EFFECT OF STRUCTURAL VERTICAL IRREGULARITIES ON RESPONSE OF A TEN STOREY BUILDING

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विद्या तत्व ज्योतिसमः

MAY 2011

Submitted in partial fulfillment of the Degree of Bachelor of Technology

DEPARTMENT OF CIVIL ENGINEERING
JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY-WAKNAGHAT

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CERTIFICATE

This is to certify that the work entitled, **"EFFECT OF STRUCTURAL VERTICAL IRREGULARITIES ON RESPONSE OF A TEN STOREY BUILDING"** submitted by ABHINAV SAXENA, HARSH SINGLA, KUNIKA SHARMA, MANISH MAHESHWARI, SUNNY CHOPRA in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering of Jaypee University of Information Technology has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Supervisor

Poonam 27/5/11

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Senior lecturer
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Dated on:

Certified the above mentioned project work has been carried out by the set group of students.

26/5/11

Dr. Ashok Kumar Gupta
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ACKNOWLEDGEMENT

The success of any project depends largely on the encouragement and guidelines of many others. Therefore we take this opportunity to express our sincere gratitude to the people who have been instrumental in the successful completion of the project. We would like to express our sincere appreciation and gratitude to our guide **Mrs. Poonam Dhiman** without whose able guidance, tremendous support and continuous motivation, the project would not have been carried to perfection. We sincerely thank them for spending all his valuable time and energies during the execution of project. We take this opportunity to express our sincere gratitude to the Head of Department of Civil Engineering, **Prof. (Dr.) Ashok Gupta** for all his support and valuable inputs. We thank him for reviving Civil Engineering as the overall mentor by leading with a vision and teaching us with a knowledge imparting attitude. The successful compilation of final year project depends on the knowledge and attitude inculcated in the total length of the course. So we want to express our sincere gratitude to all the faculties who taught us during the four years of B.Tech.

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CANDIDATE'S DECLARATION

We hereby certify that the work which is being presented in this report, **"EFFECT OF STRUCTURAL VERTICAL IRREGULARITIES ON RESPONSE OF A TEN STOREY BUILDING"** in partial fulfillment of the requirement for the award of B.Tech degree, submitted in the Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of our own work carried out from July 2010 to May 2011 under the guidance of Mrs. Poonam Dhiman, Senior Lecturer in Civil Engineering Department. We have not submitted the matter embodied in the report for the award of any other degree.

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SUMMARY

In this project we have used ETABS and STAAD.pro in order to analyze the effect of structural vertical irregularities on response of a ten storey building.

The six frames viz.

- Base frame,
- A frame having 4th and 5th storey soft,
- A frame with heavy loading on 4th and 7th storey,
- A frame with heavy loading on the Top storey,
- A frame having 1st and 2nd storey soft,
- A frame with floating column.

The above six cases are referred from IS 1893 (part 1) : 2002 and the loads were applied on the buildings as per IS 875 : 1987. The basic structure of all the frames is similar to the base frame except for the structure irregularities which were introduced in all the frames/buildings. Then they were analyzed on ETABS for their behavior during earthquake load when the building is situated in the 4th zone. The buildings/frames were analyzed on the basis of displacement, storey drift and storey shear.

LIST OF FIGURES AND TABLES

LIST OF ABBREVIATIONS

Symbol	Meaning
DL	Dead load
LL	Live Load
EL	Live Load
WL	Wind load
A_{st}	Area of tension reinforcement
f_y	Characteristic strength of steel
f_{ck}	Characteristic compressive strength of concrete
α_x, α_y	Bending moment coefficient for 2 way slab
e_x	Length of shorter side of slab
e_y	Length of longer side of slab
Pu	Axial load on compression member
d'	Clear cover
A_{sc}	Area of compression reinforcement
Q_{ult}	Axial load carrying capacity of the base
A_b	Cross-sectional area of base
α	Reduction factor
E_x	Eccentricity
E	Young modulus
I	Moment of inertia

CHAPTER 1

INTRODUCTION

"Earthquake don't kill people, buildings do."

1)INTRODUCTION

1.1) Introduction to project

The objective of this project is to analyze the effect of structural vertical irregularities on response of a ten storey building.

Practice problem

A two storey building with dimensions 4m x 4m with one bay having earthquake and other combinations loads is analyzed and designed both on STAAD Pro and ETABS in order to get a good command on the working of softwares.

Actual project problem

The ten storey buildings having different irregularities as taken from
IS 1893 (Part 1) : 2002 are analyzed on ETABS to study their behavior on application of loads.

1.2) Introduction to STAAD.pro

The STAAD.pro is explained briefly in the section below.

1.2.1) Introduction

STAAD.pro features a state-of-the-art user interface, visualization tools, powerful analysis and design engines with advanced finite element and dynamic analysis capabilities. From model generation, analysis and design to visualization and result verification, STAAD.prois the professional's choice for steel, concrete, timber, aluminum and cold-formed steel design of low and high-rise buildings, culverts, petrochemical plants, tunnels, bridges, piles and much more.The following key STAAD.pro tools help simplify ordinarily tedious tasks:

The STAAD.Pro Graphical User Interface incorporates Research Engineers' innovative tabbed page layout. By selecting tabs, starting from the top of the screen and heading down, you input all the necessary data for creating, analyzing and designing a model. Utilizing tabs minimizes the learning curve and helps insure you never miss a step.

The STAAD.Pro Structure Wizard contains a library of trusses and frames. Use the Structure Wizard to quickly generate models by specifying height, width, breadth and number of bays in each direction. Create any customizable parametric structures for repeated use. Ideal for skyscrapers, bridges and roof structures.

1.2.2) <u>Features ofSTAAD.Pro</u>

"Concurrent Engineering" based user environment for model development, analysis, design, visualization and verification

Full range of analysis including static, P-delta, pushover, response spectrum, time history, cable (linear and non-linear), buckling and steel, concrete and timber design included with no extra charge.

Object-oriented intuitive 2D/3D graphical model generation.

Pull down menus, floating tool bars, tool tip help.

Quick data input through property sheets and spreadsheets.

1.2.3) <u>Load Types and Generation</u>

Categorized load into specific load group types like dead, wind, live, seismic, snow, user-defined, etc. Automatically generate load combinations based on standard loading codes such as ASCE etc.

One way loading to simulate load distribution on one-way slabs

Patch and pressure loading on solid (brick) elements

Element pressure loads can be applied along a global direction on any imaginary surface without having elements located on that surface

Automatic wind load generator for complex inclined surfaces, irregular panels and multiple levels also taking into consideration user-defined panels

Loading for Joints, Members/Elements including Concentrated, Uniform Linear, Trapezoidal, Temperature, Strain, Support Displacement, Prestress and Fixed-end Loads.

1.3) Introduction to ETABS

1.3.1)Introduction

ETABS is a sophisticated, yet easy to use, special purpose analysis and design program developed specifically for building systems. ETABS Version 8 features an intuitive and powerful graphical interface coupled with unmatched modeling, analytical, and design procedures, all integrated using a common database. Although quick and easy for simple structures, ETABS can also handle the largest and most complex building models, including a wide range of nonlinear behaviors, making it the tool of choice for structural engineers in the building industry.

1.3.2)History and Advantages of ETABS

Dating back more than 30 years to the original development of ETABS, the predecessor of ETABS, it was clearly recognized that buildings constituted very special class of structures. Early releases of ETABS provided input, output and numerical solution techniques that took into consideration, the characteristics unique to building type structures, providing tool that offered significant savings in time and increased accuracy over general purpose programs.

Some of its advantages are

Most buildings are of straightforward geometry with horizontal beams and vertical columns. Although any building configurations possible with ETABS, in most cases, a simple grid system defined by horizontal floors and vertical column lines can establish building geometry with minimal effort.

Many of the floor levels in buildings are similar. This commonality can be used numerically to reduce computational effort.

The input and output conventions used correspond to common building terminology. With ETABS, the models are defined logically floor-by-floor, column-by-column, bay-by-bay and wall-by-wall and not as a stream of non-descript nodes and elements as in general purpose programs..Thus the structural definitions simple, concise and meaningful.

In most buildings, the dimensions of the members are large in relation to the bay widths and story heights. Those dimensions have a significant effect on the stiffness of the frame. ETABS corrects for such effects in the formulation of the member stiffness, unlike most general-purpose programs that work on centerline-to-centerline dimensions.

The results produced by the programs should be in a form directly usable by the engineer. General-purpose computer programs produce results in a general form that may need additional processing before they are usable in structural design.

1.3.3) FEATURES

ETABS offers the widest assortment of analysis and design tools available for the structural engineer working on building structures. The following list represents just a portion of the types of systems and analyses that ETABS can handle easily

Multi-story commercial, government and health care facilities

Parking garages with circular and linear ramps

Staggered truss buildings

Buildings with steel, concrete, composite or joist floor framing

Buildings based on multiple rectangular and/or cylindrical gridsystems

Flat and waffle slab concrete buildings

Buildings subjected to any number of vertical and lateral load cases and combinations, including automated.

Wind and seismic loads

i. Multiple response spectrum load cases, with built-in input curves
ii. Automated transfer of vertical loads on floors to beams and walls
iii. P-Delta analysis with static or dynamic analysis
iv. Explicit panel-zone deformations
v. Construction sequence loading analysis
vi. Multiple linear and nonlinear time history load cases in any direction
vii. Foundation/support settlement
viii. Large displacement analyses

ix. Nonlinear static pushover
x. Buildings with base isolators and dampers
xi. Floor modeling with rigid or semi-rigid diaphragms
xii. Automated vertical live load reductions
And much, much more!

1.3.4)Load Combinations

ETABS allows for the named combination of any previously defined load case or load combination. When a load combination is defined, it applies to the results for every object in the model. The four types of combinations are as follows:

ADD (Additive): Results from the included load cases or combos are added.

ENVE (Envelope): Results from the included load cases or combos are enveloped to find the maximum and minimum values.

ABS (Absolute): The absolute values of the results from the included load cases or combos are added.

SRSS: The square root of the sum of the squares of the results from the included load cases or combos is computed.

Design is always based on load combinations, not directly on load cases. You may create a combination that contains just a single load case. Each design algorithm creates its own default combinations; supplement them with your own design combination if needed.

CHAPTER 2

Introduction to Earthquake

2.1) Definition of an earthquake

A sudden and violent shaking of the ground, sometimes causing great destruction, as a result of movements within the earth's crust or volcanic action.

2.1.1) Types of earthquakes

Interplate- An interplate earthquake is an earthquake that occurs at the boundary between two tectonic plates.

Intraplate - An intraplate earthquake is an earthquake that occurs in the interior of a tectonic plate.

2.1.2) Types of cracks and faults

Dip slip - the slip generated at the fault during earthquakes along both vertical and horizontal directions.

Strike slip - the slip generated at the fault during earthquakes along lateral directions.

2.1.3) Inertia forces in structure

Earthquake cause shaking of ground so building will experience motion at its base. The roof remains intact in position but since column connects roof and base they tend to drag the roof along. When ground moves building is thrown backwards and roof experience inertia force. The inertia forces developed in roof are transferred to the columns and ultimately to the ground. This transferring through the columns lead to the development of some internal forces in columns called stiffness forces.

2.1.4) Stiffness forces

During earthquakes columns undergoes relative movement between its ends. If given a free will they would like to come back to their original vertical position i.e. they resist deformations. In this they don't have to carry any horizontal earthquake forces through them. But since during quake time it is forced to bend, stiffness forces are developed. Larger the relative movement between the ends greater is the force also if the column

Stiffness force in column = column stiffness x relative movement between the ends

2.1.5) Reason for twist in buildings

Non uniformity of structure
Non uniform loading

Buildings with unequal vertical members (*i.e., columns/walls) also the* floors twist about a vertical axis and displace horizontally. Buildings that are irregular shapes in plan tend to twist under earthquake shaking. Twist in buildings is called *torsion.* If this twist cannot be avoided, special calculations need to be done to account for this additional shear forces in the design of buildings; the Indian seismic code (IS 1893, 2002) has provisions for such calculations.

2.2) The earthquake design philosophy

Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damaged; however building parts that do not carry load may sustain repairable damage

Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake

Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse.

2.2.1) Ductility of buildings

The amount and location of steel in a member should be such that the failure of the member is by steel reaching its strength in tension before concrete reaches its strength in compression. This type of failure is *ductile failure, and hence is preferred over a* failure where concrete fails first in compression.

The correct building components need to be made ductile. The failure of a column can affect the stability of the whole building, but the failure of a beam causes localized effect. Therefore, it is better to make *beams to be the ductile* weak links than *columns. This method of designing RC* buildings is called the *strong-column weak-beam design* method

2.3)An earthquake-resistant building has four virtues in it, namely

1) Good Structural Configuration.

2) Lateral Strength.

3) Adequate Stiffness.

4) Good Ductility.

2.4) Behavior of masonry walls

Masonry walls are slender because of their small thickness compared to their height and length. A simple way of making these walls behave well during earthquake shaking is by making them act together as a box along with the roof at the top and with the foundation at the bottom. A number of construction aspects are required to ensure this box action.

Firstly, connections between the walls should be good. This can be achieved by

(a) Ensuring good interlocking of the masonry courses at the junctions, and

(b) Employing horizontal bands at various levels, particularly at the lintel level.

Secondly, the sizes of door and window openings need to be kept small. The smaller the openings, the larger is the resistance offered by the wall.

Thirdly, the tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and weight to thickness ratios.

2.4.1) Improvement Behavior of Masonry Walls

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Thirdly, the tendency of a wall to topple when pushed in the weak direction can be reduced by limiting its length-to-thickness and height to- thickness ratios.

2.4.2) Box Action in Masonry Buildings

Brick masonry buildings have large mass and hence attract large horizontal forces during earthquake shaking. They develop numerous cracks under both compressive and tensile forces caused by earthquake shaking. The focus of *earthquake resistant* masonry building construction is to ensure that these effects are sustained without major damage or collapse. Appropriate choice of structural configuration can help achieve this. The structural configuration of masonry buildings.

RC consist of two primary materials, namely concrete (sand, crushed stones and cement) with reinforcing steel bars. A typical RC building is made of horizontal member (beams and slabs) and vertical members (columns and walls) and supported by foundations rest on the ground forming all together an RC frame.

Earthquake shaking generates inertia forces in the building proportional to building mass which vary with floor levels. This inertial force developed at floor levels, travel downwards through slab and beams to columns and walls to the foundation from where they are immersed in the ground.

In RC buildings, the vertical and horizontal members (beams and columns) are built integrally with each other. Beams in RC building have two sets of reinforcement:

(a) ***Longitudinal bars***: it is placed along the length and provide resistance to flexural cracking at side's of the beam.

(b) ***Stirrups*****:** it carry the vertical force, resist diagonal shear cracking, it protects the concrete from bulging outward due to flexure, and also prevent buckling of compressed longitudinal bars.

Columns in RC building contain two type of steel reinforcement:

(a) Longitudinal bars (placed vertically along the length)

(b) Transverse ties (placed horizontally at regular intervals)

Columns can sustain to axial failure and shear failure and shear failure.

Under Earthquake shaking beams adjoining a joint are subjected to moment in the same direction. Under these moments, the top bars in the beam columns joint are pulled in one direction and bottom one in the opposite direction. These forces are balanced by bond stress developed between concrete and steel in the joint region.

The columns in the ground storey do not have any partition walls between them, these are open ground storey buildings. They have two distinct characteristics:

(a) It is relatively flexible in the ground, i.e. the horizontal displacement in the ground goes in large than any other storey above it.

(b) It is relatively weak in the ground storey, i.e. total horizontal earthquake force it can carry in the ground storey is significantly smaller than each of the storey above it.

Short columns are more damaged during earthquake due to the fact that in a earthquake, a tall and a short column of same cross section move horizontally by same amount Δ. Short columns are stiffer as compared to tall columns, and it attracts large earthquake forces. Larger the stiffness, Larger is the force required to deform it.

A vertical plate like RC wall is called Shear walls. In addition slabs, beams and columns, they are laid from the foundation and carried continuously throughout the building. They are like vertically oriented wide beams that carry on loads downwards to the foundation.

Two basic technologies of reducing earthquake affect on building are:

(a) Base Isolation Device

(b) Seismic Dampers

The idea of base isolation is to detach the building from the ground so that earthquake motions are not transmitted up through building.

Seismic dampers are device introduced to above the energy provided by ground motion of building.

2.5) Protection from Earthquakes

For a building to remain safe during earthquake shaking, columns should be stronger than beams, and foundations should be stronger than columns.

If columns are made weaker, they suffer severe local damage, at the top and bottom of a particular storey.

2.5.1) Earthquake Resistant Building Design Philosophy

a) Under minor but frequent shaking, the main members of the buildings that carry vertical and horizontal forces should not be damaged; however buildings parts that do not carry load may sustain repairable damage.

b) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts that do not carry load may sustain repairable damage.

c) Under strong but rare shaking, the main members may sustain severe damage, but the building should not collapse.

There are various new techniques which help in reducing the impact of earthquake forces on buildings. Most of these techniques are expensive to implement. The concept of base isolation is explained through an example building resting on frictionless rollers. When the ground shakes, the rollers freely roll, but the building above does not move. Thus, no force is transferred to the building due to the shaking of the ground; simply, the building does not experience the earthquake.Now, if the same building is rested on the flexible pads that offer resistance against lateral movements (fig 1b), then some effect of the ground shaking will be transferred to the building above. If the flexible pads are properly chosen, the forces induced by ground shaking can be a few times smaller than that experienced by the building built directly on ground, namely a fixed base building. The flexible pads are called base-isolators, whereas the structures protected by means of these devices are called base-isolated buildings.

2.5.2) Energy Dissipation Devices for Earthquake Resistance

Another approach for controlling seismic damage in buildings and improving their seismic performance is by installing Seismic Dampers in place of structural elements, such as diagonal braces. These dampers act like the hydraulic shock absorbers in cars where, much of the sudden jerks are absorbed in the hydraulic fluids and only little is transmitted above to the chassis of the car. When seismic energy is transmitted through them, dampers absorb part of it, and thus damp the motion of the building.

2.5.3) Active Control Devices for Earthquake Resistance

a) Sensors to measure external excitation and/or structural response.

b) Computer hardware and software to compute control forces on the basis of observed excitation and/or structural response.

CHAPTER 3

PRACTICE PROBLEM FORMULATION

3) Analysis and design of two storey building

A Building with two storey and one bay of dimension 4 x 4m is taken for analysis and designing under dead load, live load, earthquake load and other combination using STAAD.pro and ETABS for the learning of these softwares.

3.1) Problem formulation

3.1.1) Details of Building

The building has 1 bay 4m wide and 2 storeys.

- Grade of concrete =M 20
- Grade of steel =Fe 415
- Column size =150 mm
- Beam size =150 X 250 mm
- Live load on floor =3.8kN/m^2
- Live load on roof =1.5 kN/m^2
- Dead load on floor =2.48kN/m^2
- dead load on roof=2.48kN/m^2
- Floor finishes =1 kN/m^2
- Roof treatment =1.5 kN/m^2
- Storey height =4 m
- Density of concrete =25 kN/m^3

3.1.2) Analysis model for building

Number of members:	16
Number of joints:	8
Loading:	Self weight, Earthquake load, Dead and Live load
Analysis:	UsingSTAAD.Pro and ETABS

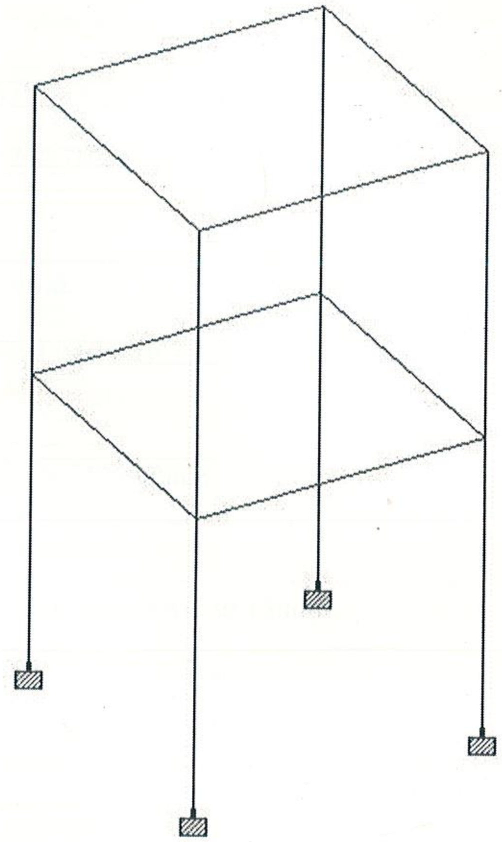


Figure 3.1 : Model of two storey building

3.1.3) Design of roof slab

Specifications:-

Dimension of roof slab	=4x4m
Depth of slab	=100mm
Dead Load	=2.48kN/m^2
Imposed Load	=1.5kN/m^2
Grade of Concrete	= M20
Grade of Steel	= Fe_{415}
Modular ratio for concrete	=1.0

Slab is to be designed as under-reinforced section so that

$p_t < pt_{lim}$

Let p_t = 0.20% (>.12%)

f_s = .58x415 = 240.7MPa ------ (.58 x f_y)

mf_t = 1.6

mf = 1.0 x 1.6 = 1.6

effective depth= d = (4 x 10^3) + d

20 x 1.6

or d = 4 x 10^3 = 129.03mm

(2.0 x (1.6-1))

Let 10mm bars with clear cover of 15mm

D = 129.03 + 15 + 5 = 149.03 mm

Provide D = 150 mm

$\Rightarrow$ d_x = 150-15-5 = 130 mm
$\Rightarrow$ d_y= 130-10 =120 mm
Effective span l_x= 4 + .130 = 4.130m

l_y= 4 +.120= 4.120m

Aspect ratio = r = l_y/l_x

= 4.120/4.130

= .99 =1.0

Consider 1m wide strip

Self wt. = w_g = .1 x 1 x 25 = 2.5 KN/m

D.L = 2.48 KN/m

L.L = 1.5 KN/m

Total load = 6.48 KN/m

Factored load = 6.48 x 1.5 = 9.72 KN/m

Now since the slab corners are prevented from lifting up

Let slab thickness be 100mm

$r = l_y/l_x = 1$

Therefore,

$\alpha x = .056$ -(as per table 26 IS-456)

$\alpha y = .056$

Moment

$M_{u,x} = \alpha_x w l_x^2$

$= .056 \times 9.72 \times (4.130)^2$

$= 9.28 KN/m^2$

Similarly,

$M_{u,y} = 9.28 KN/m^2$ required depth to resist this moment

$D = \sqrt{9.28 \times 10^6} / \sqrt{(.1388 \times 20 \times 1000)}$

$= 57.81 < 130$ mm

$$\text{Reinforcement} = \frac{f_{ck}bd}{2f_y}\left[1 - \sqrt{\frac{1 - 4.6\,M_u}{f_{ck}bd^2}}\right]$$

$$\frac{2o \times 1ooo \times 130 \ \times}{2 \times 415}\ 1 - \left[1 - \sqrt{\frac{(4.6 \times 9.72 \times 1000)}{(20 \times 1000 \times 130^2)}}\right]$$

$= 1139.32 \text{ mm}^2$

$$A_{st} \text{ provided} = \frac{1000 \times 78.5}{70} = 1121.42\text{mm}^2$$

Hence provide 10mm Ø bars @ 70mm spacing.

3.1.4) Corner reinforcement

Since the slab is torsionally restrained at corners, corner reinforcement is to beprovided at $l_x/5 = 0.826\text{m} = 826\text{mm}$

Say 30mm in both directions at top and bottom

Reinforcement in each layer = $0.75 \times A_{st,x}$ (in four layers)

Each layer, $A_{st} = .75 \times 1121.92 = 841.44\text{mm}^2$

at a spacing of $s = \frac{1000 \times 78.5}{841.44} = 93.2\text{mm}$

Hence provide 10mm Ø bars @ 95mm c/c both ways at top & bottom.

3.2) ANALYSIS OF BUILDING USING STAAD.pro

3.2.1) Data Input for Analysis with STAAD.pro

STAAD.pro requires data input in some form like graphical or text. The following data was fed to STAAD.pro graphically

1. Member lengths and locations
2. Mutual Connectivity of members
3. Supports
4. Assigning type and properties of members
5. Assignment of loads due to wind and earthquake, dead and live loads

Following data were inserted as text

1. Load Combinations
2. Load List for Analysis
3. Desired analysis results like Nodal displacements, Support reactions etc.

Summary of Nodal Displacements

The following table has been obtained from STAAD.pro results. It is obvious that the load cases containing Wind Load in X and Z directions are most critical.

TABLE 3.1- Summary of nodal displacement

	A	Horz	Horz.	Vert.	Horiz.	Res.	Rot.	H	I
	Node	L/C	X (mm)	Y (mm)	Z (mm)	(mm)	rX (rad)	rY (rad)	rZ (rad)
Max X	3	24 1.5(D.L. + W.L.X)	27.049	-0.193	0.004	27.05	0.002	0	- 0.003
Min X	4	25 1.5(D.L. - W.L.X)	- 27.044	-0.193	0.004	27.045	0.002	0	0.003
Max Y	3	5 W.L.X	18.03	0.024	0	18.03	0	0	- 0.001
Min Y	9	7 1.5(D.L+L.L)	0.006	-0.308	-0.006	0.308	- 0.002	0	- 0.002
Max Z	3	26 1.5(D.L. + W.L.Z)	0.004	-0.193	27.049	27.05	0.003	0	- 0.002
Min Z	9	27 1.5(D.L. - W.L.Z)	0.004	-0.193	- 27.044	27.045	- 0.003	0	- 0.002
Max rX	2	26 1.5(D.L. + W.L.Z)	-0.003	-0.125	15.408	15.408	0.004	0	- 0.001
Min rX	8	27 1.5(D.L. - W.L.Z)	-0.003	-0.125	- 15.398	15.399	- 0.004	0	- 0.001
Max rY	3	31 0.9D.L. - 1.5W.L.Z	0.002	-0.173	- 27.043	27.044	0	0	- 0.001
Min rY	3	26 1.5(D.L. + W.L.Z)	0.004	-0.193	27.049	27.05	0.003	0	- 0.002
Max rZ	5	25 1.5(D.L. - W.L.X)	- 15.398	-0.125	-0.003	15.399	0.001	0	0.004
Min rZ	2	24 1.5(D.L. + W.L.X)	15.408	-0.125	-0.003	15.408	0.001	0	- 0.004
Max Rst	3	24 1.5(D.L. + W.L.X)	27.049	-0.193	0.004	27.05	0.002	0	- 0.003

3.2 Summary of Member End Forces

The following table has been obtained from STAAD.pro results.

TABLE 3.2 - Summary of Member end forces

	Beam	L/C	Node	Fx (Kn)	Fy (Kn)	Fz (Kn)	Mx (kN-m)	MY (kN-m)	MZ (kN-m)
Max Fx	1	7 1.5(D.L+L.L)	1	35.04	-0.739	0.739	0	-0.984	-0.984
Min Fx	1	5 W.I.X	1	-3.104	2.061	0	0	0	4.932
Max Fy	6	24 1.5(D.L. + W.L.X)	5	0.907	9.743	0	0	0	10.782
Min Fy	6	25 1.5(D.L. - W.L.X)	2	-3.099	-9.743	0	0	0	10.783
Max Fz	1	27 1.5(D.L. - W.L.Z)	1	30.696	-0.536	3.628	0	-8.112	-0.714
Min Fz	5	27 1.5(D.L. - W.L.Z)	5	30.696	0.536	-3.628	0	6.401	1.431
Max Mx	11	26 1.5(D.L. + W.L.Z)	7	30.696	-0.536	-3.626	0	8.107	-0.714
Min Mx	11	31 0.9D.L. - 1.5W.L.Z	7	10.968	-0.322	2.768	0	-6.965	-0.428
Max My	11	26 1.5(D.L. + W.L.Z)	7	30.696	-0.536	-3.626	0	8.107	-0.714
Min My	1	27 1.5(D.L. - W.L.Z)	1	30.696	-0.536	3.628	0	-8.112	-0.714
Max Mz	7	27 1.5(D.L. - W.L.Z)	2	-3.099	9.743	0	0	0	10.783
Min Mz	1	25 1.5(D.L. - W.L.X)	1	30.696	-3.628	0.536	0	-0.714	-8.112

Summary of Support reactions

TABLE 3.3- Support reactions								
	A	Horz	Horz	Vert.	Horz.	Moment	G	H
	Node	L/C	FxkN	FykN	FzkN	MxkN-m	My kN-m	MzkN-m
Max Fx	1	25 1.5(D.L. - W.L.X)	5.608	30.696	0.536	0.714	0	-8.112
Min Fx	1	28 0.9D.L. + 1.5W.L.X	-4.75	10.968	0.322	0.428	0	6.97
Max Fy	1	7 1.5(D.L+L.L)	0.739	35.04	0.739	0.984	0	-0.984
Min Fy	1	5 W.L.X	-3.381	-3.104	0	0	0	4.932
Max Fz	1	27 1.5(D.L. - W.L.Z)	0.536	30.696	5.608	8.112	0	-0.714
Min Fz	1	30 0.9D.L. + 1.5W.L.Z	0.322	10.968	-4.75	-6.97	0	-0.428
Max Mx	1	27 1.5(D.L. - W.L.Z)	0.536	30.696	5.608	8.112	0	-0.714
Min Mx	7	26 1.5(D.L. + W.L.Z)	0.536	30.696	-3.626	-8.107	0	-0.714
Max My	7	26 1.5(D.L. + W.L.Z)	0.536	30.696	-3.626	-8.107	0	-0.714
Min My	7	31 0.9D.L. - 1.5W.L.Z	0.322	10.968	2.768	6.965	0	-0.428
Max Mz	6	24 1.5(D.L. + W.L.X)	-3.626	30.696	0.536	0.714	0	8.107
Min Mz	1	25 1.5(D.L. - W.L.X)	5.608	30.696	0.536	0.714	0	-8.112

DESIGN OF BUILDING- design results

TABLE 3. 4- DESIGN RESULTS

Beam	Analysis Property	Design Property	Ratio	Ay (mm^2)	Az (mm^2)	Ax (mm^2)	Dw (mm)	Bf (mm)	Iz (mm^4)	Iy (mm^4)	Ix (mm^4)
1	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E6
2	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E
3	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E
4	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E6
5	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E
6	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6
7	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6
8	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6
9	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6
10	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6
11	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E6
12	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E6
13	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6
14	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E
15	Cir 7.87	0X200	0.000	28E3	28E3	31.4E3	200.000	0.000	78.5E6	78.5E6	157E
16	Rect 5.91x9.84	250X150	0.000	31.9E3	31.9E3	37.5E3	150.000	250.000	70.3E6	195E6	176E6

CHAPTER 4

LITERATURE REVIEW

4.1)Seismic Zones of India

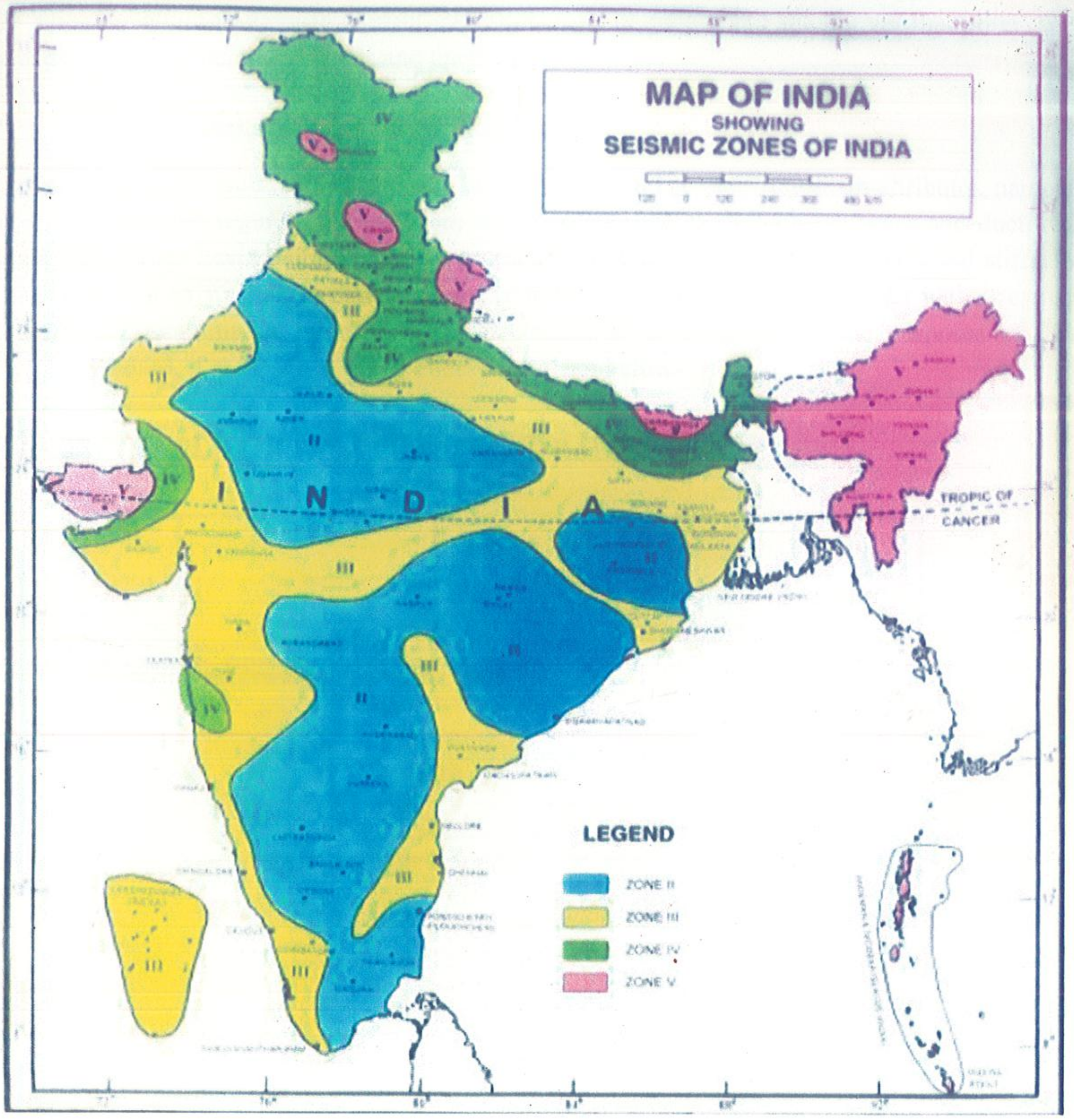


Figure 4.1 : Map showing the seismic zones of India

Towns falling at the boundary of zones demarcation line between two zones shall be considered in high zone.

4.2)BUILDINGS

A building should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility

4.2.1)Regular and Irregular Configurations:

To perform well in an earthquake, a building should possess four main attributes, namely simple and regular configuration, and adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation, suffer much less damage than buildings with irregular configurations. A building shall be considered as irregular for the purposes of this standard, if at least one of the conditions given below-

4.2.2)Definitions of Irregular Buildings - Plan Irregularities

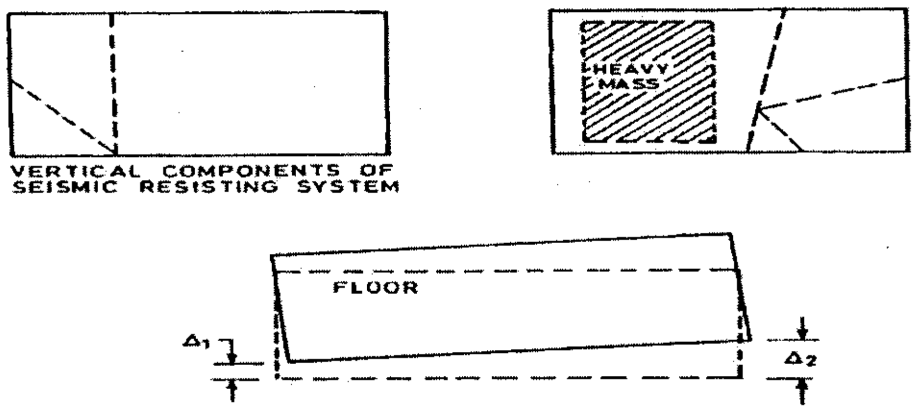


Figure 4.2a :Torsional Irregularity

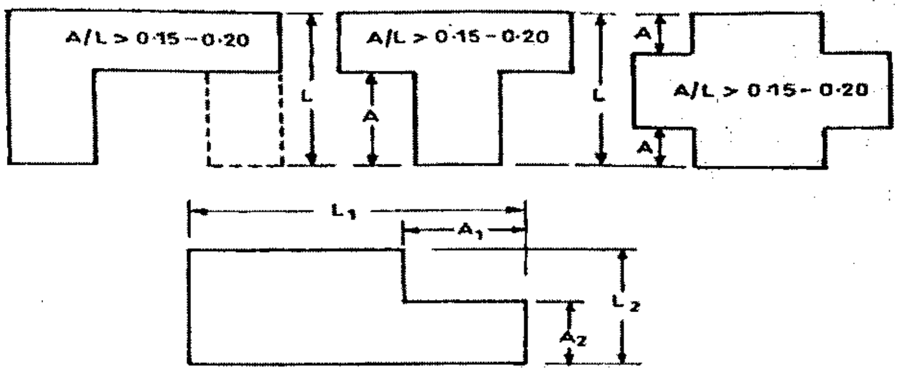


Figure 4.2b :Re-entrant corner

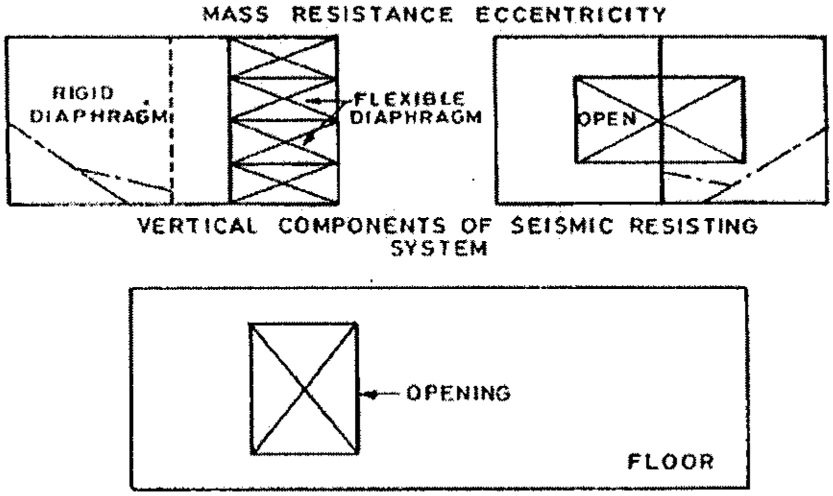


Figure 4.2c :Diaphragm Discontinuity

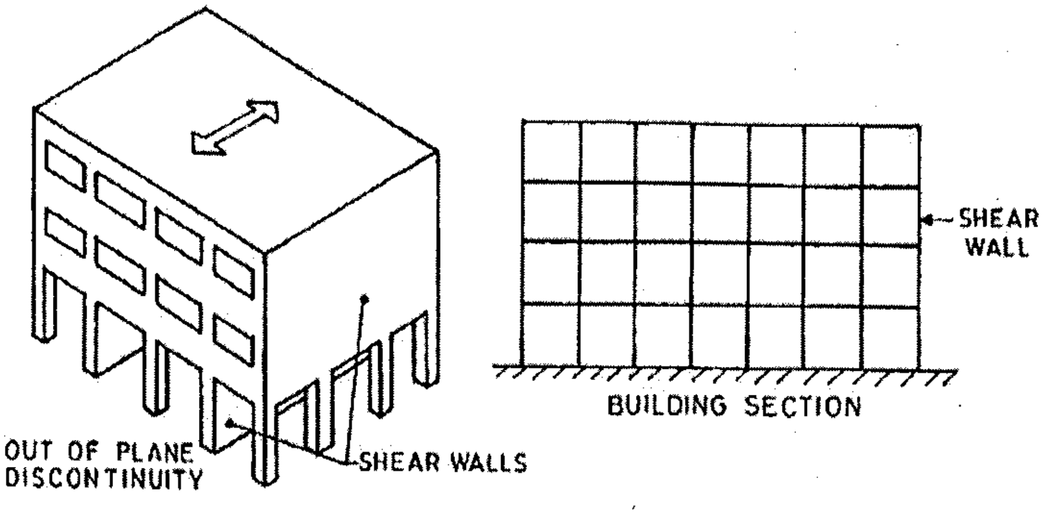


Figure 4.2d :Out of Plane Offset

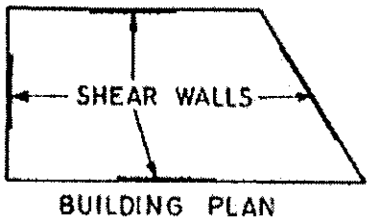


Figure 4.2e :Non parallel system

4.3)Irregularity Type and Description

4.3.1)*i)*Torsion Irregularities

Torsion irregularity shall be considered when floor diaphragms are rigid in their own plan in relation to the vertical structural elements that resists the lateral forces. Torsion irregularities is considered to exist when the maximum storey drift, computed with design eccentricity, at one end of the structure transverse to an axis is more than 1.2 times of the average of the storey drifts at the two ends of the structure.

Significant torsion will be taken as the condition where the distance between the storey's centre of rigidity and storey's centre of mass is greater than 20% of the width of the structure in either major plan dimension. Torsion or excessive lateral deflection is generated in asymmetrical building, or eccentric and asymmetrical layout of the bracing system that may result in permanent set or even partial collapse. Torsion is most effectively resisted at point farthest away from the centre of twist, such as at the corner and perimeter of the buildings.

***ii)* Re-entrant Corners**

The re-entrant, lack of continuity or " inside " corner is the common characteristics of over all building configurations that, in plan, assume the shape of an **L**, **T**, **H**, +, or combination of these shapes occurs due to lack of tensile capacity and force concentration. According to **IS 1893 (Part 1): 2002**, plan configuration of a structure and its lateral force resisting system contain re-entrant corners, where both projections of structure beyond the re-entrant corner are greater than 15% of its plan dimension in the given direction. The re-entrant corners of the building are subjected to two types of problem. The first is variation in rigidity and second problem is torsion.

***iii)* Diaphragm Discontinuity**

The diaphragm is a horizontal resistance element that transfers forces between vertical resistance elements. The diaphragm discontinuity may occur with abrupt variation in stiffness, including those having cut-out or open areas greater than 50% of the gross enclosed diaphragm area, or change in effective diaphragm stiffness of more than 50% from one storey to the next

***iv)*Out-of-Plane Offsets**

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

***v)* Non-parallel Systems**

The vertical load resisting elements are not parallel or symmetrical about the major orthogonal axis of the lateral-force resisting system. This condition results in a high probability of torsional forces under a ground motion, because the center of mass and resistance does not coincide. The narrower portion of the building tends to be more flexible than the wider ones, which will increase the tendency of torsion

4.4)Definition of Irregular Buildings-Vertical Irregularities

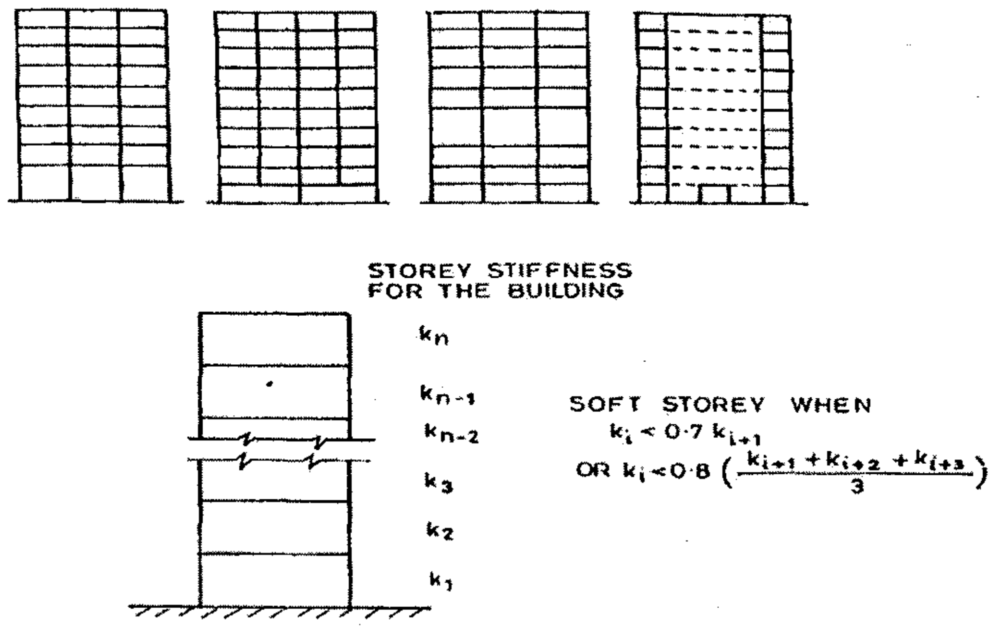


Figure 4.3a :Stiffness Irregularity

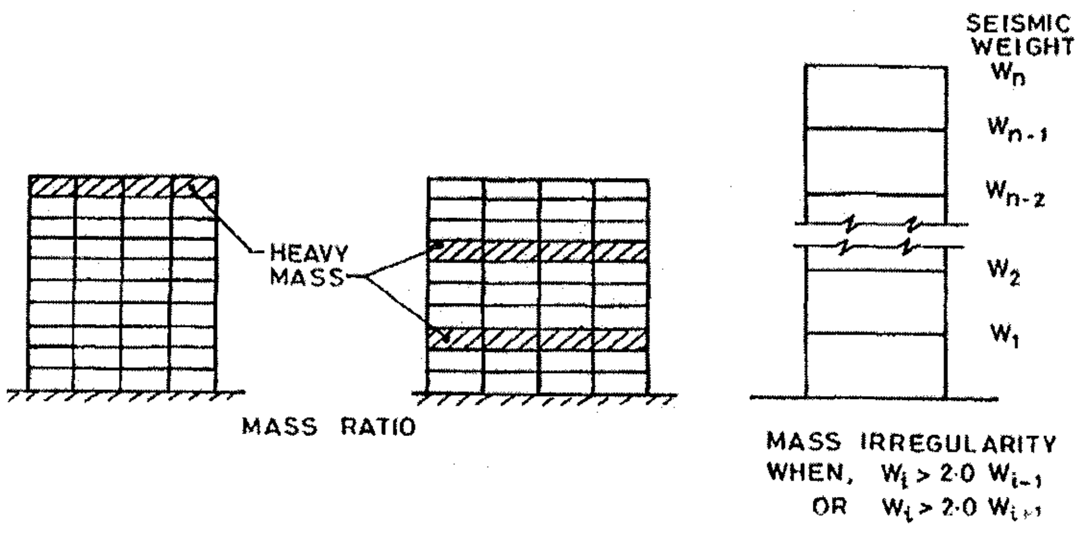


Figure 4.3b :Mass Irregularity

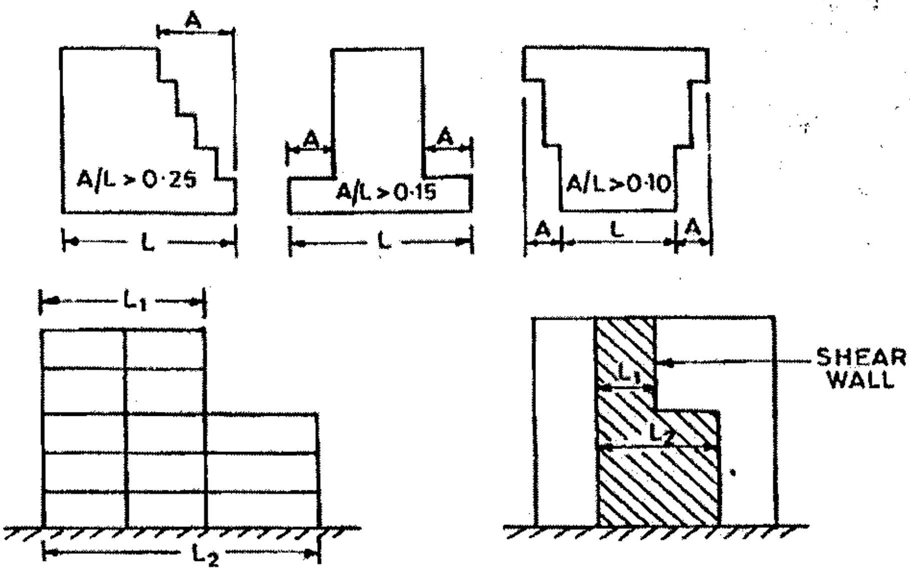


Figure 4.3c : Vertical Geometric Irregularity when L2>1.5 L

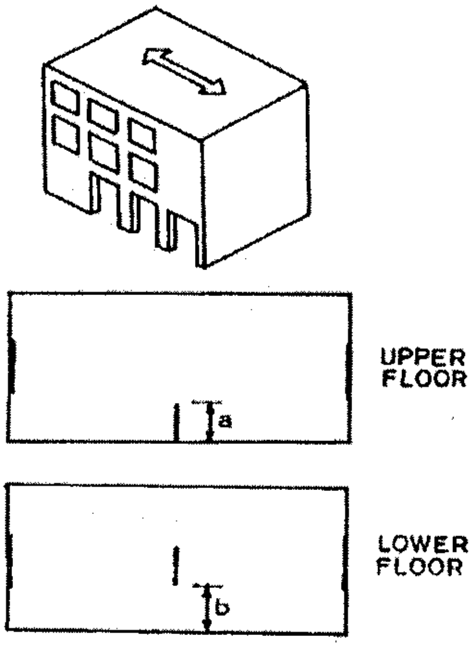


Figure 4.3d : In-Plane Discontinuity in Vertical Elements Resisting Lateral Force when b > a

4.4.1)Irregularity Type and Description

i-a) Stiffness Irregularity -Soft Storey

A soft storey is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above.

i-b) Stiffness Irregularity -Extreme Soft Storey

A extreme soft storey is one in which the lateral stiffness is less than 60 percent of that in the storey above or less than 70 percent of the average stiffness of the three storeys above. For example, buildings on STILTS will fall under this category.

ii) Mass Irregularities

Mass irregularities are considered to exist where the effective mass of any storey is more than 200% of effective mass of an adjacent storey. The effective mass is the real mass consisting of the dead weight of the floor plus the actual weight of partition and equipment. Excess mass can lead to increase in lateral inertial forces, reduced ductility of vertical load resisting elements, and increased tendency towards collapse due to P-Δ effect. Irregularities of mass distribution in vertical and horizontal planes can result in irregular response and complex dynamics. The central force of gravity is shifted above the base in the case of heavy masses in upper floors resulting in large bending moments.

iii) Vertical Geometric Irregularity

Geometric irregularity is considered, when the horizontal dimension of the lateral force resisting system in any storey is more than 150% of that in an adjacent storey. The setback can also be visualized as a vertical reentrant corner. The general solution of a setback problem is the total seismic separation in plan through separation section, so that the portion of building are free to vibrate independently.

iv)In-Plane Discontinuity in Vertical ElementsResisting Lateral Force

A in-plane offset of the lateral force resisting elements greater than the length of those elements.

v)Discontinuity in Capacity - Weak Storey

A weak storey is one in which the storey lateral strength is less than 80 percent of that in the storey above, the storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction.

Seismic weight of building

Seismic weight of all floors $= M_1 + M_2 + M_3 + M_3$

$= 64.45 + 64.45 + 64.45 + 37.08$

$= 230.43$ ton

Note: The seismic weight of each floor is its full dead load plus approximate amount of imposed load, as specified in Clause 7.3.1 and 7.3.2 of IS 1893(Part 1): 2002. Any weight supported in between stories shall be distributed to the floors above and below in inverse proportion to its distance from the floors.

Determination of Fundamental Natural Period

The approximate fundamental natural period of a vibration (T_a), in seconds, of a moment resisting frame building without brick infill panels may be estimated by the empirical expression

$$T_a = 0.075 \times h^{0.75} = 0.075 \times 14^{0.75} = 0.5423\ s$$

Where h is the height of the building in meters.

Determination of Design Base Shear

Design seismic base shear, $V_B = A_h W$

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{S_a}{g} = \frac{0.24}{2}\frac{1}{5}\,1.842 = 0.0443$$

For $T_a = 0.5423 \rightarrow \frac{S_a}{g} = \frac{1}{T_a} = 1.842$, for rock site from

Figure 2 of IS 1893(Part 1) : 2002

Design seismic base shear, $V_a = 0.0443 \times (230.43 \times 9.81) = 99.933\ kN$

Vertical Distribution of Base Shear

The design base shear (V_B) computed shall be distributed alone the height of the building as per the expression,

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

Where,

$Q_i = Design\ lateral\ forces\ at\ floor\ i,$

$W_i = Siesmic\ weight\ of\ the\ floor\ i,$

$h_i = Height\ of\ floor\ i, measured\ from\ base,$

n = Number of stories

Using above equation , base shear is distributed as follows:

$$Q_1 = V_B \left(\frac{W_1 h_1^2}{W_1 h_1^2 + W_2 h_2^2 + W_3 h_3^2 + W_4 h_4^2}\right)$$

$$Q_1 = 99.933 \left(\frac{632.25 \times 3.5^2}{632.25 \times 3.5^2 + 632.25 \times 7^2 + 632.25 \times 10.5^2 + 363.82 \times 14^2}\right)$$

$= 4.306$ kN

Similarly,

$$Q_2 = 0.1742 \times 99.933 = 17.224\ kN$$

$$Q_3 = 0.3872 \times 99.933 = 38.733\ kN$$

$$Q_4 = 0.3967 \times 99.933 = 39.646\ kN$$

Lateral force distribution at various floor levels

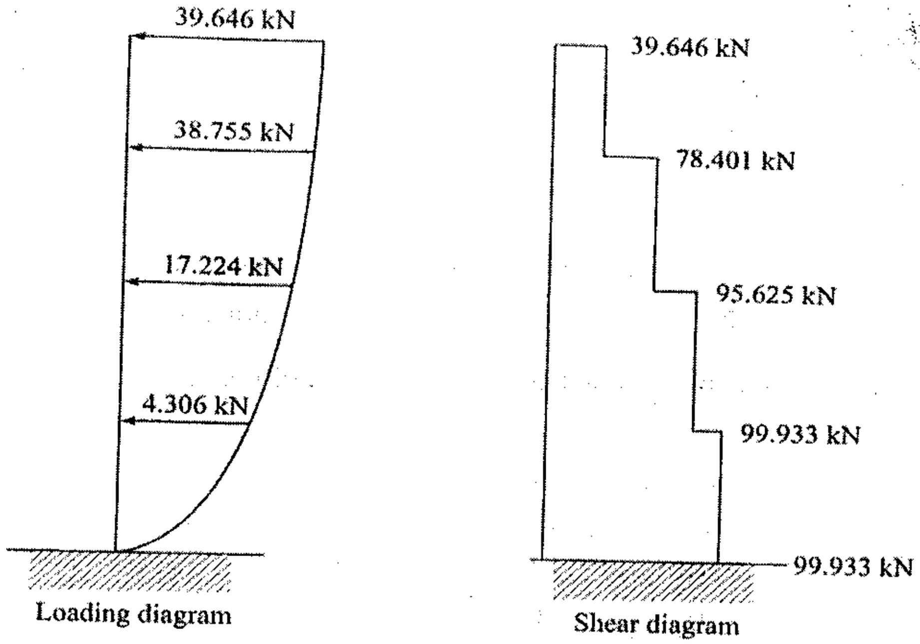


Figure 4.4 :Loading and shear diagram

Chapter 5

PROBLEM FORMULATION

5) Analysis of Ten storey building for vertical irregularities:

A ten storey building with different structural irregularities are analyzed on ETABS.

5.1)Reference of the problem

The problem considered for this project is taken from two references i.e. a paper published in 1997 and IS 1893 (part 1) : 2002 whose description are given as under .

5.1.1) Ref.no.1

"*Seismic response of buildings frames with vertical structural irregularities*", published in journal of structural engineering in January 1997 by EggertV.Valmundsson and James M. Nau, Member, ASCE

* Refer to Appendix B.

5.1.2) Ref.no.2

From the Bureau of Indian Standards, Criteria for earthquake resistant Design of structures part 1 general provisions and buildings, IS 1893 (part 1) : 2002.

5.2) Problem formulation

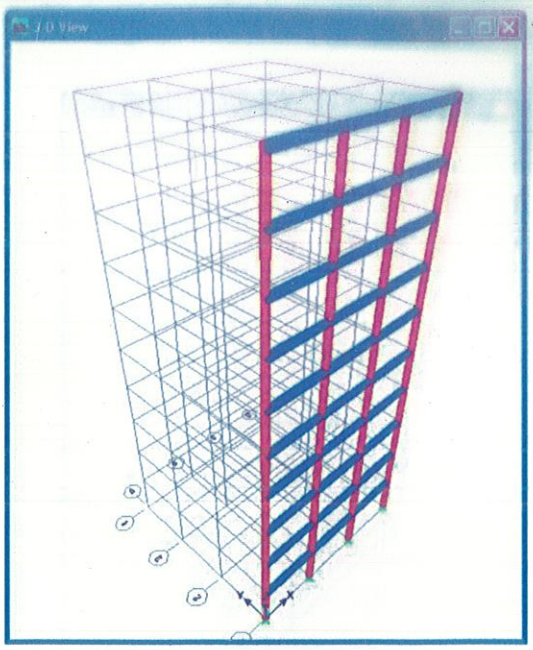
The ten storey buildings having different irregularities as taken from IS 1893 (Part 1) : 2002 are analysed on ETABS to study their behavior on application of loads.

No of cases assumed = 6

These are discussed as below

1. Base frame

This is the basic and the regular structure of the building with no irregularities and having three bays and ten storeys, with a storey height of 3.5m and the bay width of 5m.

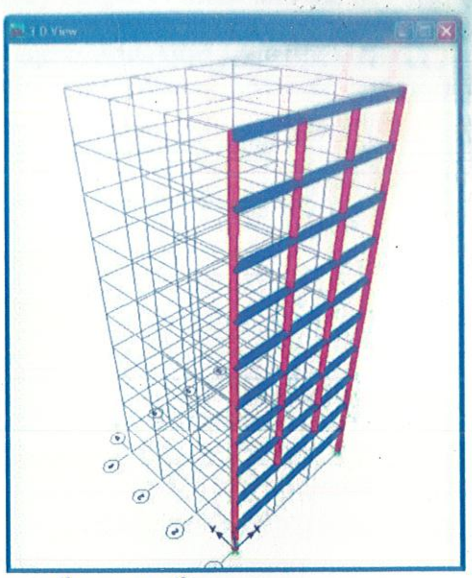


The basic specifications of the building are as follows

Dimensions of the beam	= 0.45× 0.25
Column size	= 0.30 × 0.30
Beam length	= 5 m
Column length	= 3.5 m
Load combination	= DL + LL + EQL
Dead Load	= 8.5 kN
Live Load	= 10 kN

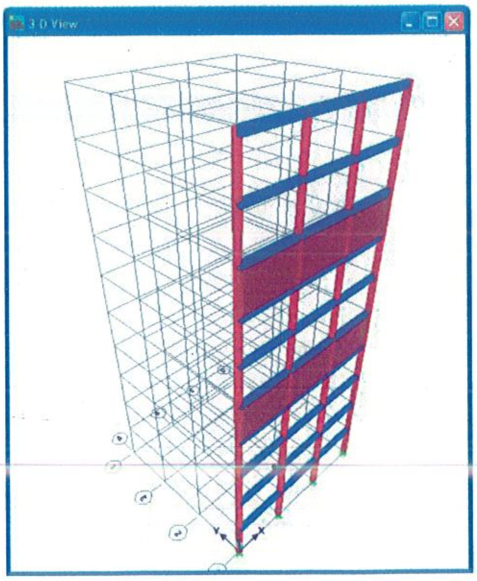
2. **A frame with floating column**

This is the frame which unlike the base frame, some irregularities have been introduced i.e. the two middle columns are being left hanging on the first storey and hence not reaching the ground, making the building with floating columns.It has three bays and ten storeys, with a storey height of 3.5m and the bay width of 5m.



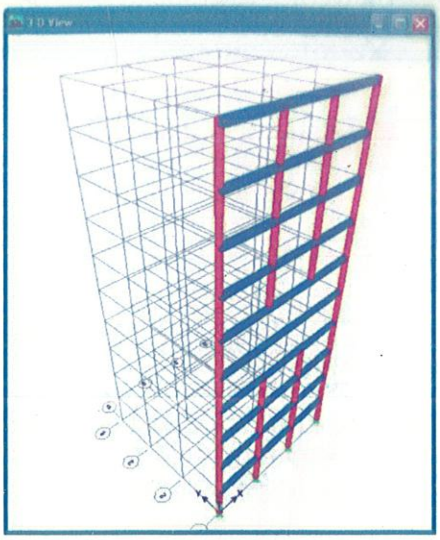
3. **A frame with heavy loading on 4th and 7th storey**

This is the frame which unlike the base frame, some irregularities have been introduced i.e. heavy loading has been introduced in the two storeys of the building i.e. 4th and 7th storey, hence making the building irregular.It has three bays and ten storeys, with a storey height of 3.5m and the bay width of 5m.



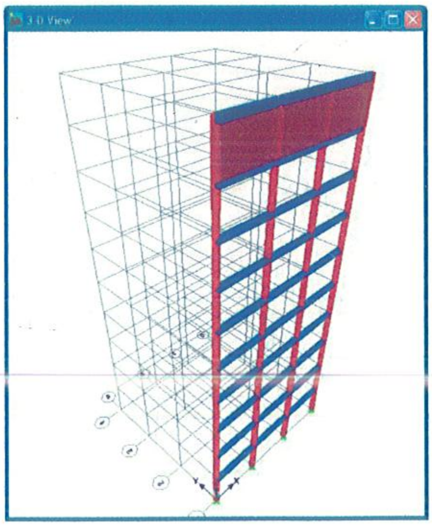
4. **A frame having 4th and 5th storey soft**

This is the frame which unlike the base frame, some irregularities have been introduced i.e. in the 4th and the 5th storey no floor slab has been provided which makes these two storeys soft, hence making the building irregular. It has three bays and ten storeys, with a storey height of 3.5m and the bay width of 5m.



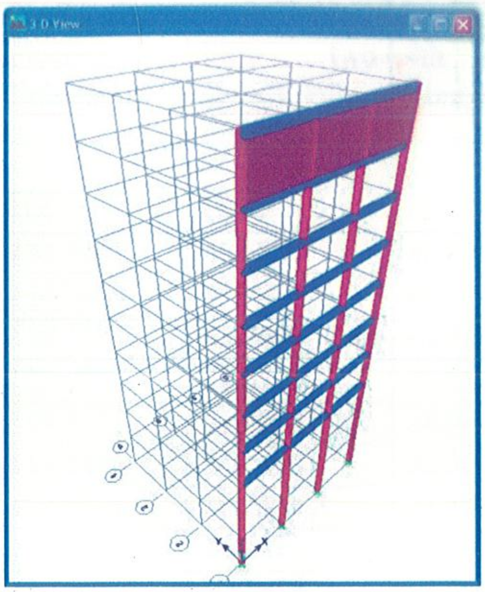
5. **A frame with heavy loading on the Top storey**

This is the frame which unlike the base frame, some irregularities have been introduced i.e. in the top storey swimming pool has been introduced hence making the top storey heavy, hence making the building irregular. It has three bays and ten storeys, with a storey height of 3.5m and the bay width of 5m.



6. A frame having 1st and 2nd storey soft

This is the frame which unlike the base frame, some irregularities have been introduced i.e. in the 1st and the 2nd storey no floor slab has been provided which makes these two storeys soft, hence making the building irregular. It has three bays and ten storeys, with a storey height of 3.5m and the bay width of 5m.



5.3) Analysis of results

The above discussed frames are analyzed for their behavior under the load combinations of (DL+LL+EQL) using ETabs.

The results after analyzing are discussed as below:

5.3.1) Displacement in the frames

After analyzing the above considered six cases in ETABS the displacement shown by the building in universal X direction at all ten storeys is given in table 5.

TABLE 5.1 – Displacement in X direction

	Base Frame	Top storey heavy	Floating Columns	First two storeys soft	4th and 7th storeys heavy	4-5th storeys soft
Story	UX	UX	UX	UX	UX	UX
STORY10	32.8944	54.7926	134.7907	46.5885	46.7261	47.189
STORY9	31.7777	54.6869	134.7023	45.8299	45.5418	46.4236
STORY8	29.7106	50.175	130.4993	43.7662	43.7754	44.3027
STORY7	26.9246	44.3691	125.0986	41.0559	43.6665	41.5488
STORY6	23.5828	38.1508	119.3131	37.7978	38.0646	37.9406
STORY5	19.8416	31.682	113.2898	34.1496	32.3048	27.0572
STORY4	15.8341	25.0611	107.0725	30.2335	32.2094	15.7235
STORY3	11.6716	18.3703	99.5636	26.181	24.9491	11.179
STORY2	7.4441	11.6795	71.7422	21.7459	15.9561	7.1457
STORY1	3.2694	5.1138	24.8356	9.7551	6.9974	3.1288
BASE	0	0	0	0	0	0

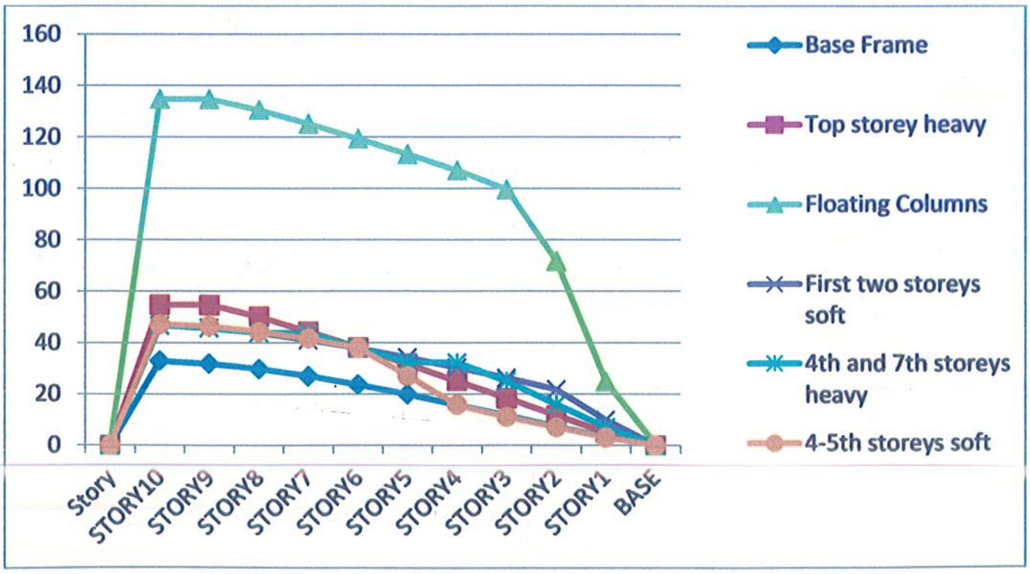


Figure 5.1 : Graph showing displacement in X direction

5.3.2) Storey drift in the above frames

After analyzing the above considered six cases in ETABS the storey drift shown by the building in universal X direction at all ten storeys is given in table 6.

TABLE 5.2 – Storey drift in X direction

	Base Frame	Top storey heavy	Floating Columns	First two storeys soft	4th and 7th storeys heavy	4-5th storeys soft
Story	UX	UX	UX	UX	UX	UX
STORY10	1.1167	0.1057	0.0884	0.7586	1.1843	0.7654
STORY9	2.0671	4.5119	4.203	2.0637	1.7664	2.1209
STORY8	2.786	5.8059	5.4007	2.7103	0.1089	2.7539
STORY7	3.3418	6.2183	5.7855	3.2581	5.6019	3.6082
STORY6	3.7412	6.4688	6.0233	3.6482	5.7598	10.8834
STORY5	4.0075	6.6209	6.2173	3.9161	0.0954	11.3337
STORY4	4.1625	6.6908	7.5089	4.0525	7.2603	4.5445
STORY3	4.2275	6.6908	27.8214	4.4351	8.993	4.0333
STORY2	4.1747	6.5657	46.9066	11.9908	8.9587	4.0169
STORY1	3.2694	5.1138	24.8356	9.7551	6.9974	3.1288
BASE	0	0	0	0	0	0

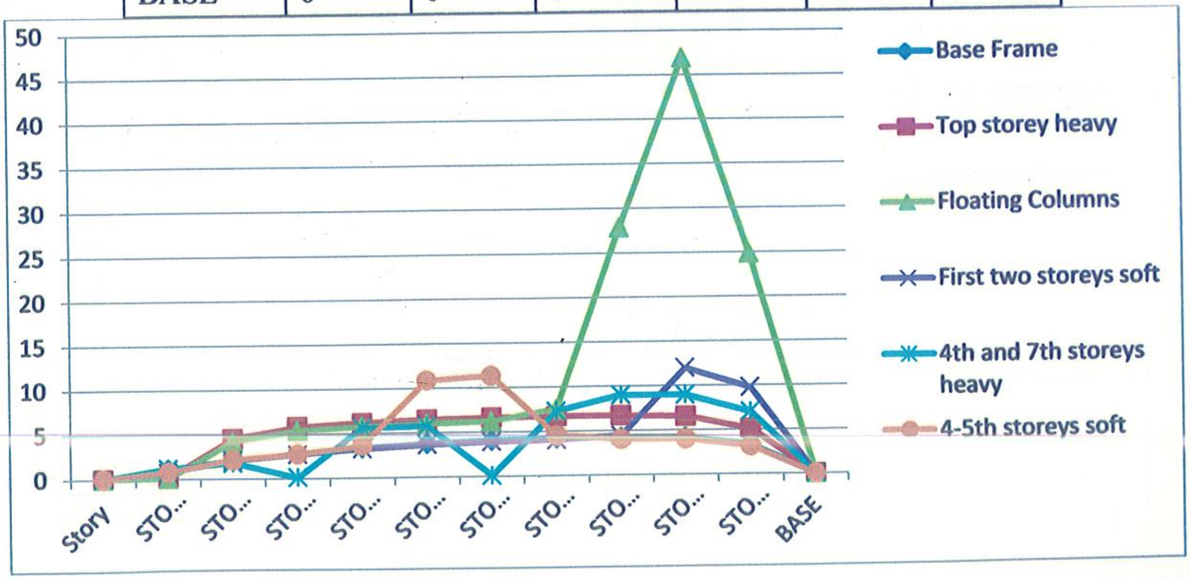


Figure 5.2 : Graph showing Storey drift in X direction

5.3.3) Storey shear in the above frames

After analyzing the above considered six cases in ETABS the storey shear shown by the building in universal X direction at all ten storeys is given in table 7.

TABLE 5.3 – Storey shear in X direction

	Base Frame	Top storey heavy	Floating Columns	First two storeys soft	4th and 7th storeys heavy	4-5th storeys soft
Story	**VX**	**VX**	**VX**	**VX**	**VX**	**VX**
STORY10	-9.39	-28.13	-26.16	-9.13	-9.2	-9.26
STORY9	-19.16	-52.33	-48.68	-18.63	-18.78	-18.89
STORY8	-26.87	-57.41	-53.4	-26.13	-47.61	-26.5
STORY7	-32.78	-61.29	-57.02	-31.87	-69.68	-32.33
STORY6	-37.12	-64.15	-59.67	-36.09	-73.93	-36.24
STORY5	-40.14	-66.13	-61.52	-39.03	-85.19	-38.63
STORY4	-42.06	-67.4	-62.7	-40.9	-92.4	-40.32
STORY3	-43.15	-68.12	-63.36	-41.96	-93.47	-41.39
STORY2	-43.63	-68.43	-63.49	-42.39	-93.94	-41.87
STORY1	-43.75	-68.51	-63.52	-42.48	-94.06	-41.98

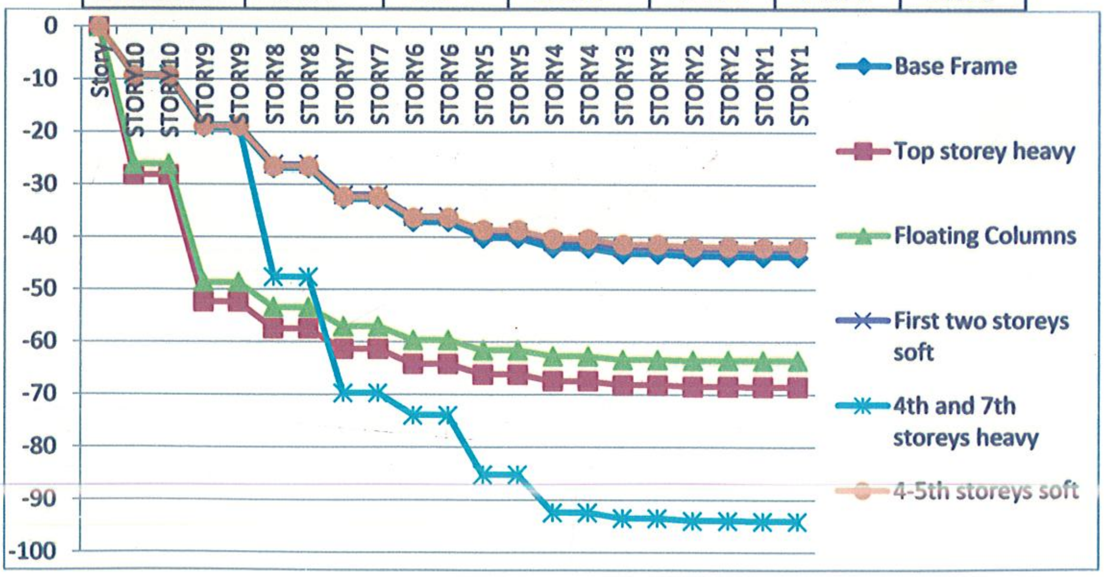
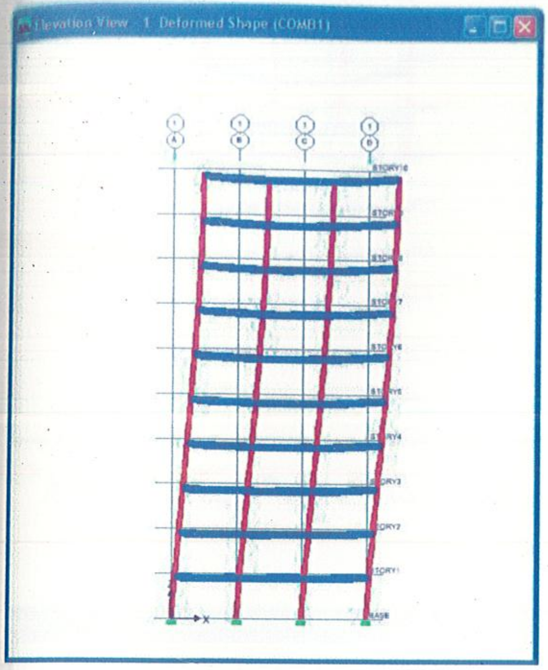
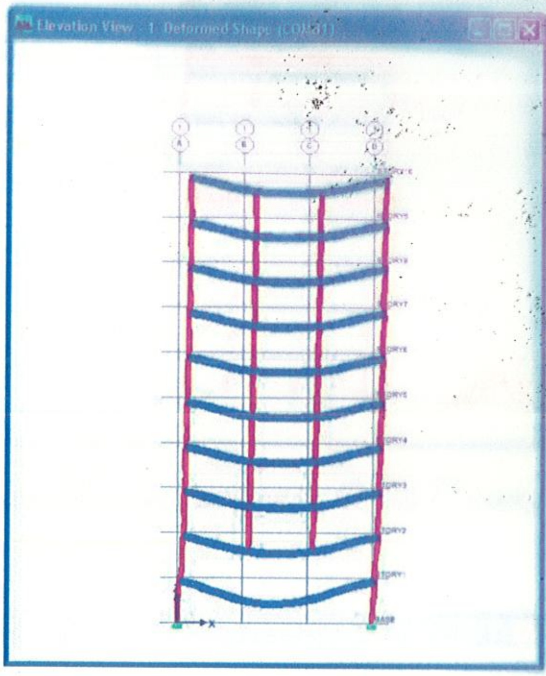


Figure 5.3 : Graph showing Storey shear in X direction

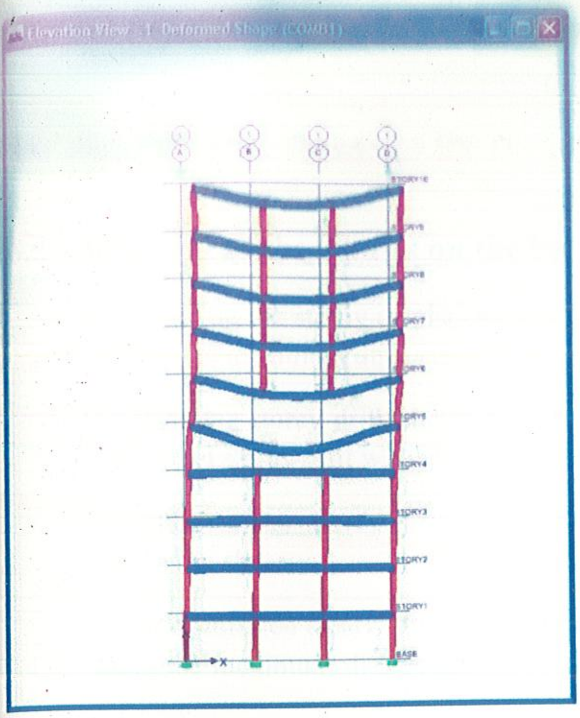
5.4) The deflected shapes of the frames are shown below



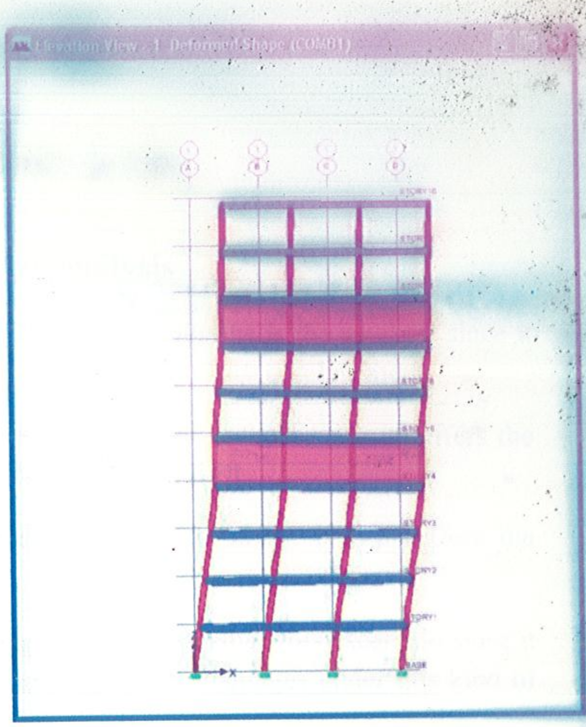
Base frame



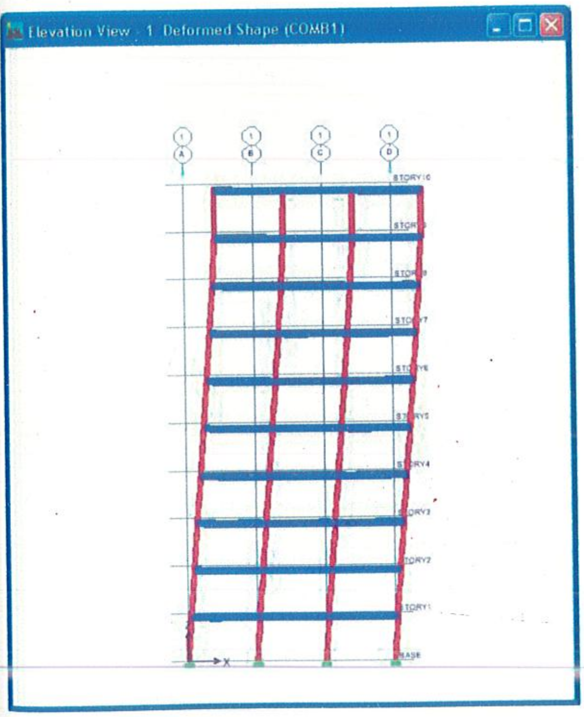
A frame with floating column



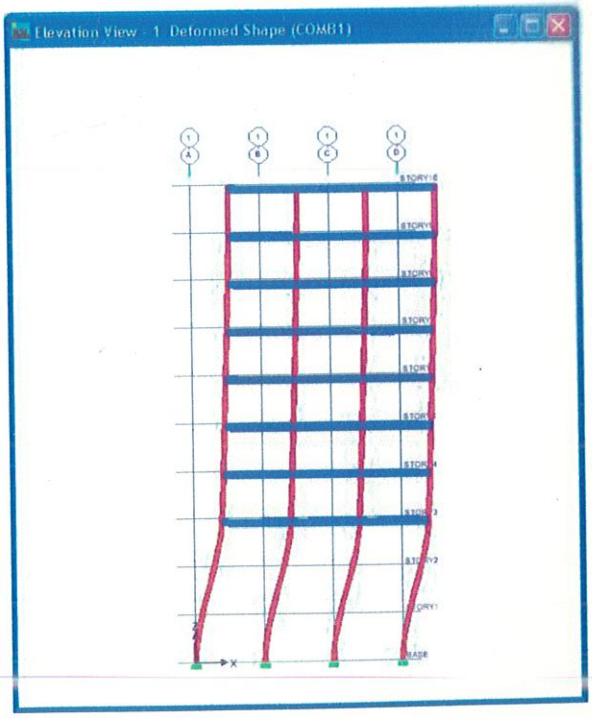
A frame having 4th and 5th storey soft



A frame with heavy loading on 4th and 7th storey



A frame with heavy loading on the Top storey



A frame having 1st and 2nd storey soft

5.5) Judgment of the frames on the basis of above analysis

Considering the storey displacement, the frame with floating columns is the weakest since it suffers the maximum displacement while the bas frame suffers the least displacement.

Considering storey drift, the frame with floating columns is the weakest since it suffers the maximum storey drift while the bas frame suffers the least storey drift.

Considering the storey shear,the frame with with 4th and 7th storey heavy suffers the maximum shear.

it can be inferred clearly that the frame with floating columns faces the worse scenario since it faces the maximum displacement and drift and is most prone to damages under this kind of loading.

While on the other hand, it can be seen that the base frame has least deflection and drift hence causing the minimum damage to the building.

CONCLUSION

In this project we have analyzed various structures having different irregularities but same dimensions of 10-storey building with the help of ETABS. We have analyzed each and every aspect of the building so as to check their behavior with different irregularities. These buildings are situated in the 4th zone region which is an earthquake prone area hence earthquake forces, the total dead and live load are calculated that are to be considered.

After knowing deflection, shear and drift of the building frame due to earthquake forces and the loads, the behavior of the buildings were examined and analyzed .The base frame came out with the most satisfactory results while on the other hand, the building with floating columns faces the worse scenario because of the maximum deflection and drift hence is most prone to damages under this kind of loading. The other buildings which had irregularities also showed unsatisfactory results to some extent.

This analysis proves that irregularities are always harmful for the structure and doesn't always gives the satisfactory result, hence as far as possible irregularities in the building must be avoided .But if irregularities have to be introduced for any reason they must be designed properly following the conditions of IS 1893 and IS 456.

APPENDIX-A

STAAD.pro listing of the program for analysis and design of the two storey building

STAAD SPACE

START JOB INFORMATION

ENGINEER DATE 18-Nov-10

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

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5 4.00001 4.00001 0; 6 4.00001 0 0; 7 0 0 4.00001; 8 0 4.00001 4.00001;

9 0 8.00002 4.00001; 10 4.00001 8.00002 4.00001; 11 4.00001 4.00001 4.00001;

12 4.00001 0 4.00001;

MEMBER INCIDENCES

1 1 2; 2 2 3; 3 3 4; 4 4 5; 5 5 6; 6 5 2; 7 2 8; 8 3 9; 9 4 10; 10 5 11;

11 7 8; 12 8 9; 13 9 10; 14 10 11; 15 11 12; 16 11 8;

DEFINE MATERIAL START

ISOTROPIC CONCRETE

E 2.17184e+007

POISSON 0.17

DENSITY 23.5615

ALPHA 5.5e-006

DAMP 0.05

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END DEFINE MATERIAL
MEMBER PROPERTY INDIAN
3 6 TO 10 13 16 PRIS YD 0.15 ZD 0.25
MEMBER PROPERTY INDIAN
1 2 4 5 11 12 14 15 PRIS YD 0.2
CONSTANTS
MATERIAL CONCRETE MEMB 1 TO 16
SUPPORTS
1 6 7 12 FIXED
DEFINE WIND LOAD
TYPE 1
INT 0.329922 0.329922 0.333774 0.33748 0.341051 0.344498 0.347832 0.351061 -
 0.354193 0.357234 0.360191 0.363068 0.365871 0.368605 0.371272 HEIG 0 -
4.57201 4.8357 5.09939 5.36309 5.62678 5.89047 6.15417 6.41786 6.68155 -
6.94525 7.20894 7.47263 7.73632 8.00002
*SEISMIC loads
DEFINE 1893 LOAD
ZONE 0.24 RF 3 I 1 SS 2 DM 0.05 DT 6
SELFWEIGHT
LOAD 1 LOADTYPE None  TITLE EL IN X DIRECTION
1893 LOAD X 1
LOAD 2 LOADTYPE None  TITLE EL IN Z DIRECTION
1893 LOAD Z 1
LOAD 3 LOADTYPE Dead  TITLE DEAD LOAD
MEMBER LOAD
```

3 6 TO 10 13 16 LIN Y 0 0 -2.48

3 6 TO 10 13 16 UNI GY -0.93

LOAD 4 LOADTYPE Live TITLE LIVE LOAD

MEMBER LOAD

3 6 TO 10 13 16 LIN Y 0 0 -1.5

LOAD 5 LOADTYPE Wind TITLE WIND LOAD IN X DIRECTION

WIND LOAD X 1 TYPE 1 YR 0 8 ZR 0 4

LOAD 6 LOADTYPE Wind TITLE WIND LOAD IN Z DIRECTION

WIND LOAD Z 1 TYPE 1 XR 0 4 YR 0 8

LOAD COMB 7 1.5(D.L+L.L)

3 1.5 4 1.5

LOAD COMB 8 1.2(D.L.+L.L.+E.L.-X)

3 1.2 4 1.2 1 1.2

LOAD COMB 9 1.2(D.L.+L.L.-E.L.-X)

3 1.2 4 1.2 1 -1.2

LOAD COMB 10 1.2(D.L.+L.L.+E.L.-Z)

3 1.2 4 1.2 2 1.2

LOAD COMB 11 1.2(D.L.+L.L.-E.L.Z)

3 1.2 4 1.2 2 -1.2

LOAD COMB 12 1.5(D.L.+E.L.X)

3 1.5 1 1.5

LOAD COMB 13 1.5(D.L.+E.L.Z)

3 1.5 2 1.5

LOAD COMB 14 1.5(D.L.-E.L.X)

3 1.5 1 -1.5

LOAD COMB 15 1.5(D.L.-E.L.Z)

3 1.5 2 -1.5

LOAD COMB 16 0.9D.L. + 1.5E.L.X

3 0.9 1 1.5

LOAD COMB 17 0.9D.L. - 1.5E.L.X

3 0.9 1 -1.5

LOAD COMB 18 0.9D.L. + 1.5E.L.Z

3 0.9 2 1.5

LOAD COMB 19 0.9D.L. - 1.5E.L.Z

3 0.9 2 -1.5

LOAD COMB 20 1.2(D.L. + L.L. + W.L.X)

3 1.2 4 1.2 5 1.2

LOAD COMB 21 1.2(D.L. + L.L. - W.L.X)

3 1.2 4 1.2 5 -1.2

LOAD COMB 22 1.2(D.L. + L.L. + W.L.Z)

3 1.2 4 1.2 6 1.2

LOAD COMB 23 1.2(D.L. + L.L. - W.L.Z)

3 1.2 4 1.2 6 -1.2

LOAD COMB 24 1.5(D.L. + W.L.X)

3 1.5 5 1.5

LOAD COMB 25 1.5(D.L. - W.L.X)

3 1.5 5 -1.5

LOAD COMB 26 1.5(D.L. + W.L.Z)

3 1.5 6 1.5

LOAD COMB 27 1.5(D.L. - W.L.Z)

```
3 1.5 6 -1.5
LOAD COMB 28 0.9D.L. + 1.5W.L.X
3 0.9 5 1.5
LOAD COMB 29 0.9D.L. - 1.5W.L.X
3 0.9 5 -1.5
LOAD COMB 30 0.9D.L. + 1.5W.L.Z
3 0.9 6 1.5
LOAD COMB 31 0.9D.L. - 1.5W.L.Z
3 0.9 6 -1.5
PERFORM ANALYSIS
LOAD LIST 7 TO 31
PRINT ANALYSIS RESULTS
START CONCRETE DESIGN
CODE INDIAN
CLEAR 0.025 MEMB 3 6 TO 10 13 16
CLEAR 0.03 MEMB 1 2 4 5 11 12 14 15
FC 25000 ALL
FYMAIN 415000 ALL
FYSEC 415000 ALL
MAXMAIN 20 MEMB 1 2 4 5 11 12 14 15
MAXMAIN 12 MEMB 3 6 TO 10 13 16
MINMAIN 10 MEMB 1 2 4 5 11 12 14 15
MINMAIN 8 MEMB 3 6 TO 10 13 16
MAXSEC 10 ALL
MINSEC 8 ALL
```

```
REINF 0 ALL
TRACK 2 ALL
DESIGN BEAM 3 6 TO 10 13 16
DESIGN COLUMN 1 2 4 5 11 12 14 15
CONCRETE TAKE
END CONCRETE DESIGN
FINISH
```

APPENDIX-B

Seismic Response of Building Frames with Vertical Structural Irregularities

By Eggert V. Valmundsson[1] and James M. Nau,[2] Member, ASCE

Abstract: Earthquake design codes require different methods of analysis for regular and irregular structures, but it is only recently that codes have included specific criteria that define irregular structures. In this paper, the mass, strength, and stiffness limits for regular buildings as specified by the Uniform Building Code (UBC) are evaluated. The structures studied are two-dimensional building frames with 5, 10, and 20 stories. Six fundamental periods are considered for each structure group. Irregularities are introduced by changing the properties of one story or floor. Floor-mass ratios ranging from 0.1 to 5.0 are considered, and first-story stiffness and strength ratios varying from 1.0 to 0.5 are included. The response is calculated for design ductility levels of 1 (elastic), 2, 6, and 10 for four earthquake records. Conclusions are derived regarding the effects of the irregularities on shear forces and maximum ductility demands. It is found that the mass and stiffness criteria of UBC result in moderate increases in response quantities of irregular structures compared to regular structures. The strength criterion, however, results in large increases in response quantities and thus is not consistent with the mass and stiffness requirements. Based on these findings, several modifications to the criteria are proposed, which include a revised formula for estimating the fundamental period for buildings with nonuniform distributions of mass.

INTRODUCTION

In most earthquake design codes, the equivalent lateral force (ELF) approach is used to establish design forces. The ELF procedure is based on several assumptions that are reasonable for most regular structures, namely, those without major discontinuities in mass, stiffness, and strength over the height. It is necessary, therefore, to develop criteria to determine when the ELF method can be applied to irregular structures, without reducing the level of safety. Although these requirements have been recognized for years, it was not until recently that building codes have quantified maximum allowable limits on the irregularities that a structure can have to base its design on ELF methods. The first such criteria in the Uniform Building Code (UBC) were published in its 1988 edition. These criteria are based on the 1988 edition of the so-called "Blue Book" (*Recommended* 1990). These limits are somewhat arbitrary and were inserted for completeness of the code. As pointed out by Porush (1989), "It is true that research is needed to verify these limits. However, without such limits there cannot be unambiguous enforceable provisions."

The objective of the present paper is to evaluate the definitions of regular and irregular structures for the three vertical irregularities involving mass, stiffness, and strength. This objective will be accomplished by calculating the time history (TH) of the elastic response and comparing it to the response predicted by ELF methods, and by comparing actual ductility demands from inelastic analysis to the design ductilities. Structures with 5, 10, and 20 stories are considered with design ductilities of 1 (elastic), 2, 6, and 10.

BACKGROUND

In the ELF procedure, two important assumptions are made. First, the ELF approach is based on linear analysis, with the effects of yielding approximated by an elastic spectral acceleration reduced by a modification factor. Second, it is assumed that a linear lateral-force distribution is a reasonable and conservative representation of the actual dynamic response. These assumptions are satisfactory for regular structures. For structures with irregular vertical configurations, however, these assumptions (particularly the second) may no longer apply, and loads and deformations may be significantly different from those predicted by the ELF procedure (*Recommended* 1991). Therefore, the ELF approach is strictly applicable only to regular structures, and it is necessary to develop rules to determine when it can be used. According to UBC, the ELF method may be used for

1. All structures, regular and irregular, less than five stories or 19.8 m (65 ft) tall.
2. Regular structures less than 73.2 m (240 ft) tall.

With some exceptions, dynamic analysis must be used for irregular buildings and for regular buildings exceeding 73.2 m (240 ft) in height. A structure is considered to be irregular if it has significant physical discontinuities in its configuration or in its lateral-force resisting system. According to UBC, structures with mass, strength, and stiffness irregularities are as follows.

1. Weight (mass) irregularity: Mass irregularity is considered to exist where the effective mass of any story is more than 150% of the effective mass of an adjacent story. A roof that is lighter than the floor below need not be considered.
2. Stiffness irregularity—soft story: A soft story is one in which the lateral stiffness is less than 70% of that of the story above or less than 80% of the average stiffness of the three stories above.
3. Discontinuity in capacity–weak story: A weak story is one in which the story strength is less than 80% of that in the story above. The story strength is the total strength of all seismic-resisting elements sharing the story shear for the direction under consideration.

The purpose of the present study is to investigate the appropriateness of the numerical limits of these definitions. For example, is a structure with a mass discontinuity, as previously defined, actually irregular, i.e., do the true story forces differ markedly from those predicted by ELF? In addition, the consistency of these requirements is examined. That is, do the

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Note. Associate Editor: David Bonneville. Discussion open until June 1, 1997. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on May 25, 1995. This paper is part of the ***Journal of Structural Engineering***, Vol. 123, No. 1, January, 1997. ©ASCE, ISSN 0733-9445/97/0001-0030–0041/$4.00 + $.50 per page. Paper No. 10825.

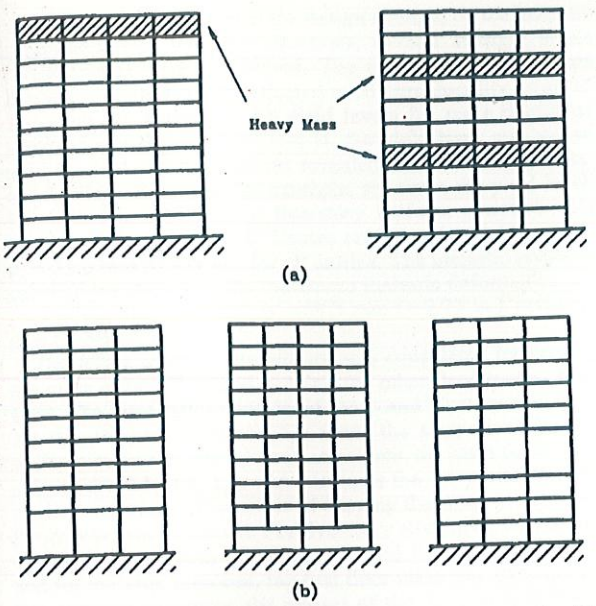

FIG. 1. Vertical Structural Irregularities: (a) Mass Ratio; (b) Stiffness Ratio

specified limits for mass, stiffness, and strength discontinuities produce comparable changes in response when compared to regular structures?

Examples of mass and stiffness irregularities are shown in Fig. 1. The mass of each story is the dead load plus an appropriate portion of the live load. Since the mass of a story is largely dead load from the floor structure, a discontinuity is usually due to a different use of one floor compared to other floors in the structure (parking floor, mechanical floor, etc.). Although UBC specifies independent stiffness and strength criteria to determine if a structure is regular, it is important to note that in many practical cases strength changes with stiffness. For example, decreasing the moment of inertia reduces both the stiffness and strength of the member. Reducing the number of members, as illustrated in Fig. 1(b) where two of the columns terminate at the second floor, may also reduce both the strength and stiffness. Stiffness and strength can also be unintentionally increased by nonstructural elements.

OUTLINE OF STUDY

The structures considered in the present study are framed buildings with heights of 5, 10, and 20 stories. It should be noted that framed construction may not be appropriate for all 10-story–20-story buildings, especially in high seismic zones. Many modern buildings in this range of height are designed with shear walls for the lateral-load-resisting system. The structural models and parameters used in this study do not represent shear-wall buildings per se. The plane frames are idealized as lumped mass systems with one degree of freedom per floor, the simplest dynamic model that building codes permit. For this study, the beams are assumed to be much stiffer than the columns. This simple two-dimensional model was adopted in the interest of conducting an extensive parametric study. Considering all combinations of mass, stiffness, and strength ratios, building heights, fundamental periods, and ground motions, more than 5,000 cases are included in the results presented in this paper.

While the shear building model represents a major simplification, it is adequate for the determination of *overall* structural response, as shown by Cruz and Chopra (1986). Cruz and Chopra considered uniform plane frames with 5 and 20 stories and 5-story frames with various distributions of mass and stiffness over the height. Their results showed that *overall* response, as characterized by top-floor displacement, base shear, and overturning moments, does not differ markedly in frames for which $I_b/I_c = \infty$ (the shear building) and in frames in which $I_b/I_c = 0.5$, where I_b = moment inertia of the beams; and I_c = moment of inertia of the columns. *Local* response, however, as exemplified by beam moment, column moment, and column axial force is strongly influenced by the beam-to-column stiffness ratio.

For each structure height, six uniform structures with constant mass, stiffness, and strength, are considered. In most practical designs, the stiffness and strength decrease with height. However, in this investigation, the uniform buildings represent the reference cases for the parametric study. For example, to judge the adequacy of the ELF procedure for an irregular building, the results are compared to those for the uniform building. The floor mass was taken as 35 Mg (0.2 kip-s²/in.), and the stiffnesses were calculated to give a set of desired fundamental periods. To establish upper and lower bounds for the fundamental periods, values obtained from measured accelerograph records during the 1971 San Fernando earthquake were considered (*Recommended* 1991). For steel framed structures, the average fundamental period T_{avg} and lower bound T_{lb} are given by

$$T_{avg} = 0.119h_n^{3/4} \text{ and } T_{lb} = 0.0853h_n^{3/4} \tag{1}$$

where h_n = total building height in meters. For concrete framed structures, the average and lower bound are given by

$$T_{avg} = 0.0853h_n^{3/4} \text{ and } T_{lb} = 0.0609h_n^{3/4} \tag{2}$$

The average story heights for these structures were 4.0 m (13.0 ft) for the steel frames and 3.0 m (9.7 ft) for the concrete frames. In this study, an average story height of 3.66 m (12.0 ft) was selected, and the fundamental periods for the uniform structures were limited to

$$0.0609h_n^{3/4} < T < 0.119h_n^{3/4} \tag{3}$$

Table 1 gives the properties of the uniform structures considered in this study. To ensure that the stiffnesses were appropriate for the class of buildings considered, the interstory drifts were evaluated according to UBC requirements for moderate seismic conditions. Drifts were computed for a zone factor $Z = 0.3$, importance factor $I = 1$, site coefficient $S = 1.2$, and response modification factor $R_w = 6$. While UBC places some restrictions on the types of structural systems for the various earthquake zones, table 16-N of UBC shows that R_w is at least 6 for the frame systems permitted in seismic zones 3 and 4. The maximum calculated drifts from the lateral design forces for the regular structures are 0.0021 for 5 stories ($T = 1$ s), 0.0025 for 10 stories ($T = 1.8$ s), and 0.0025 for 20 stories ($T = 2.9$ s). Thus, all drifts for the regular structures are within the UBC limit of 0.004.

TABLE 1. Mass, Stiffness, and Fundamental Periods for Uniform Structures

Structure (1)	Quantity (2)	Period Number 1 (3)	2 (4)	3 (5)	4 (6)	5 (7)	6 (8)
5-story 18.3 m	Mass (Mg)	35.0	35.0	35.0	35.0	35.0	35.0
	Stiffness (MN/m)	68.3	47.5	34.9	26.6	21.0	17.1
	Period (s)	0.5	0.6	0.7	0.8	0.9	1.0
10-story 36.6 m	Mass (Mg)	35.0	35.0	35.0	35.0	35.0	35.0
	Stiffness (MN/m)	96.7	61.8	42.9	31.5	24.2	19.1
	Period (s)	0.8	1.0	1.2	1.4	1.6	1.8
20-story 73.2 m	Mass (Mg)	35.0	35.0	35.0	35.0	35.0	35.0
	Stiffness (MN/m)	120	81.6	58.8	44.5	34.9	28.0
	Period (s)	1.4	1.7	2.0	2.3	2.6	2.9

To assess the influence of the design strength on the inelastic response of the irregular structures, several strength levels (yield levels) were considered. The story yield levels were determined to provide preselected maximum ductility levels of 2, 6, and 10. Obtaining the yield levels for these ductilities involved a trial procedure, that is, the yield level was varied until the inelastic calculations revealed that the maximum inelastic displacement is, for example, six times the yield level. For instance, consider the five-story uniform structure with $T = 1$ s subjected to the El Centro record and a yield level of $u_y = 11.6$ mm (0.455 in.) for all stories. The inelastic response calculations show that the maximum inelastic deformation occurs in the first story and is -69.3 mm (-2.73 in.), and thus $\mu = 69.3/11.6 = 6$.

The effect of mass irregularity was considered by varying the mass of one floor and keeping the other floor masses constant. The mass ratios considered for 5- and 10-story structures are 0.1, 0.5, 1.5, 2.0, and 5.0 times the story mass for the uniform structure. For 20-story structures, the mass ratios considered are 0.1, 0.5, 1.5, and 5.0 times the story mass for the uniform structure. The effect of varying the mass of different floors was also evaluated. For five-story structures, the masses of the third and fifth floors were varied for elastic response, and for inelastic response, the first floor mass was also altered. For 10-story structures, the masses of the 5th and 10th floors were varied for elastic response, and also at the first floor for inelastic response. For 20-story structures, the 14th and 20th floor masses were varied for elastic response. For inelastic response, the masses at the 1st, 10th, and 20th floors were varied. As an example, Fig. 2 shows the variable masses for the 10-story structures.

The effect of stiffness irregularity was considered by varying the stiffness of the first story only, since it was considered the most severe case. The stiffness of the first story was reduced to 90, 80, 70, 60, and 50% of the stiffness of the first story of the uniform structure while the strength was held constant. This can be seen in Fig. 3(a). In this figure, k_u denotes the first story stiffness of the uniform structure, and k_n is the reduced first-story stiffness of the nonuniform structure. The yield displacement u_y is denoted in the same manner.

To determine if the designs of the irregular structures were consistent with UBC requirements, the drifts were *estimated* for the mass and stiffness ratios previously listed. For these *estimates*, the forces from the UBC equivalent lateral-force procedure for the design conditions described previously for regular structures were used. The results show that the maximum drift for any structure with a mass ratio of 5.0 is 0.0037. For the smallest stiffness ratios of 0.5 and 0.6, however, the drifts exceed the UBC limit, i.e., the maximum drift for a stiffness ratio of 0.5 is 0.0050. While these drifts exceed the limit of 0.004, it should be noted that the stiffness ratios of 0.5 and 0.6 are much lower than the 80% limit UBC places on regular structures.

The effect of strength irregularity was investigated by varying the strength (yield level) of the first story, again because

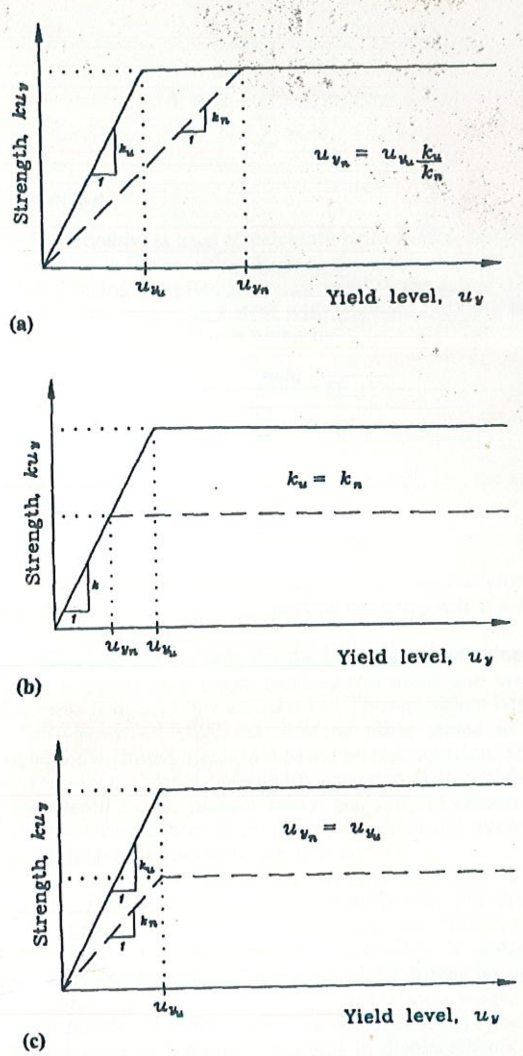

FIG. 3. **Stiffness and Strength Variation: (a) Decreasing Stiffness, Constant Strength; (b) Decreasing Strength, Constant Stiffness; (c) Decreasing Stiffness and Strength**

it was considered the most severe yet practical case. The first-story strength was reduced to 90, 80, 70, 60, and 50% of the strength for the uniform case. This can be seen in Fig. 3(b). The stiffness was kept the same as for the uniform structure. The relationship between strength and stiffness was investigated by reducing the first story stiffness to 90, 80, 70, 60, and 50% of the stiffness of the first story of the uniform structure and by decreasing the story strength proportionately, as illustrated in Fig. 3(c).

The structures in this study were subjected to four earthquakes.

1. Pacoima Dam record of February 9, 1971, S16E component.
2. Parkfield record of June 27, 1966, N65E component.
3. El Centro record of May 18, 1940, S00E component.
4. Taft record of July 21, 1952, S69E component.

A 2 s pulse was added to the earthquake records to account for the effects arising from the ground motion lost before the recording instrument was triggered. The prefixed pulse was developed by Pecknold and Riddell (1978), and a detailed dis-

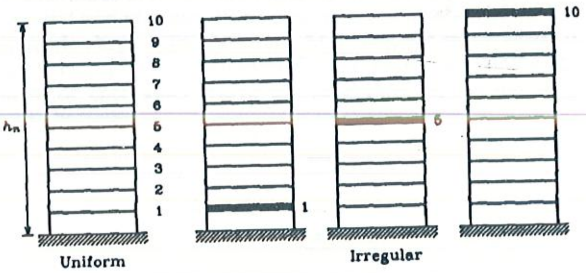

FIG. 2. **Mass Variation for 10-Story Structure**

cussion and formulation can be found in their paper. The earthquake records were not altered or scaled in any manner.

METHODS OF ANALYSIS

Introducing structural irregularities has two major effects on the dynamic response: the fundamental period shifts and the mode-shape changes. Because of the peaks and valleys in the earthquake response spectrum, a shift in the period results in a different spectral acceleration, S_a. This means that results obtained from different methods of analysis can differ solely due to the difference in S_a. It can also be expected that this difference will increase as more extreme irregularities are introduced, since the differences between the shifted and unshifted periods will be larger. The effects of this period shift are examined in greater detail later in this paper.

To investigate the effects of the different approximations made in the derivation of the ELF method, the response was calculated using three different ELF approaches. These three ELF methods are compared with the results from TH analysis. Thus, four methods of analysis are used.

Time History

This entails TH response using direct integration. Results from this method are considered "exact" since the actual histories of displacements and forces are calculated.

ELF$_R$ Method

The ELF method is used to calculate shear forces using the actual first mode shape and S_a for the true (shifted) fundamental period. ELF$_R$ is the best estimate that a first-mode approximation can provide. For the ith story, the maximum inertia force is

$$f_{s,i} = m_i \phi_i \Gamma S_a \tag{4}$$

where m_i = mass of the ith story; ϕ_i = component of the fundamental mode shape for the ith story; S_a = pseudospectral acceleration; and Γ = modal participation factor given by

$$\Gamma = \frac{\sum_{i=1}^{N} m_i \phi_i}{\sum_{i=1}^{N} m_i \phi_i^2} \tag{5}$$

The base shear V is

$$V = \sum_{i=1}^{N} f_{s,i} = \Gamma S_a \sum_{i=1}^{N} m_i \phi_i = S_a \frac{\left(\sum_{i=1}^{N} m_i \phi_i\right)^2}{\sum_{i=1}^{N} m_i \phi_i^2} \tag{6}$$

ELF$_L$ Method

The ELF method is used to calculate shear forces, assuming the first mode is linear and using S_a for the true (shifted) fundamental period. Thus, ELF$_L$ enables the evaluation of the linear mode-shape approximation. Assuming the mode shape is linear, i.e., $\{\phi_i\}$ is proportional to a vector containing the height of each story above the base $\{h_i\}$, (4) for the story forces becomes

$$f_{s,i} = S_a m_i h_i \frac{\sum_{i=1}^{N} m_i h_i}{\sum_{i=1}^{N} m_i h_i^2} \tag{7}$$

and the base shear is

$$V = S_a \frac{\left(\sum_{i=1}^{N} m_i h_i\right)^2}{\sum_{i=1}^{N} m_i h_i^2} \tag{8}$$

ELF$_C$ Method

The ELF method is used to calculate shear forces, assuming the first mode is linear, using S_a from the original (unshifted) period and taking the effective mass equal to the actual mass. ELF$_C$ gives the forces consistent with standard building code practice. The story forces are given by

$$f_{s,i} = S_a \frac{m_i h_i}{\sum_{i=1}^{N} m_i h_i} M \tag{9}$$

where $M = \Sigma m_i$ = total actual mass; and $h_i = ih$, i.e., the story heights are equal. The base shear is

$$V = S_a M = (2\pi/T) S_v M \tag{10}$$

where $S_a = \omega S_v = (2\pi/T) S_v$ in which ω = fundamental circular natural frequency; S_v = pseudospectral velocity; and T = fundamental natural period.

Eq. (10) gives the formula for the base shear, assuming the structure responds in a linear fundamental mode and the effective mass is equal to the actual mass. The equivalent lateral-force provisions of UBC, as well as other codes in use throughout the United States, are based on this equation. These codes introduce a variety of modifications to (10), which are not considered in the present study. Eq. (10) represents the extent of the application of the theory of structural dynamics to the equivalent lateral-force procedure.

For the elastic calculations, the irregularities considered will affect the story shear forces and displacements. For the inelastic structures, the base shear forces will not change, since an elastoplastic force-deformation relationship is assumed. However, the magnitudes of the story shear forces on upper floors, which typically remain within the elastic range, will change somewhat. For the inelastic calculations, the effects of the irregularities will produce changes in displacements and, consequently, changes in ductility demands. The objective, again, is to evaluate the magnitude of the changes in response that the various irregularities produce.

RESULTS

Mass Irregularity

The ratio of the base shear V calculated from each of the ELF methods to the time history was determined. This ratio gives some measure of the accuracy of each ELF procedure compared to TH results. The error in the base shear was calculated as follows:

$$\text{Error} = (\text{ELF} - \text{TH})/\text{TH} \times 100\% \tag{11}$$

The minimum error in the shear force *in any story* was also determined for the ELF$_C$ method. This *minimum* error shows how much the ELF$_C$ procedure can *underestimate* the actual shear force in any story. Finally, average errors in base shear for all ELF methods and the average minimum error in any story for ELF$_C$ were also calculated. These errors will be clarified in the following discussion.

5-Story Structures

For the elastic calculations, the ratios of the base shears determined from the different ELF methods to the TH results

TABLE 2. Ratio of Base Shear Forces and Errors for Five-Story Structures with Mass Irregularity

Period (s) (1)	Method (2)	0.1 AVG (3)	0.1 STD (4)	0.5 AVG (5)	0.5 STD (6)	1.0 AVG (7)	1.0 STD (8)	1.5 AVG (9)	1.5 STD (10)	2.0 AVG (11)	2.0 STD (12)	5.0 AVG (13)	5.0 STD (14)
	(*a*) Mass ratio at fifth floor												
0.5	ELF_R	1.01	0.03	0.99	0.03	0.99	0.06	0.97	0.02	0.98	0.01	0.96	0.05
	ELF_L	0.94	0.02	0.91	0.03	0.92	0.06	0.91	0.02	0.93	0.00	0.94	0.05
	ELF_C	1.04	0.33	0.95	0.14	1.13	0.07	1.22	0.18	1.44	0.54	1.66	0.21
	Minimum %	1	32	−7	14	7	4	22	18	40	47	65	22
0.6	ELF_R	0.99	0.06	0.97	0.02	0.98	0.02	0.99	0.06	0.96	0.06	0.95	0.02
	ELF_L	0.92	0.05	0.89	0.02	0.91	0.02	0.93	0.06	0.91	0.06	0.94	0.02
	ELF_C	0.94	0.25	1.01	0.23	1.11	0.03	1.20	0.14	1.28	0.23	1.89	0.97
	Minimum %	−8	29	1	23	3	15	7	19	19	24	85	97
0.7	ELF_R	0.99	0.01	0.99	0.04	0.97	0.07	0.97	0.06	1.00	0.06	0.97	0.02
	ELF_L	0.91	0.01	0.91	0.04	0.91	0.06	0.91	0.05	0.95	0.06	0.95	0.02
	ELF_C	0.95	0.10	1.01	0.10	1.11	0.08	1.20	0.22	1.30	0.52	2.01	1.03
	Minimum %	−11	7	−10	7	0	14	13	22	26	52	88	100
0.8	ELF_R	0.99	0.06	0.96	0.08	0.98	0.04	0.99	0.03	0.96	0.01	0.94	0.08
	ELF_L	0.92	0.06	0.88	0.07	0.91	0.04	0.93	0.03	0.91	0.01	0.92	0.08
	ELF_C	1.01	0.28	1.09	0.23	1.12	0.05	1.19	0.22	1.35	0.35	1.84	0.51
	Minimum %	−12	18	4	24	8	2	15	19	27	30	80	48
0.9	ELF_R	0.97	0.06	0.98	0.04	0.98	0.04	0.96	0.02	1.00	0.01	0.95	0.03
	ELF_L	0.90	0.06	0.90	0.03	0.91	0.04	0.90	0.02	0.95	0.01	0.93	0.03
	ELF_C	1.02	0.30	0.99	0.15	1.12	0.04	1.23	0.22	1.40	0.30	1.92	0.79
	Minimum %	−1	30	−2	16	6	3	14	8	21	10	89	79
1.0	ELF_R	0.99	0.04	0.98	0.04	0.97	0.03	1.02	0.04	1.02	0.06	0.97	0.06
	ELF_L	0.92	0.04	0.91	0.04	0.90	0.03	0.96	0.04	0.97	0.05	0.95	0.06
	ELF_C	0.90	0.24	1.02	0.16	1.10	0.03	1.32	0.14	1.38	0.19	1.66	0.66
	Minimum %	−11	25	−4	16	1	12	7	15	20	28	56	70
[Average]	ELF_R %	−1	1	−2	1	−2[a]	1	−2	2	−1	5	−4	1
	ELF_L %	−8	1	−10	1	−9[b]	1	−8	2	−6	5	−6	1
	ELF_C %	−2	5	1	4	11[c]	1	23	4	36	5	83	13
	Minimum %	−7	5	−3	5	4[d]	3	13	5	25	7	77	12
	(*b*) Mass ratio at third floor												
[Average]	ELF_R %	−7	4	13	8	−2	1	−1	3	−1	1	0	1
	ELF_L %	−13	4	5	7	−9	1	−8	2	−7	1	−6	1
	ELF_C %	3	8	7	3	11	1	11	3	23	3	57	7
	Minimum %	−6	10	1	4	4	3	4	3	15	6	53	7

Note: AVG = average; STD = standard deviation.
[a]Underestimates the base shear by 2%.
[b]Underestimates the base shear by 9%.
[c]Overestimates the base shear by 11%.
[d]Average minimum error of the story shear forces.

are presented in Table 2. The averages (AVG) and standard deviations (STD) of the results for the four earthquakes for each structure are shown. Mass ratios change with columns of the table and fundamental periods with rows. The periods listed are the fundamental periods of the uniform structures. A significant change with period in the ratio of the response predicted by the ELF methods and TH could not be inferred; therefore, the results were averaged over all periods and shown at the bottom of Table 2. The values presented are average errors in percent.

Positive errors represent overestimates, and negative errors indicate underestimates. For the uniform case (mass ratio = 1.0), the determination of the base shear is quite accurate. The ELF procedure using the real first mode, ELF_R, underestimates the base shear by 2%. The linear mode shape, ELF_L, underestimates it by 9%, but the linear mode and "original" period (the period for the uniform structure), ELF_C, overestimate it by 11%. This difference arises because ELF_R and ELF_L use the effective mass associated with the mode shape while ELF_C uses the actual mass, which exceeds the effective mass by 14% for the uniform five-story structures. The average minimum error of the story shear forces for all periods was 4%, although an underestimate of −23% (not shown in the table) was calculated for one earthquake. This finding shows that the scatter in seismic response results is generally large. The standard deviation for the uniform structures is quite small, varying from 0.02 to 0.08, which suggests that the response of the structure is largely in the first mode. When the top-mass ratio is 0.1, the average of the base-shear response using ELF_R and ELF_L does not change, but the ELF_C and minimum error decrease by roughly 10 percentage points from the uniform case. The standard deviation for ELF_C increases but remains the same for the other two methods. This result is expected and is primarily due to the change in the ordinate of response spectrum S_v associated with the period shift. When the top-mass ratio is greater than one, the overestimate of the ELF_C method increases, but the other methods do not give significantly different results. For example, for a mass ratio of 5.0, ELF_R underestimates the shear force by 4%, ELF_L by 6%, but ELF_C overestimates it by 83%. The same trends were observed when the mass of the third floor was varied, as shown in the average results at the bottom of Table 2, but the effects are not as great as for the top-mass variation. On the basis of this result, it was decided not to vary the mass of the first floor.

The results of the inelastic calculations are shown in Fig. 4 for design ductilities of 2, 6, and 10. Each plot shows the effect on ductility demand when the mass is altered on the first, third, and fifth floors. The results are represented as ratios of the ductility demand for the irregular structure to the design ductility for the regular structure. As described previously, the calculations were made by determining the yield levels required to give the desired design ductility for the uniform structure. The irregularity was then introduced, and the ductility demand was calculated from inelastic analysis using the yield displacement for the uniform structure. An example is shown in Table 3, which gives the results for a five-story struc-

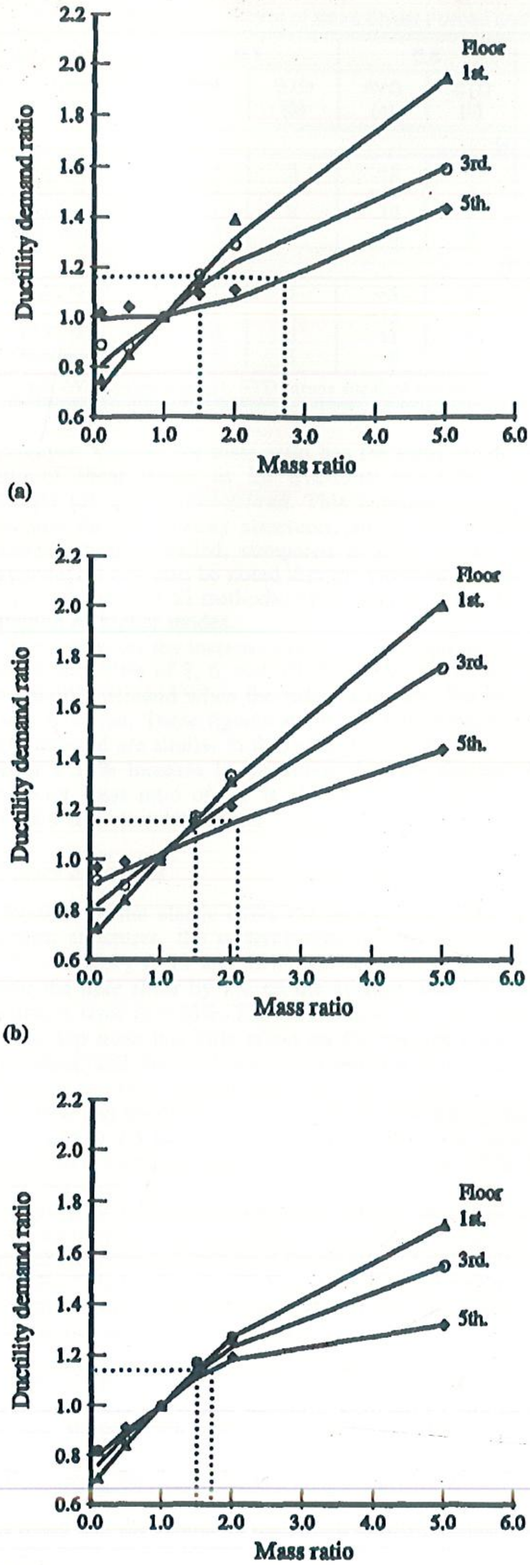

FIG. 4. Maximum Ductility Demand for 5-Story Structures with Mass Variation: (a) Design Ductility = 2; (b) Design Ductility = 6; (c) Design Ductility = 10

TABLE 3. Ductility Demand for Five-Story Structure with Third-Floor Mass Ratio of 1.5 and Design Ductility of 6

Earthquake (1)	Story (2)	Period 0.5 (3)	0.6 (4)	0.7 (5)	0.8 (6)	0.9 (7)	1.0 (8)
Pacoima Dam	1	7.77	8.04	6.65	7.04	6.76	6.88
	2	3.18	4.56	4.75	2.16	1.84	1.71
	3	1.54	1.27	1.28	1.34	1.62	1.62
	4	1.04	1.11	1.14	1.00	1.01	1.14
	5	0.75	0.69	0.77	0.72	0.63	0.77
[Maximum]		7.77	8.04	6.65	7.04	6.76	6.88
El Centro	1	7.12	5.99	4.55	7.33	7.60	7.31
	2	1.72	2.34	2.03	3.78	3.05	3.40
	3	1.10	1.17	1.36	2.17	1.47	1.63
	4	0.91	0.99	0.98	1.15	1.15	1.11
	5	0.56	0.74	0.73	0.79	0.80	0.74
[Maximum]		7.12	5.99	4.55	7.33	7.60	7.31
Taft	1	7.10	6.72	7.42	6.93	7.08	6.13
	2	4.29	1.55	2.00	1.40	2.18	2.56
	3	2.16	1.38	1.41	1.39	1.80	2.00
	4	1.02	0.98	0.97	1.08	1.31	1.26
	5	0.67	0.60	0.59	0.74	0.81	0.81
[Maximum]		7.10	6.72	7.42	6.93	7.08	6.13
Parkfield	1	7.77	7.01	6.17	6.64	6.80	7.99
	2	1.42	1.36	1.85	3.37	3.38	2.28
	3	1.08	1.17	1.04	1.46	2.40	2.13
	4	0.91	0.91	0.92	1.00	1.18	1.21
	5	0.55	0.55	0.63	0.69	0.72	0.75
[Maximum]		7.77	7.01	6.17	6.64	6.80	7.99
[Average of maximum values]		7.44	6.94	6.20	6.98	7.06	7.08
[Total average]							6.95
[Standard deviation]							0.86

ture with a third-floor mass ratio of 1.5 and a design ductility of 6. The ductility demand at every floor, for the six periods and four earthquakes, is shown. A maximum ductility demand for every structure is found and averaged for the four earthquakes. Finally, an average for all periods is calculated (called total average in the table). The ductility demand is 6.95, and the ductility demand ratio is 6.95/6 = 1.16. This point can be found in Fig. 4(b).

The results in Fig. 4 for the uniform structures are the points corresponding to a mass ratio of one and a ductility-demand ratio of one. From the plots, it is noted that the ductility demand changes most when the mass on the first floor is varied, and the relationship between ductility demand and mass ratio is approximately linear. When the mass ratio of the first floor is 1.5, the maximum ductility demand is increased by almost 20% (shown with dotted lines) and is essentially independent of design ductility. It can also be noted that to obtain a similar increase in ductility demand by varying the fifth-floor mass, a mass ratio exceeding 2 is required for design ductilities up to 6.

From these results, it can be noted that the elastic response is affected most by a mass discontinuity located high in a building, but inelastic response is influenced primarily by mass discontinuities located on lower floors.

10-Story Structures

Results for the errors in shear forces for the elastic calculations can be found in Table 4. Only average errors are shown in this table. For the uniform structures (mass ratio = 1.0), the average underestimate of the base shear using ELF_R is similar to that for the five-story structures, or 3%. ELF with the linear mode shape, ELF_L, produces an average underestimate of 10%. This suggests that most of the response is still in the first mode and that the linear mode assumption is still valid. The average overestimate of ELF_C is 15%, and the minimum error is −3%. Compared with the five-story structures, this result indicates that the force distribution is becoming more nonlinear, i.e., the overestimate of the base shear increases but the minimum error

TABLE 4. Errors of Base Shear Forces and Minimum Error for 10-Story Structures with Mass Irregularity

Method (1)	0.1		0.5		1.0		1.5		2.0		5.0	
	AVG (2)	STD (3)	AVG (4)	STD (5)	AVG (6)	STD (7)	AVG (8)	STD (9)	AVG (10)	STD (11)	AVG (12)	STD (13)
(a) Mass ratio at 10th floor												
ELF_R %	−5	3	−5	4	−3	3	−4	3	−5	4	−6	6
ELF_L %	−12	3	−12	3	−10	3	−11	3	−11	3	−10	5
ELF_C %	7	9	10	7	15	4	19	4	26	8	64	17
Minimum %	−8	9	5	9	−3	9	1	7	6	8	40	12
(b) Mass ratio at 5th floor												
ELF_R %	−7	5	−5	4	−3	3	−2	3	−3	3	−2	3
ELF_L %	−13	4	−12	4	−10	3	−10	3	−10	2	−10	3
ELF_C %	10	8	11	6	15	4	16	3	17	4	35	13
Minimum %	−4	8	−4	9	−3	9	−2	9	0	7	23	9

Note: AVG means average; STD means standard deviation.

decreases. Varying the mass ratio has the same effect on the ratio of shear forces as for five-story structures, but the changes are not as pronounced. This is probably due to the fact that for the 10-story structures, about 10% of the total structure mass is varied, compared to 20% for the 5-story structures. It can also be noted that the standard deviation increases slightly for all methods, which may be due to the contribution of higher modes.

The results for the inelastic cases can be found in Fig. 5 for design ductilities of 2, 6, and 10. Each plot shows the effect on ductility demand when the mass at the 1st, 5th, and 10th floors is varied. These figures show that the changes in ductility demand are similar to those for the five-story structures. About a 10% increase in maximum ductility demand for a first-floor mass ratio of 1.5 is observed, compared to about 20% for five-story structures.

20-Story Structures

Results for the elastic cases can be found in Table 5. For uniform structures, the underestimate of the base shear by ELF_R and ELF_L is 11 and 18%, respectively. ELF_C overestimates the base shear by 7% on the average, and the average minimum error is −26%. The results also show that decreasing the top mass has little effect on the average ratio of the base shear, and the minimum error remains about the same. When the top-mass ratio is increased, the overestimate of the base shear and the minimum error increase. Increasing the top-mass ratio to 1.5 has a negligible effect; however, when the mass ratio is increased above 1.5, the overestimate of the base shear increases.

Varying the mass on the 14th floor has the same effect. For these 20-story structures, a large variation was noted in the error of story shear forces over the structure height calculated by ELF_C. This implies that ELF_C does not reproduce the actual force distribution well and that higher modes are contributing more to the response. Furthermore, it implies that the force distribution is not linear over the height of the structure. For this period range (1.4–2.9 s), the top force F_t introduced in UBC to account for higher modes is between 10 and 20% of the base shear, which results in a significant redistribution of the base shear.

The results for the inelastic cases are presented in Fig. 6. Comparing these results to those for 5- and 10-story structures, it is noted that the change in maximum ductility demand when the first-floor mass is varied is less for 20-story structures than for both 5- and 10-story buildings. Furthermore, varying the mass ratio of the 20th floor does not have any appreciable effect on the maximum ductility demand. It was noted that varying the mass ratio of upper floors changed the ductility demand of several floors below, but it had no effect on the maximum ductility demand, which usually occurs in the first floor.

Change in Period

When a mass (or stiffness) irregularity is introduced, the fundamental period of the structure shifts, i.e., the nonuniform structure does not have the same fundamental period as the uniform structure. For the mass and stiffness irregularities considered in this paper, the effect of mass variation on the period is greater, especially for the five-story structures. The building code formulas used to estimate the fundamental building period T, e.g., (1) and (2), apply strictly to regular structures. Hence, as illustrated in Table 2, the code-based ELF procedure, ELF_C, may result in a considerable overestimate of the base shear. For five-story structures with a mass ratio of 5.0, the average minimum error is +77% for the heavy mass at the top floor and +53% for the heavy mass at the third floor. If the period shift can be accounted for, these results can be substantially improved.

For the structures in this study, it was found that the relative shift in period Δ for a structure with a different mass at the ith floor could be reasonably estimated from

$$\Delta = \frac{3}{4}\left(\frac{M_{nu}}{M_u} - 1\right)\frac{i}{N} \tag{12}$$

where M_{nu} = total mass of the nonuniform structure; M_u = total mass of the uniform structure; and N = total number of stories. The period of the nonuniform structure T_{nu} can then be found from the period of the uniform structure T as follows:

$$T_{nu} = (1 + \Delta)T \tag{13}$$

A comparison of the fundamental periods estimated from this formula and the periods obtained from the eigenvalue solution, for a variation of the mass at the top floor, is shown in Table 6. In this table, the fundamental periods of the uniform structures (mass ratio of 1.0) are 0.5, 0.8, and 1.4 s for the 5-, 10-, and 20-story buildings, respectively. As shown, the fundamental period predicted by the formula in (13) is generally in good agreement with the results from the eigenvalue solution. This is especially true for the five-story structures where the period changes the most.

To demonstrate the effect of the shifted period and (13) for its estimation, the response of a five-story structure with a top-mass ratio of 5.0 and an original fundamental period of 0.9 s was considered. This original period corresponds to that for the uniform five-story structure. The actual fundamental period for this irregular structure is 1.437 s, and the estimated period from (13) is 1.440 s. The spectral accelerations, $S_a = (2\pi/T)S_v$, for the original and shifted periods for the four

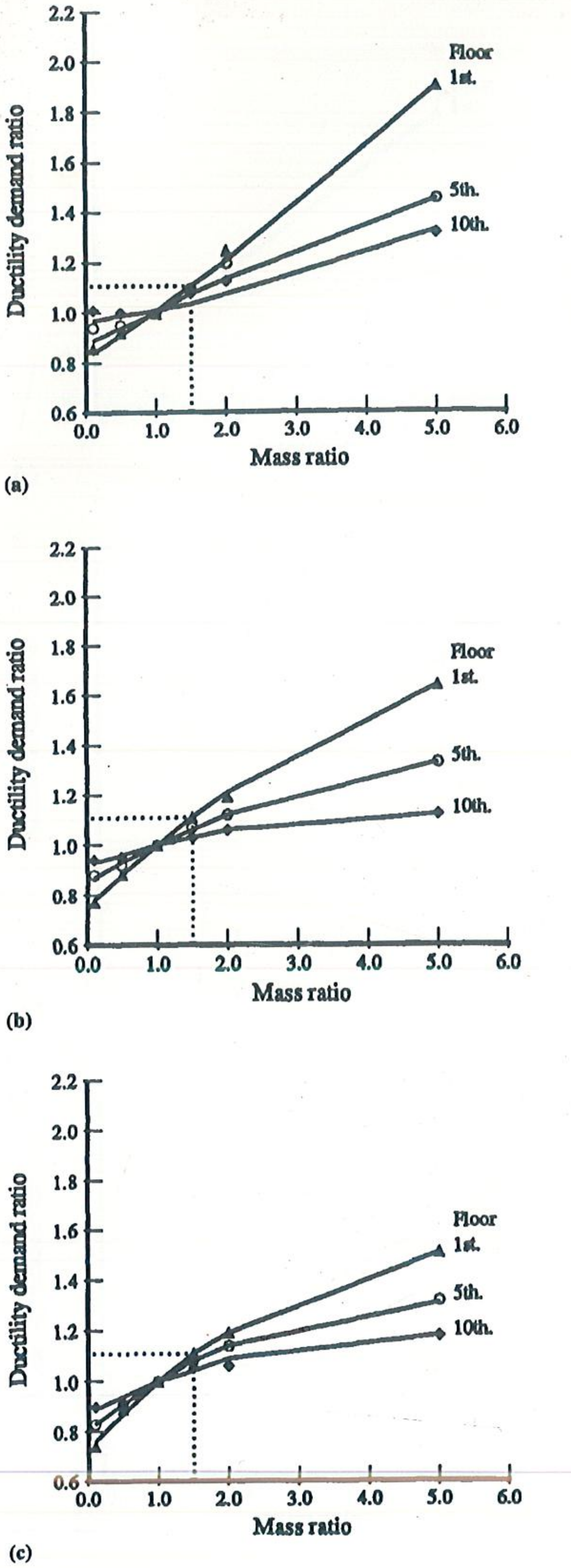

FIG. 5. Maximum Ductility Demand for 10-Story Structures with Mass Variation: (a) Design Ductility = 2; (b) Design Ductility = 6; (c) Design Ductility = 10

TABLE 5. Errors of Base Shear Forces and Minimum Error for 20-Story Structures with Mass Irregularity

Average (1)	0.1 AVG (2)	0.1 STD (3)	0.5 AVG (4)	0.5 STD (5)	1.0 AVG (6)	1.0 STD (7)	1.5 AVG (8)	1.5 STD (9)	5.0 AVG (10)	5.0 STD (11)
(a) Mass ratio at 20th floor										
ELF_R %	−11	4	−13	4	−11	3	−11	4	−13	4
ELF_L %	−18	4	−19	4	−18	3	−17	4	−18	4
ELF_C %	1	4	3	4	7	4	11	4	36	13
Minimum %	−28	10	−27	11	−26	12	−24	13	−5	17
(b) Mass ratio at 14th floor										
ELF_R %	−13	5	−12	4	−11	3	−11	4	−9	5
ELF_L %	−20	5	−19	4	−18	3	−17	4	−14	5
ELF_C %	1	7	4	6	7	4	10	4	31	13
Minimum %	−26	12	−26	12	−26	12	−24	12	−25	13

Note: AVG means average; STD means standard deviation.

earthquakes are shown in Table 7. Note that in all cases the spectral acceleration for the shifted period is less, and sometimes much less, than that for the original period. Hence, the ELF_C procedure that uses the unshifted period, i.e., the period for the uniform structure, overestimates the story shears. A comparison of the story shear forces from TH analysis and those predicted by the ELF_C procedure using both the original (unshifted) and the shifted fundamental periods are shown in Table 8 for the El Centro record. It is evident from these results that the difference in fundamental periods may have a great effect on the spectral acceleration, and hence on the story forces computed from the ELF procedure. However, by accounting for the period shift, the error in the elastic-force calculations can be decreased to a level similar to that for a uniform structure.

Stiffness and Strength Irregularities

Fig. 7 shows the change in the average maximum ductility demand and first-story drift, when the strength is kept constant and the stiffness reduced [see Fig. 3(a)]. These results are presented as ratios of drifts and ductilities for the irregular structure to the uniform structure. In all cases, the ductility demand decreases and displacements increase as stiffness is reduced. For a 30% reduction in stiffness, drifts increase by about 30% on average, and the ductility demand decreases by nearly 20%. This apparent anomaly may be explained by noting that as the structure becomes softer, drifts increase, but since the strength is constant, the yield displacement increases. Therefore, the ductility demand decreases. For a structure with a design ductility of 2, the increase in drifts is greater than for a structure with a design ductility of 6 or 10, but the decrease in ductility is not as great. This result is probably due to the fact that as the ductility increases, the maximum response occurs further into the inelastic branch of the force-displacement curve and is therefore not as sensitive to the slope of the elastic portion. There is also no significant difference in change in the response of structures with a design ductility of 6 and 10, which indicates that the maximum response of these structures is essentially independent of elastic stiffness.

For the strength irregularities [see Fig. 3(b)], the results were averaged in the same way as for the stiffness irregularities. The results are presented in Fig. 8. The ductility demand increases greatly for all structures, with greater increases for taller structures. For five-story structures, the increase in ductility demand does not change markedly with design ductility. However, the design ductility has some effect for the taller structures, where the ductility demand increases with the number of stories and decreases with larger design ductility. For a 20% decrease in strength, the increase in ductility demand is 80% for 5-story structures, 100–130% for 10-story structures, and 130–210% for 20-story structures, depending on design ductility.

TABLE 6. Comparison of Actual and Predicted Periods for Nonuniform Structures			
Story (1)	Mass ratio at top floor (2)	Shifted period (s) (3)	$(1+\Delta)T$ (s) (4)
5	0.1	0.42	0.43
5	0.5	0.45	0.46
5	1.0	0.50	0.50
5	1.5	0.54	0.54
5	2.0	0.59	0.57
5	5.0	0.80	0.80
10	0.1	0.73	0.75
10	0.5	0.76	0.77
10	1.0	0.80	0.80
10	1.5	0.84	0.83
10	2.0	0.87	0.86
10	5.0	1.08	1.04
20	0.1	1.34	1.35
20	0.5	1.37	1.37
20	1.0	1.40	1.40
20	1.5	1.43	1.43
20	2.0	1.47	1.45
20	5.0	1.66	1.61

TABLE 7. Comparison of Spectral Accelerations

Record (1)	S_a (m/s²) $T = 0.9$ s (2)	$T = 1.44$ s (3)
Pacoima	11.3	8.71
El Centro	5.26	1.81
Taft	2.36	1.22
Parkfield	5.28	4.98

TABLE 8. Comparison of Story Shear Forces for El Centro Record

Story (1)	Story Shear (kN) Time history (2)	ELF_C ($T = 0.9$ s) (3)	ELF_C ($T = 1.44$ s) (4)
1	520	1659	569
2	507	1610	556
3	463	1517	520
4	427	1374	472
5	374	1183	409

Fig. 9 shows the ductility demand when the strength is reduced proportionately to stiffness [see Fig. 3(c)]. It can be seen that the ductility demand increases markedly as the strength and stiffness are reduced. For a design ductility of 2 and a 30% decrease in strength and stiffness, the increase in ductility is 80% for 5-story structures, 130% for 10-story structures, and more than 200% for 20-story structures. The increase in ductility demand is generally less for a design ductility of 6 and 10. This increase in ductility demand is similar to that for a 20% decrease in strength (with constant stiffness), as shown in Fig. 8.

CONCLUSIONS

In this study, the earthquake response of 5-, 10-, and 20-story framed structures with nonuniform mass, stiffness, and strength distributions has been evaluated. The structures were modeled as two-dimensional shear buildings. The response calculated from TH analysis was compared with that predicted by the ELF procedure embodied in UBC. Based on this comparison, the aim was to evaluate the current requirements under which a structure can be considered regular, and the ELF provisions applicable. From the results of the parametric study, the following conclusions can be drawn.

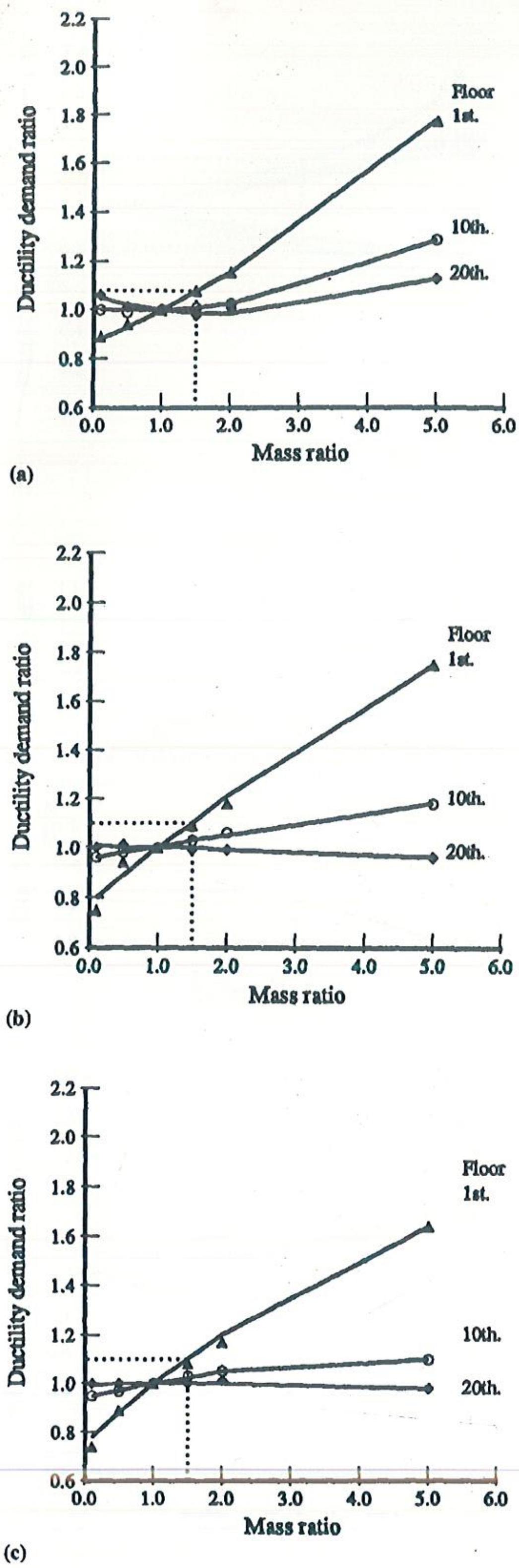

FIG. 6. Maximum Ductility Demand for 20-Story Structures with Mass Variation: (a) Design Ductility = 2; (b) Design Ductility = 6; (c) Design Ductility = 10

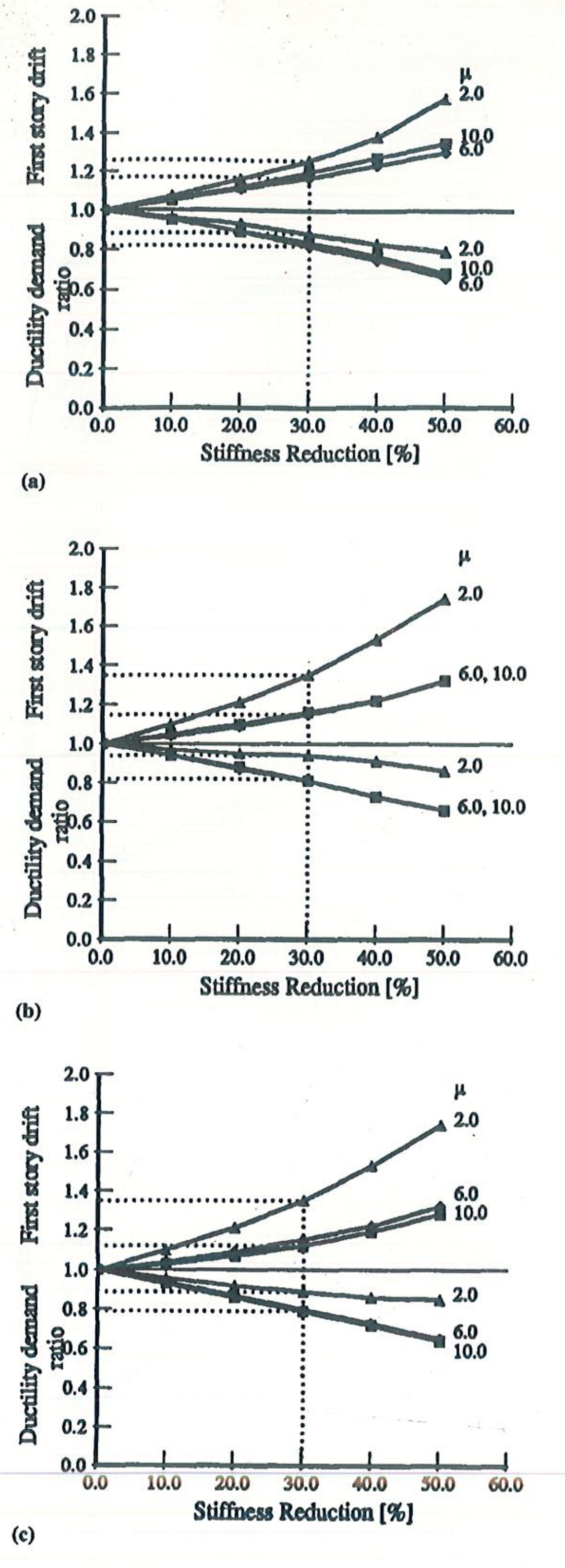

FIG. 7. Maximum Ductility Demand and First-Story Drift for Structures with Stiffness Irregularities: (a) 5-Story Structures; (b) 10-Story Structures; (c) 20-Story Structures

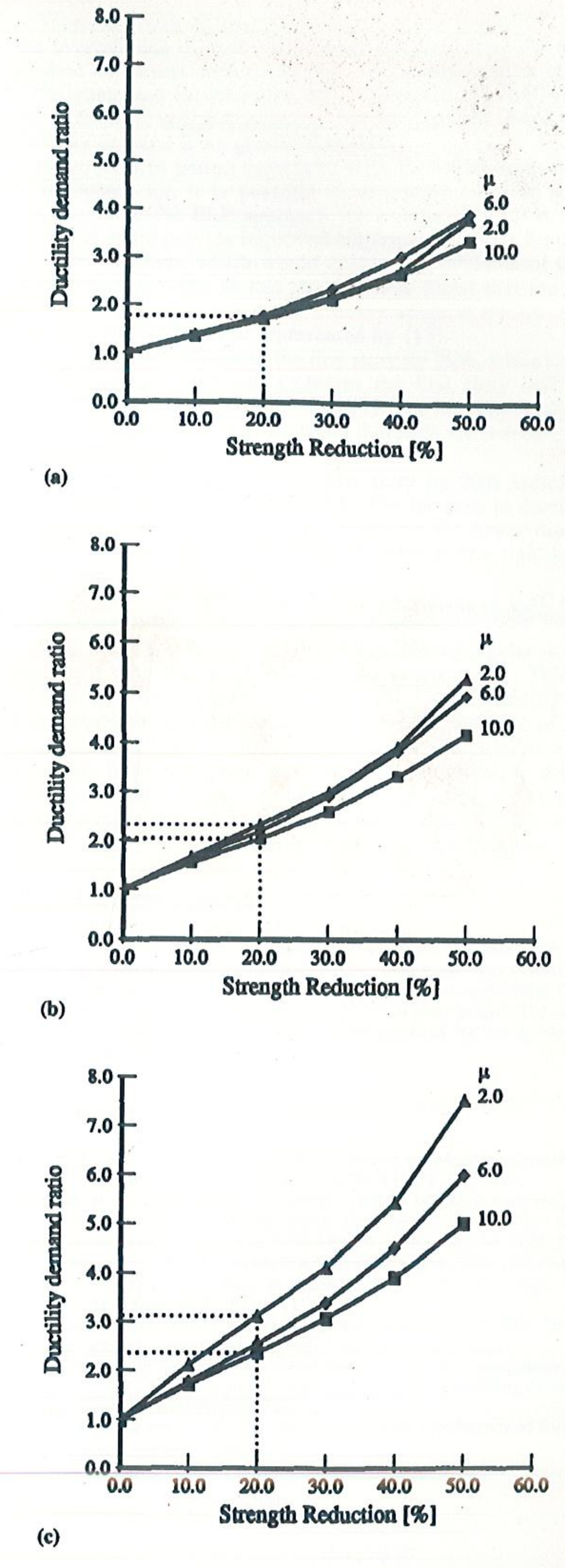

FIG. 8. Maximum Ductility Demand for Structures with Strength Irregularities: (a) 5-Story Structures; (b) 10-Story Structures; (c) 20-Story Structures

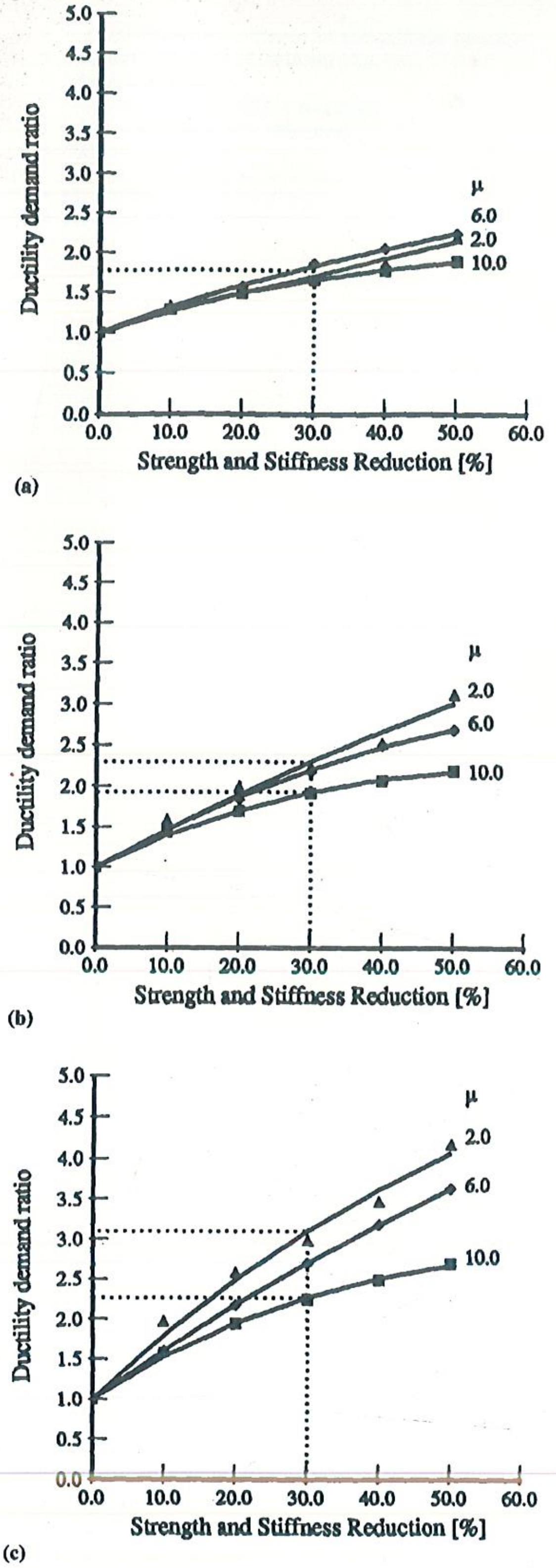

FIG. 9. Maximum Ductility Demand for Structures with Strength and Stiffness Irregularities: (a) 5-Story Structures; (b) 10-Story Structures; (c) 20-Story Structures

When the ratio of the mass of one floor to the next is 1.5, the overestimate of base shear forces obtained from the ELF method consistent with earthquake code provisions is about 10%, compared to structures with a uniform distribution of mass. Inelastic analysis indicates that the expected increase in ductility demand is no greater than 20%.

If the shift in period associated with the variation in mass is accounted for, it is possible to determine the base shear reliably using the ELF approach for a mass ratio up to 5.0. The code could provide improved empirical equations for nonuniform buildings, which would enable an evaluation of their fundamental periods. In this study, it was found that the relative increase in period Δ for a heavy mass at the *i*th floor could be reasonably well represented by (13).

Reducing the stiffness of the first story by 30%, while keeping the strength constant, increases the first story drift by 20–40%, depending on design ductility. The increase is largest for a design ductility of 2. Ductility demands are reduced, but not significantly.

Reducing the strength of the first story by 20% increases the ductility demand by 100–200%. The increase in ductility demand is greatest for 20-story structures for lower design ductilities. The effect of the design ductility is less significant for 5- and 10-story structures.

The weak-story criterion in the ELF provisions of UBC that allows a structure with the first story 20% weaker (20% lower strength) than the floor above to be considered regular is not consistent with the mass and stiffness requirements. This is important, since in practice, the uncertainty in determining the actual strength of a story is usually 10–20%. Because of the variation in strength of structural members and possible contributions from nonstructural elements, it is difficult to determine the strength accurately. Based on this study, the criteria should require that the first story be no weaker than the story above in order for the structure to be considered regular.

ACKNOWLEDGMENTS

The results presented in this paper are based on the thesis for the Master of Science degree of the first writer. This study was conducted in, and supported by, the Department of Civil Engineering at North Carolina State University. This support is gratefully acknowledged. The writers would also like to thank Fjóla G. Sigtryggsdóttir for her generous assistance throughout this study.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

$f_{s,i}$ = force at story i;
h_i = height of floor i above base;
h_n = total building height;

k = stiffness;
k_n = reduced first-story stiffness of nonuniform structure;
k_u = first-story stiffness of uniform structure;
M = total actual mass;
M_{nu} = total mass of nonuniform structure;
M_u = total mass of uniform structure;
m_i = mass of floor i;
N = number of degrees of freedom;
S_a = pseudospectral acceleration;
S_v = pseudospectral velocity;
T = fundamental period;
T_{avg} = average measured building period;
T_{lb} = lower bound of measured building period;
T_{nu} = period of nonuniform structure;
u_y = yield displacement;
V = base shear force;
Γ = modal participation factor;
Δ = relative shift in period;
ϕ_i = modal displacement; and
ω = circular natural eigenfrequency.

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