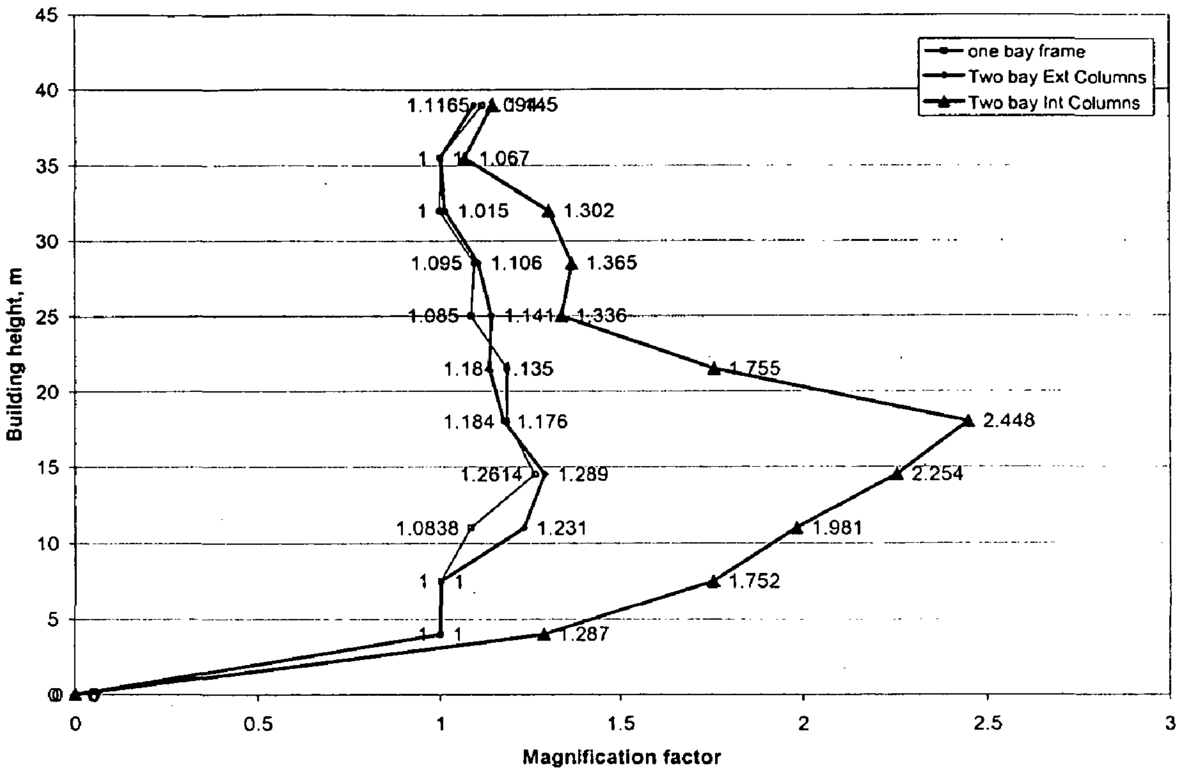



**Fig. 5.7 Comparison of moment magnification factors for interior and exterior columns of one and two bay frames**

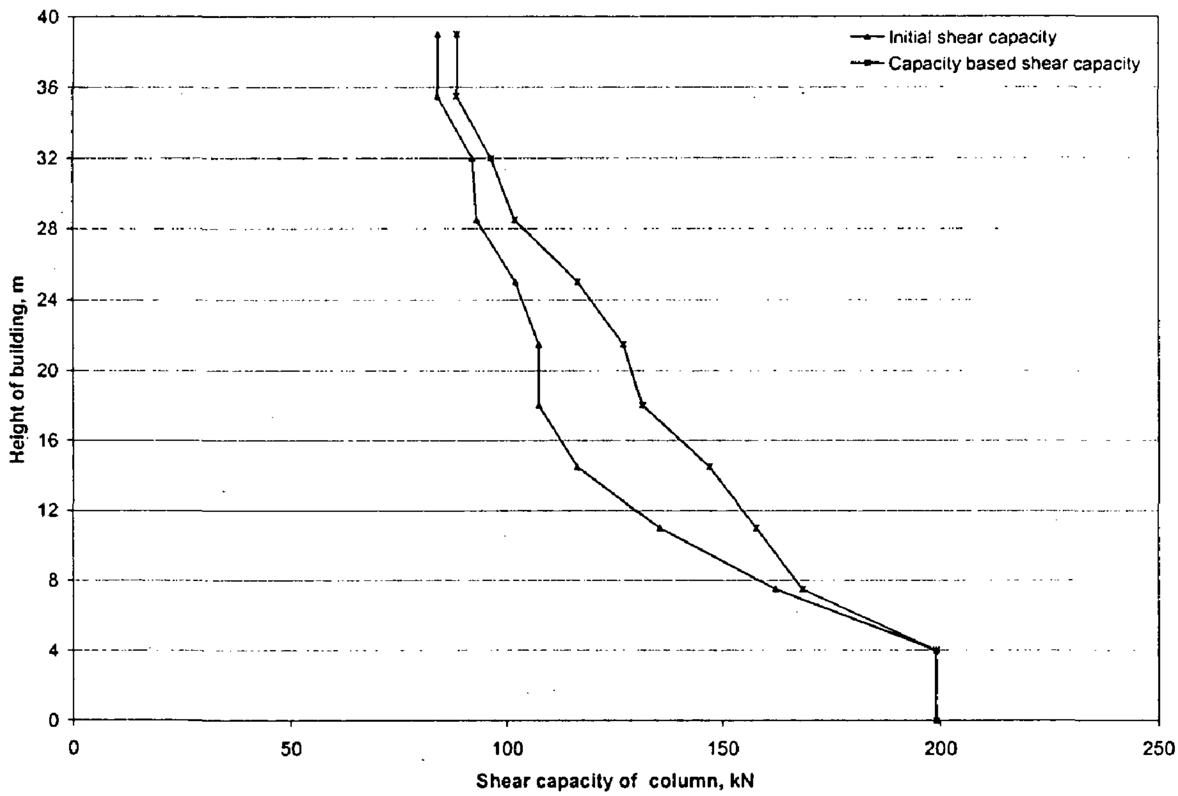



**Fig. 5.8 Comparison of dynamic analysis shear capacity to capacity based shear capacity of columns of one bay frame**

# SUMMARY AND CONCLUSIONS

## 6.1 SUMMARY

A G+10 RC building with one and two bay frames were taken for the study of a soft storey problem. The analysis and design was performed using the five different approaches as follows

a. *IS 1893(Part 1):2002* Code criteria
b. Capacity Based Design
c. Energy Based Design
d. Equal Displacement Based Design
e. Direct Displacement Based Design

Analysis of frame has been carried out using IS 1893(Part 1):2002 without considering the stiffness of infill walls. Capacity based design has been performed using the results obtained from the previous analysis of the frame for further revision of capacity. In energy based design, multiplication factors has been determined by assuming that sections of all columns are kept same in all floors for uniform structure as those from the initial analysis and infills are added for analysis of rigid structures. Equal and direct displacement based designs has been carried out with an assumed spectral velocity. The design of members has been carried out using SP-16.

## 6.2 CONCLUSIONS

The conclusions were drawn from the present study

IS 1893(Part 1):2002 Design Criteria

1. The base shear capacity, calculated from the column design moments observed to be 7.6 times higher than that of design base shear, calculated for design base earthquake with a response reduction factor of 5.0 of the design response spectra of IS 1893 (Part 1):2002. So that, the soft storey can resist 76% of the base shear of the maximum considered earthquake with out any reduction factor (i.e., R=1) for one-bay frame, and similarly 69 % for two bay frame.

Capacity Based Design

2. The moment magnification factors determined by using capacity based design method are observed to be

   (i) nearly equal to 1.0 at the joints of top and bottom storeys

   (ii) maximum at the joints of intermediate storeys

3. The moment magnification factor for one and two bay frames for the same span and loading are found to be

   (i) nearly equal at all the exterior joints

   (ii) higher near the interior joints compared to exterior joints

   The observations stated above, are attributed to the presence of (i) one beam and two columns at the exterior joint and (ii) two beams and two columns at interior joint, respectively.

4. An increase in strength of around 25 to 30% was observed at every section of the beam due to flange action of slab in one seismic direction.

Energy Based Design

5. The multiplying factors for the design of resisting elements of soft storey for G+10 storeyed one and two bay framed buildings are 1.825 and 1.928 for one and two bay frames respectively as against given 2.5 in IS 1893 (Part 1):2002.

6. The soft storey designed according to the multiplying factors obtained from energy based design concept can resist 55 % and 46% of the base shear of maximum considered earthquake with reduction factor of R=1 for one–bay and two bay frames respectively.

Equal Displacement Based Design

7. The design base shears calculated taking the spectral velocity of design based earthquake of zone V for rocky soil are found to be 52% of the design base shear of the design base earthquake with R = 5 for both one and two bay frames.

Direct Displacement Based Design

8. The design base shears calculated taking the spectral velocity of design based earthquake of zone V for rocky soil are found to be 79% and 84% of the design base shear of the design base earthquake with R = 5 for one and two bay frames respectively.

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# Appendix – A

## CALCULATION OF LOADS

**Table A-1 Loading data**

| | | | |
|---|---|---|---|
| Dead load (DL) | Finishes<br>Terrace water proofing(TWF) =1.5 kN/m$^2$ Floor finish(FF) = 0.5 kN/m$^2$ | Weight of Slab(Ws) 25 D kN/m2, Where D is the total depth of slab (Assume total depth of slab = 120mm) | Weight of walls<br>External walls<br>(230 mm thick)<br>= 4.5 kN/m/m |
| Live load (LL) | Roof Load= 2 kN/m<br>Floor Load = 4 kN/m$^2$ | | |

**Table A-2 Dead load calculation**

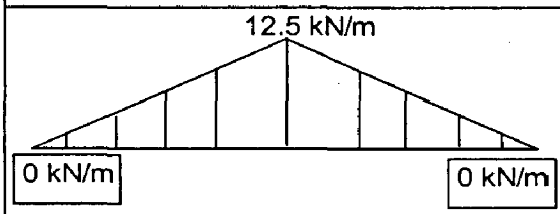
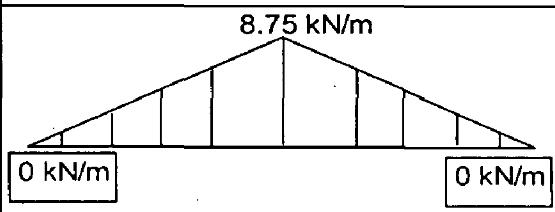
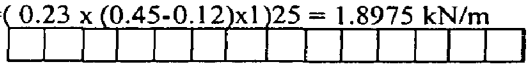
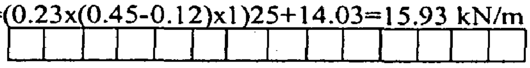
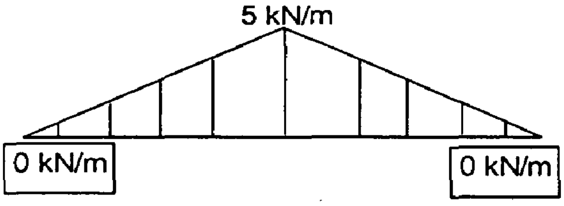
| | |
|---|---|
| ***Dead load at Roof level(For Beams):***<br>Weight of slab = 25 D = 25 x 0.12=3.0kN/m2<br>Weight of finishes<br>= FF+ TWF = 0.5 +1.5 = 2.0 kN/m2<br>Total intensity of load = 5.0 kN/m2<br>Slab weight on beam according to yieldline theory (triangular ditribution )<br>= ordinate times load intensity<br>=0x5(at left end) 2.5x5(at mid span) 0x5(at leftend) | ***Dead load at Floor level(For Beams):***<br>Weight of slab = 25 D = 25 x 0.12=3.0kN/m2<br>Weight of Floor finish<br>= FF = 0.5 kN/m$^2$<br>Total intensity of load = 3.0 +0.5 = 3.5 kN/m2<br>Slab weight on beam according to yieldline theory (triangular ditribution )<br>= ordinate times load intensity<br>0x3.5(at left end)2.5x3.5(at mid span) 0x3.5(at leftend) |
|  |  |
| Self weight of beam<br>=( 0.23 x (0.45-0.12)x1)25 = 1.8975 kN/m<br> | Self weight of beam + wall<br>=(0.23x(0.45-0.12)x1)25+14.03=15.93 kN/m<br> |
| ***Dead load at Roof level (For columns):***<br>(From Transverse beams, slab and column self weight)<br>slab dead load =( 0.5 x 2.5 x 12.5 )x2<br>= 31.25 kN on each column.<br>beam dead load =( 0.5 x 1.8975 x 5 )x2<br>= 9.4875kN/m on each column.<br>column self weight =( 0.3 x .45 x 1 )x25x 3.5<br>= 11.8125 kN on each column for 3.5 m column.<br>column self weight =( 0.3 x .45 x 1 )x25x 4<br>= 13.5 kN on each column for 4 m column. | ***Dead load at Floor level (For columns):***<br>(From Transverse beams, slab and column self weight)<br>slab dead load =( 0.5 x 2.5 x 8.75 )x2<br>= 21.875 kN on each column.<br>beam dead load =( 0.5 x 1.8975 x 5 )x2<br>= 9.4875kN/m on each column.<br>column self weight =( 0.3 x .45 x 1 )x25x 3.5<br>= 11.8125 kN on each column for 3.5m column.<br>column self weight =( 0.3 x .45 x 1 )x25x 4<br>= 13.5 kN on each column for 4 m column.<br>Wall load = 14.03 x (5-0.23)=66.92 on each column |

**Table A-3 Live load calculation**

<table>
<tr>
<td>Live load at Roof level(For Beams):<br>Intensity of live load = 2.0 kN/m2<br>Live load on beam according to yieldline theory<br>(triangular ditribution )<br>= ordinate times load intensity<br>=0x2(at left end) 2.5x2(at mid span) 0x2(at leftend)</td>
<td>Live load at Roof level(For Beams):<br>Intensity of live load = 2.0 kN/m2<br>Live load on beam according to yieldline theory<br>(triangular ditribution )<br>= ordinate times load intensity<br>=0x2(at left end) 2.5x2(at mid span) x2(at leftend)</td>
</tr>
<tr>
<td>
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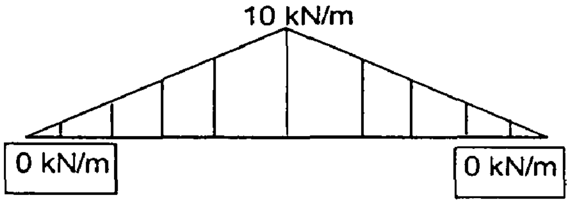
<td>
</td>
</tr>
<tr>
<td>Live load at Roof level (For columns):<br>(From slab live load)<br>slab dead load =( 0.5 x 5 x 5 ) = 12.5 kN on each column.</td>
<td>Live load at Roof level (For columns):<br>(From slab live load)<br>slab dead load =( 0.5 x 5 x 10) = 25 kN on each column.</td>
</tr>
</table>

# Appendix – B

# CALCULATION OF STIFFNESS

## B.1 CALCULATION OF LATERAL STOREY STIFFNESS

After Analysis and design of the G+10 storeyed plane frame structure with an open ground storey for the lateral and gravity loading combinations, is to be checked that its ground storey is a soft storey or not. To check that any storey is a soft storey, the lateral storey stiffness is calculated for every storey and checked the code criteria by calculating the ratio of lateral storey stiffness $K_1 / K_{i+1}$ .

According to IS 1893:2002, "a soft storey is that whose lateral storey stiffness is less than 70% of that in the storey immediately above or less than 80% of the average of combined stiffness of the three storey above". The calculated values of the lateral storey stiffness and check for code criteria are given in Table 3.3 & 3.4.

## B.2 MODELING OF MASONRY INFILL WALLS

Masonry infill-walls have been identified as a major cause of poor performance of reinforced concrete frame under high seismic loading. When infill panels are constructed with out full separation from the frame, structural stiffness is greatly increased and the natural period reduced, resulting in increased seismic forces. During seismic loading the infill-wall develops a diagonal compression strut with in the frame that convert the structural system to a type of truss. When infill panels are constructed without full separation from the frame, structural stiffness is greatly increased and the natural time period reduces, resulting in increased seismic forces.

Modeling of masonry infill-wall proposed by various researchers that is simplest and highly developed based on the concept of equivalent diagonal strut, In this method, the system is modeled as a braced frame where the infill-wall provides the web elements (equivalent diagonal struts). The geometric properties of the diagonal strut are function of the length of contact between the wall and the columns, $\alpha_h$ , and between the wall and the beams $\alpha_l$ .

### B.2.1 Equivalent Width of Diagonal Strut (W)

$$W = \frac{1}{2}\sqrt{\alpha_h^{\,2} + \alpha_L^{\,2}}\ \text{, where}\ \alpha_h = \frac{\pi}{2}\left[\frac{E_f I_c h}{2E_m t \sin 2\theta}\right]^{1/4},\ \alpha_L = \pi\left[\frac{E_f I_b L}{E_m t \sin 2\theta}\right]^{1/4}$$

$$\theta = \tan^{-1}\left(h/L\right),\ L_d = \sqrt{\left(h^2 + L^2\right)},\ A_d = tW$$

$E_f$, $E_m$, elastic modulus of the frame and masonry wall material respectively.

t, thickness of the infill wall

h, height of the infill wall

L, length of the infill wall

$I_c$, moment of inertia of the column

$I_b$, moment of inertia of beam

$L_d$, $A_d$are length and area of Equivalent Diagonal Strut

The specifications of equivalent diagonal strut and its stiffness of each storey are given in table 3.2.

### B.2.2 Stiffness of Diagonal Strut

The stiffness of equivalent diagonal strut is calculated based on the lateral stiffness of the strut i.e. given by $\frac{AE_m}{L}\cos^2\theta$

where A, Equivalent area of strut.

$E_m$, Modulus of Elasticity of Masonry,

L, Length of strut. The stiffness of strut in the (G+10) plane frame of each storey is given in Table 3.2

### B.2.3 Stiffness of Columns

The stiffness of columns is calculated by $\frac{12E_f\, I_c}{h^3}$

where, $E_f$, Modulus of elasticity of concrete

$I_c$, second moment of area of column section.

$h$, height of column.

The soft storey check is carried for the designed sections of the G+10 storeyed one-bay building plane frame. The specifications of equivalent diagonal strut is given in Table 3.2, summary of stiffness contribution from columns and infill walls of all floors is presented in Table 3.3. The soft storey check is presented in Table 3.4