International Journal of Advances in Engineering and Management (IJAEM)
Volume 3, Issue 7 July 2021, pp: 448-464 www.ijaem.net ISSN: 2395-5252

# Earthquake Performance Of G+Four Existing R.C.C Building By Using E-Tabs Software

Submitted: 25-06-2021 Revised: 01-07-2021 Accepted: 03-07-2021

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**ABSTRACT**

In past earthquakes (1997 Jabalpur earthquake) many R.C.C concrete structures have been severely damaged or collapsed, have indicated the need for evaluating the seismic adequacy of existing buildings. In particular, the seismic rehabitation of older concrete structures in high seismicity areas is a matter of growing concern, since structures venerable to damage must be identified and an acceptable level of safety must be determined.to make such assessment, simplified linear-elastic methods are not adequate.

Although different procedures are possible, the non-linear static analysis, also known as the push over analysis, is a method for evaluating the performance. on this study, the method is used to evaluate the performances of RC plane frames. Reinforced concrete frame building are becoming increasingly common in urban and rular India due to increases in population and safety in such situation is much more important.

The static pushover analysis is becoming a popular tool for seismic performance evaluation of existing structures. The expectation is that the pushover analysis will provide adequate information on seismic demand imposed by the design ground motion on the structural system and its components. the purposes of the paper is to summarize the basic concepts on which the pushover analysis can be used. Asses the accuracy of pushover predictions, identify condition under which the push over will provide adequate information and perhaps more importantly, identify cases in which the pushover predictions will be inadequate or even misleading. The paper deals with non-linear analysis of an Existing RCC frame. The main aim is to carry out the pushover curves of the RCC frame and to calculate the displacement of the frame.

The analysis is carried out by using ETABS software. Push-over curves for the frame are obtained and carried out.

**KEYWORDS**:Limitstatemethod, Staddpro, NonlinearstaicAnalysis,ETABS,Pushover curve,Capacity Spectrum method,Performance point.

## I. INTRODUCTION

**1.1 Back ground:** The major criteria now-a-days in designing RCC structures in seismic zones is control of lateral displacement resulting from lateral forces. In this thesis effort has been made to investigate the lateral displacement and Base Shear in RCC Frames. RCC Frames with G+4 are considered.

Non-linear static analysis (pushover analysis) was carried out for the frames and the frames were then compared with the push over curves. Displacement and Base shear are calculated from the curves.

The nonlinear analysis of a frame has become an important tool for the study of the concrete behaviour including its load-deflection pattern and cracks pattern. It helps in the study of various characteristics of concrete member under different load condition.

**1.2 Objective:**

- To study the performance of RC plane frames under lateral loads (Earthquake loads).
- To study the inelastic response of RC plane frames using Pushover analysis
- To study the variation of pushover curve for a plane framed structure.

**1.3 Scope:**

- Only multi-storey frames are considered.
- Plan irregularities are not considered.

Push over analysis is used as a non-linear static method to predict the actual performance of the RC Frames under lateral loadings.

**1.Methodology:**

For the purpose of study, a plan of G+4 floor levels were considered. For push over study, RC plane frames in each floor were analysed and

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designed for gravity loads as per IS 456:2000 and lateral loads (earthquake loads) as per IS 1893 (part-1):2002.

## 1.2 STRUCTURAL MODELLING

### 1.2.1 Introduction

Building codes are revised from time to time and the revision necessitates checking the adequacy of existing building for the demand as per the latest codes of practice. Code of practice for plain and reinforced concrete for general building construction was first published by the Bureau of Indian Standards (BIS) in 1953 and subsequently got revised in 1957. It was further revised in 1964. In this version and before only working stress method was in practice. The limit state design methodology was introduced in IS: 456 - I978. Latest revision for this code is IS: 456-2000. Similarly, the code for criteria for earthquake resistant design of structures IS: 1893 was introduced in 1962. This standard was subsequently revised in 1966, 1970, 1975, 1984 and 2002.

### 1.2.2 Code based Design

In India the two design approaches are used for the design of RC structures as per IS: 456 and they are i) working stress method (IS: 456-1964and IS: 456-1978) and ii) limit state method (IS: 456-1978 and IS: 456-2000). The conceptual difference between working stress method and limit state method is given in the Table 3.1. The estimation of design seismic base shear based on seismic coefficient method as per the revisions of IS: 1893.The conceptual development and methodology adopted in working stress and limit state method are discussed in the following sections along with problem definition.

### 1.2.3 Flexure member

A reinforced concrete beam should be able to resist tensile, compressive and shear stresses induced in it by loads on the beam. Concrete is fairly weak in tension and strong in compression. But steel strong in tension. Thus, tensile weakness of concrete is overcome by the provision of reinforcing steel in the tension zone round the concrete to make a reinforced concrete beam.

There are three types of reinforced concrete beams:

(1) Singly reinforced beams
(2) Doubly reinforced beams, and
(3) Flanged beams.

Doubly reinforced sections are used in situation where reversal of moments is likely (as in multistoried frame subjected to lateral loads)

### 1.2.4 Compression member

A column forms a very important component of a structure. Columns support beam which in turn support walls and slabs. It should be realized that the failure of a column results in the collapse of the structure. A column is defined as a compression member; Columns may be cast to any one of the following shapes- square, rectangular, circular, hexagonal, octagonal, etc. for column members the I.S. The procedure for design of compression member subjected to axial load and bending moment as per IS: 456-1964. Recommendations for longitudinal and transverses details are given in code book IS : 456-2000.

#### 1.2.4.1 Long columns

A column will be considered as short when the ratio of the effective length to its least lateral dimension is less than or equal to 12, otherwise the column will be considered as a long column.

### 1.2.5 Limit State Method

This method of design is based on limit state concepts. In this method, the structure shall be designed to withstand safely all loads liable to act on it throughout its life; and it shall also satisfy the serviceability requirements, such as limitations on deflection and cracking. The acceptable limit for the safety and serviceability requirement before failure occurs is called limit state method. All relevant limit states shall be considered in design to ensure an adequate degree of safety and serviceability. In general, the structure shall be designed on the basics of the most critical limit state and shall be checked for other limit states.The Design should be based on characteristic values for material strengths and applied loads, which take into account the variations in the material strengths and in the loads to be supported. The characteristic values should be based on statistical data if available; the 'design values' are derived from the characteristics values through the use of partial safety factors, one for material strengths and the other for loads. In the limit state method of design which covers forms of failure, structure are designed for limit states at which the structures causes to function, the most important thing is

1) The limit state of collapse or total failure of the structure.

2) The limit state of serviceability which includes excessive deflection and excessive local damage.

### 1.3 Modelling

All the beam members and column members are drafted in auto cad and imported to staad.pro. The loads and properties were assigned there and then imported to the respective software i.e., E-tabs. The analysation there was performed, and results tabulated. The plan considered was represented below.

### 1.4 Materials

The modulus of elasticity of reinforced concrete as per IS 456:2000 is given by

$$E_c = 5000\sqrt{f_{ck}}$$

### 1.5 Structural Elements

In this section, the details of the modelling adopted for various elements of the frame are given below.

#### 1.5.1 Beams and Columns

Beams and columns were modelled as frame elements. The elements represent the strength, stiffness and deformation capacity of the members. While modelling the beams and columns, the properties to be assigned are cross sectional dimensions, reinforcement details and the type of material used.

#### 1.5.2 Beams and Columns joints

The beam-column joints are assumed to be rigid.

#### 1.5.3 Foundation modelling

Fixed supports were provided at the ends of supporting columns.

### 1.6 loads

All loads acting on the building except wind load were considered. These are

1. Dead Load
2. Live Load
3. Lateral Load due to Earthquake

It was assumed that wind load will not govern the demands on the members.

### 1.7 Preliminary data

**PRELIMINARY DATA:**

TYPE OF THE STRUCTURE : MULTI-STOREY RIGID JOINED FRAME
ZONE : 3
NUMBER OF STORIES : FOUR (G+4)
GROUND STOREY HEIGHT :3 meters
FLOOR TO FLOOR HEIGHT :3 meters
EXTERNAL WALLS:230 mm (INCLUDING PLASTERING)
INTERNAL WALLS :150 mm (INCLUDING PLASTERING)
LIVE LOAD : 2 $KN/m^2$
MATERIALS : M25 AND Fe 415
SEISMIC ANALYSIS : EQUIVALENT STATIC METHOD [ IS :1983 PART 1:2002]
DESIGN PHILOSOPHY : LIMITS STATE METHOD [ IS :1983 PART 1:2002]
DUCTILITY DESIGN:[ IS 13920:1993]
SIZE OF EXTERIOR COLUMN:300X300 mm
SIZE OF INTERIOR COLUMN :300X300 mm
SIZE OF THE BEAM IN LONGITUDINAL AND TRANSVERSE DIRECTION :300X450 mm
TOTAL DEPTH OF SLAB:150 mm

### 1.8Loads Combinations

The load combinations considered in the analysis according to IS 1893:2002 are given Table-1

| LOAD CASE | DETAILS OF LOAD CASES |
|---|---|
| 1 | 1.5(DL+LL) |
| 2 | 1.2(DL+LL+EL) |
| 3 | 1.2(DL+LL-EL) |
| 4 | 1.5(DL+LL) |
| 5 | 1.5(DL-EL) |
| 6 | 0.9DL+1.5EL |
| 7 | 0.9DL-1.5EL |

Table-1 load combinations

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**1.9Loading Data**
DEAD LOAD [DL]
Floor Finish (FF)1KN/Sq.m
Weight of Slab25: *D KN/Sq.m [D=Total depth of slab]
(Assuming total depth of slab150mm)
Weight of Walls Terrace Water Proofing (TWF): 1.2KN/Sq.m
External wall (250mm thick)
=4.45KN/m/meter height (17.8 @ 0.25)
Internal Wall (150mm thick)
=3.25KN/m/meter height (17.8@ 0.15)
LIVE LOAD [LL]
Roof =1.5KN/$m^2$
Live load on floor: 2KN/$m^2$
EARTHQUAKE LOAD [EQ]
Referring from IS Code 1893: Part 1(2002)
αh = (Z/2) *(Sa/g) *(I/R)
Z = 0.16(Zone 3)
Sa/g = 2.5
T = 0.09* h/sqrt.d
=0.09* 15/sqrt (7.4)
=0.496 [0.10 ≤ T ≤0.55]
I (Importance factor) = 1.0
R (Response factor) =3.0[OMRF]
EARTHQUAKE LOAD ANALYSIS
Determination of total base shear
Dead load
a, weight of floor i.e (Ws+FF)
=42.30*7.40*(3.25+1.00)
=1330.33KN
b, weight of roof i.e (Ws+TWF+FF)
=42.30*7.40*(3.25+1.20+1.00)
=1705.95KN
C, weight of peripheral beams (transverse)
[{2(3.0-0.45/2-0.30/2) *3.375} *2+{1(2.4-0.30/2-0.30/2)*3.375}*2]
=35.44+14.175
=49.615KN
d, weight of peripheral beams (longitudinal)
= [{(2.8-0.30/2-0.30/2) *3.375*2} +{(3.8-0.30/2-0.30/2)*3.375*2}+{(4.5-0.30/2-0.30/2)*3.375*2}}*2+{(4.5-0.30/2-0.30/2)*3.375*2}*2]
= [(16.875+23.625+56.70] +(56.7+56.7)]
=307.8KN
e, weight of parapet wall [1.0m height,150mm thick]
=2*(42.30+7.40) *1.0*3.25
=323.05KN
f, weight of external wall (thickness of wall 230m)
=20*0.230*(70.44+13.75) *(3.00-0.45)
=987.548KN
g, interior beam (transverse)
= [(3.0-0.3) +(2.4-0.3) +(3.0-0.3) *3.375*11*2]
=455.625KN
h, interior beam (longitudinal)
= {[{(2.8-0.3) +(3.8-0.3) +(4.5-0.3)} *3.375*9]} *2
=625.725KN
i, weight of interior wall
(thickness =150mm)
Length (transverse)
= {(3.0-0.45/2-0.3/2) *2+(2.4-0.3)} *8
=58.8m
Length (longitudinal)
= {(2.8-0.3) +(3.8-0.3) +(4.5-0.3) *9*2}
=183.6m
Height=3.00-0.56=2.55m
Weight =20*0.15*(58.8+183.6) *(3.00-0.45)
=1854.36KN
j, weight of exterior column /height
=2*12*0.30*0.30*25
=54.00KN/m
k, weight of interior column/height
=2*12*0.30*0.30*25
=54.00KN/m
LIVE LOAD
Live load on roof =zero
Live load on floors
=50%of 2KN/m.sqm=1KN/m.sqm
Total live load on each floor
=42.30*7.40*1
=313.02KN
Concentrated mass:
AT ROOF
= (b+c+d+e+(f/2) +g+h+(i/2) +(j*3/2) +(k*3/2) +0.0 = 5869.89KN
AT FIRST FLOOR
=(a+c+d+f+g+h+i+(j+k) *(3.0+3.0) *1.00)+313.02) = 6572.01KN
AT SECOND, THIRD, FOURTH FLOOR
==(a+c+d+f+g+h+i+(j+k) *(3.0) +313.02) = 6248.01KN
TOTAL WEIGHT
= 5869.89+(3*6248.01) +6572.01 = 31,185.93KN
TOTAL BASE SHEAR
= αh *w
=0.06 * 31,185.93
=1871.15KN
BASE SHEAR @ EACH FRAME
Vb = 1871.15/12 = 155.92KN
DETERMINATIONS OF DESIGN LATERAL LOADS AT EACH FLOOR:

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| LOAD | W1(KN) | H1(m) | Wi*Hi sqm | Wi*hi sqm/ ∑Wi*hi sqm | Q1=Vb*Wi*hi sqm/∑Wi*hi sqm (KN) |
|---|---|---|---|---|---|
| ROOF | 5869.89 | 15.00 | 1.32X10^6 | 0.439 | 68.44 |
| FOURTH FLOOR | 6248.01 | 12.00 | 8.99X10^5 | 0.298 | 46.46 |
| THIRD FLOOR | 6248.01 | 9.00 | 5.06X10^5 | 0.168 | 26.19 |
| SECONDFLOOR | 6248.01 | 6.00 | 2.24X10^5 | 0.074 | 11.53 |
| FIRST FLOOR | 6572.01 | 3.00 | 5.9X10^4 | 0.019 | 2.96 |
| GROUND FLOOR | – | 0.00 | – | – | – |
| | | ∑ = | 3.01X10^6 | 0.99 (1.0) | 155.58 |

Table-2 Lateral Load Distribution

**1.10 Structure Analysis**

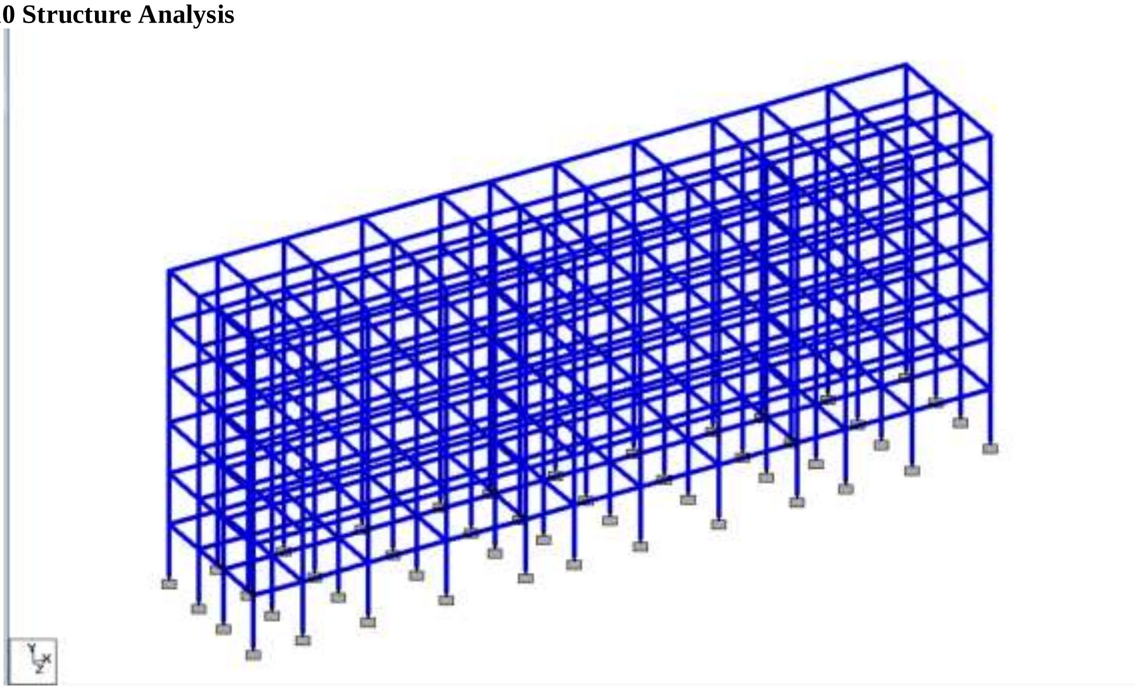

Fig-1 RCC Frame with Beams And Columns In (Staad Pro)

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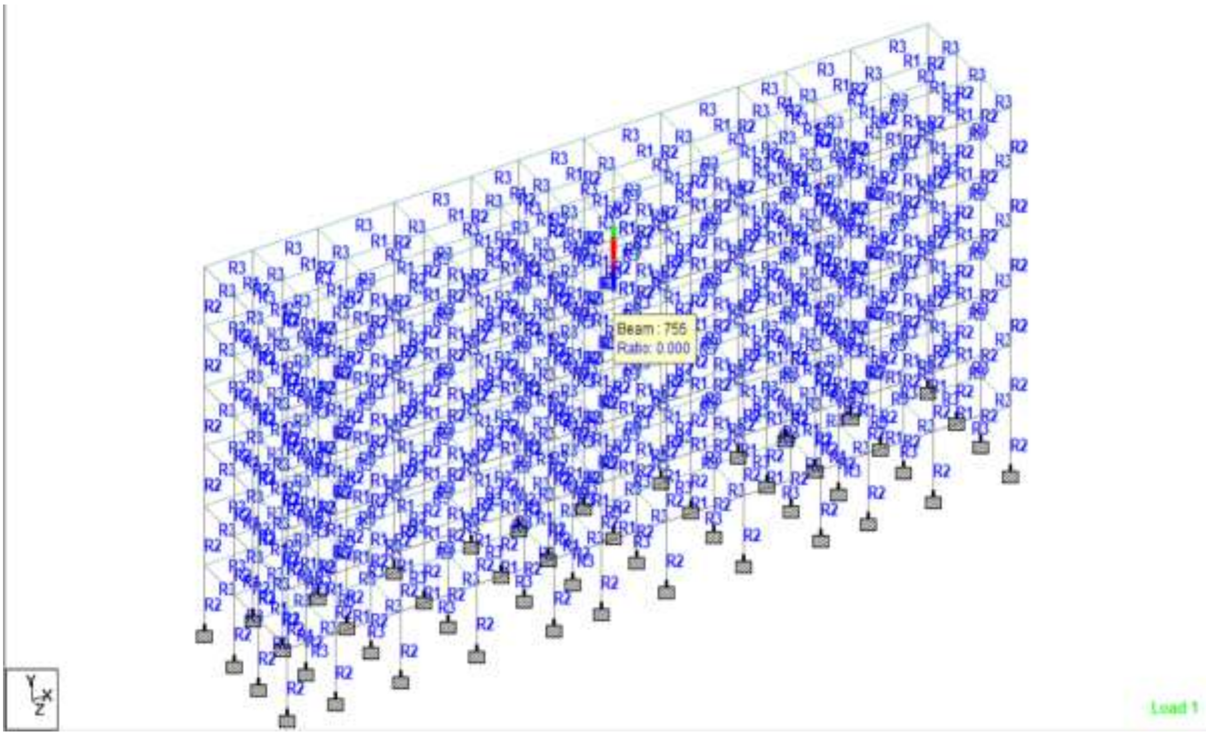


Fig-2 RCC Frame With Reactions In (Staad Pro)

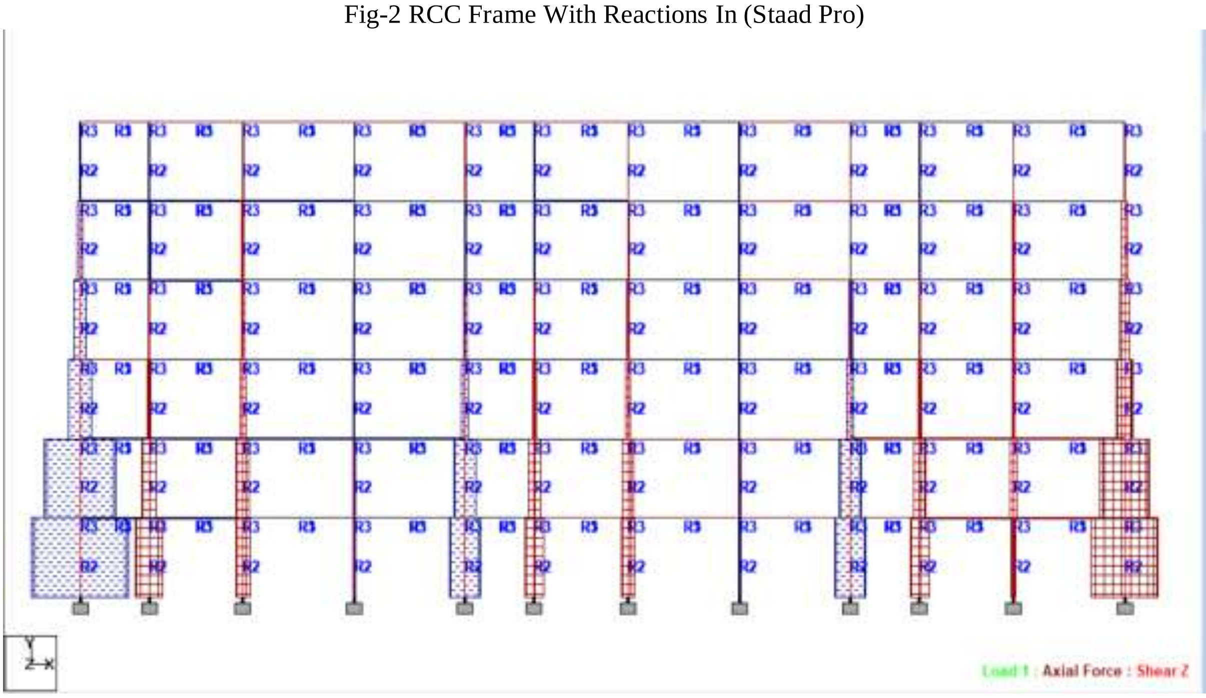


Fig-3 RCC Frame with Axial Force In (Staad Pro)

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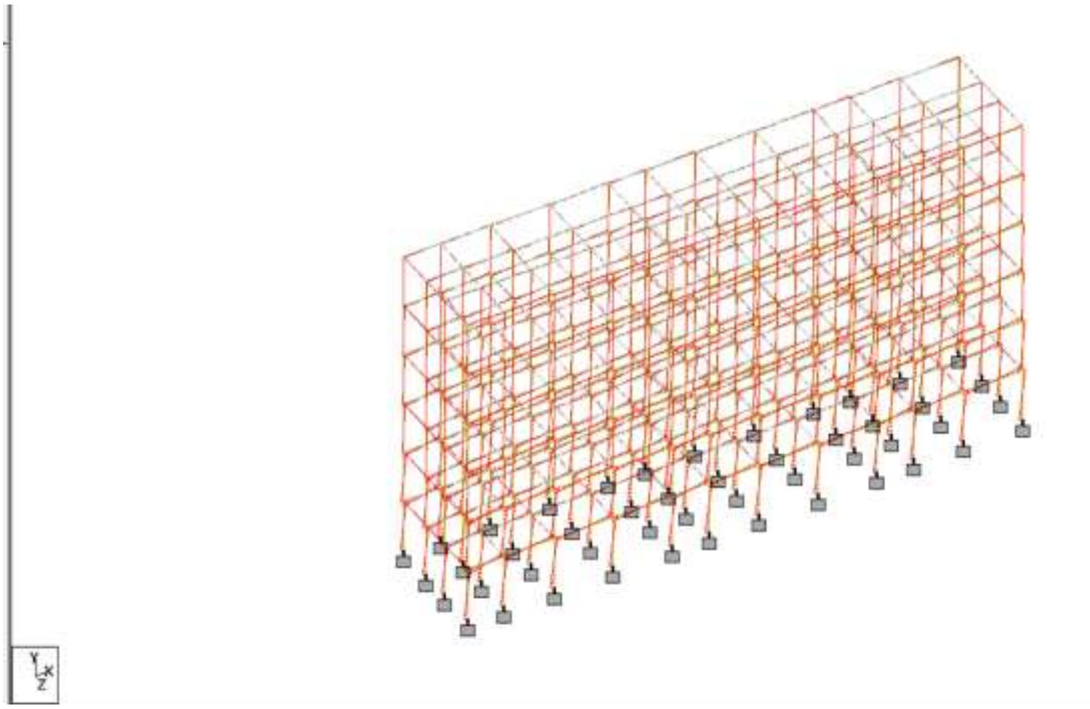

Fig-4 RCC Frame with Bending Moment In (Staad Pro)

Table-3 Result of analysis under various load combinations

| MEMBERS BEAMS | LOAD CASE | | SHEAR(FY) | MOMENT(MZ) | MAX SHEAR (fy) | MAX MOMENT (MZ) |
|---|---|---|---|---|---|---|
| 1 | 1 | | 29.695 | 13.167 | 48.774 | 49.323 |
| | 2 | | 48.774 | 39.606 | | |
| | 3 | | 48.774 | 39.606 | | |
| | 4 | | 37.713 | 48.057 | | |
| | 5 | | 37.713 | 48.057 | | |
| | 6 | | 35.137 | 49.323 | | |
| | 7 | | 35.137 | 49.323 | | |
| 2 | 1 | | 37.985 | 23.762 | 42.718 | 43.495 |
| | 2 | | 42.718 | 43.495 | | |
| | 3 | | 42.718 | 43.495 | | |
| | 4 | | 24.737 | 36.532 | | |
| | 5 | | 24.737 | 36.532 | | |
| | 6 | | 21.007 | 34.162 | | |
| | 7 | | 21.007 | 34.162 | | |
| 3 | 1 | | 42.264 | 30.815 | 44.902 | 47.82 |
| | 2 | | 44.902 | 47.82 | | |
| | 3 | | 44.902 | 47.82 | | |
| | 4 | | 23.66 | 36.937 | | |
| | 5 | | 23.66 | 36.937 | | |
| | 6 | | 19.315 | 33.747 | | |

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| | | | | | | |
|---|---|---|---|---|---|---|
| | 7 | | 19.315 | 33.747 | | |
| 4 | 1 | | 43.615 | 30.671 | 43.615 | 43.516 |
| | 2 | | 43.536 | 43.516 | | |
| | 3 | | 43.536 | 43.516 | | |
| | 4 | | 23.085 | 34.745 | | |
| | 5 | | 23.085 | 34.745 | | |
| | 6 | | 18.827 | 31.807 | | |
| | 7 | | 18.827 | 31.807 | | |
| 5 | 1 | | 29.323 | 17.78 | 41.578 | 39.315 |
| | 2 | | 41.578 | 39.315 | | |
| | 3 | | 41.578 | 39.315 | | |
| | 4 | | 30.851 | 37.919 | | |
| | 5 | | 30.851 | 37.919 | | |
| | 6 | | 28.21 | 36.196 | | |
| | 7 | | 28.21 | 36.196 | | |
| 6 | 1 | | 37.977 | 24.124 | 43.163 | 44.113 |
| | 2 | | 43.163 | 44.113 | | |
| | 3 | | 43.163 | 44.113 | | |
| | 4 | | 25.193 | 37.107 | | |
| | 5 | | 25.193 | 37.107 | | |
| | 6 | | 21.507 | 34.669 | | |

**1.11 Structure Design**
**1.11.1 Design of Beams**
Assuming 25mm dia bars with 25mm clear cover
Effective depth(d) = 450 – 25 – 25/2 = 412.5mm
d' / d = (25+12.5) / (412.5) = 0.091 =0.10
Reinforcement from table D, sp16 1980
Mulim /bdsq = 3.45 [for M25 and Fe415]
= 3.45 *300*412.5sq = 176.11KNm
Beam 1
Actual moment = 49.32KNm
Mulim = 176.11KNm
Actual moment is less than Mulim, so the section is a singly reinforced section.
Reinforcement from table 2, sp16 1980
Mu / bdsq = (49.32x10^6) / 300 *412.5sq = 0.96
Referring table3, sp16 1980 corresponding to Mu /bdsq & M25 = Pt = 0.291
Area = 0.291/100 *300*412.5 = 360.112sqmm
Provide [4 @ 16mm dia bars = 804.24sqmm]

1. Top and bottom reinforcement shall consist of atleast 2 bars throughout the member length.
2. Tension steel ratio
Min ≤ 0.24 *sqrt(fck/fy)
= 0.058 given 0.291
Hence ok
3. Max = 3.45 given 0.291
Maximum ratio at any section should not exceed = 3.45

Beam 2
Actual moment = 43.32KNm
Mulim = 176.11KNm
Actual moment is less than Mulim, so the section is a singly reinforced section.
Reinforcement from table 2, sp16 1980
Mu / bdsq = (43.32x10^6) / 300 *412.5sq = 0.85
Referring table3, sp16 1980 corresponding to Mu /bdsq & M25 = Pt = 0.246
Area = 0.246/100 *300*412.5 = 304.42sqmm
Provide [4 @ 16mm dia bars = 804.24sqmm]

1. Top and bottom reinforcement shall consist of atleast 2 bars throughout the member length.
2. Tension steel ratio
Min ≤ 0.24 *sqrt(fck/fy)
= 0.058 given 0.246
Hence ok
3. Max = 3.45 given 0.246
Maximum ratio at any section should not exceed = 3.45

Beam 3
Actual moment = 47.22KNm

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Mulim = 176.11KNm
Actual moment is less than Mulim, so the section is a singly reinforced section.
Reinforcement from table 2, sp16 1980
Mu / bdsq = (47.22x10^6) / 300 *412.5sq = 0.85
Referring table3, sp16 1980 corresponding to Mu /bdsq & M25 = Pt = 0.276
Area = 0.276/100 *300*412.5 = 341.42sqmm
provide [4 @ 16mm dia bars = 804.24sqmm]
1. Top and bottom reinforcement shall consist of atleast 2 bars throughout the member length.
2. Tension steel ratio
Min ≤ 0.24 *sqrt(fck/fy)
= 0.058 given 0.276
Hence ok
3. Max = 3.45 given 0.276
Maximum ratio at any section should not exceed = 3.45
Beam 4
Actual moment = 30.87KNm
Mulim = 176.11KNm
Actual moment is less than Mulim, so the section is a singly reinforced section.
Reinforcement from table 2, sp16 1980
Mu / bdsq = (30.87x10^6) / 300 *412.5sq = 0.60
Referring table3, sp16 1980 corresponding to Mu /bdsq & M25 = Pt = 0.171
Area = 0.171/100 *300*412.5 = 211.612sqmm
Provide [4 @ 16mm dia bars = 804.24sqmm]
1. Top and bottom reinforcement shall consist of atleast 2 bars throughout the member length.
2. Tension steel ratio
Min ≤ 0.24 *sqrt(fck/fy)
= 0.058 given 0.171
Hence ok
3. Max = 3.45 given 0.171
Maximum ratio at any section should not exceed = 3.45
Beam 5
Actual moment = 24.285KNm
Mulim = 176.11KNm
Actual moment is less than Mulim, so the section is a singly reinforced section.
Reinforcement from table 2, sp16 1980
Mu / bdsq = (24.285x10^6) / 300 *412.5sq = 0.47
Referring table3, sp16 1980 corresponding to Mu /bdsq & M25 = Pt = 0.142
Area = 0.142/100 *300*412.5 = 175.42sqmm
Provide [4 @ 16mm dia bars = 804.24sqmm]
1. Top and bottom reinforcement shall consist of atleast 2 bars throughout the member length.
2. Tension steel ratio
Min ≤ 0.24 *sqrt(fck/fy)
= 0.058 given 0.142
Hence ok
3. Max = 3.45 given 0.142
Maximum ratio at any section should not exceed = 3.45

**1.11.2 Design of Column**
size of the column: 300 x 300 mm
grade - M25
vertical reinforcement – fe415
axial load - 1700 KN
bending moment - 56 KN-m
the general required of the column for ductility will follow from is-13920:1993
vertical reinforced of the column in designed according to 456:2000.
column subjected to bending and axial load

1. is 13920 :1993 specification will be applicable
2. if axial stress >0.1 $f_{ck}$
3. 1700 x 1000/300x300 = 18.89 n/sq.mm
4. 18.89> 2.5
5. minimum dimension of the member should be less than 200mm
6. shortest cross section dimension perpendicular dimensions should not be less than 0.4
7. i.e 300/300=1.0
8. vertical longitudinal reinforcement assumes 20mm dia with 40 mm cover
9. d'= 40+10=50mm
10. d'/10=50/300=0.16
11. from chart:45, sp-16,1980 (d'/d=0.15,415 n/sq.mm)
12. pu/$f_{ck}$ bd=1700x1000/2250x1000=0.7556
13. reinforced on four side from chart 45, SP-16 ,1980
14. p/fck= 0.095, in reinforcement in %=0.095x25=2.375%
15. as=pbd/100=2.375x300x300/100=2137.4 sq.mm
16. lap splice only in central halg portion of the member hoop over the entire splice length at spacing <150 not more than 50% bar shall be spliced at are section any are of column that extenda more than 100mm should be detailed as per is: 13920:1993
17. TRANSVERSE REINFORCEMENT:
18. HOOP REQUIREMENT AS PER FIG-7, IS 13920:1993
19. IF THE LENGTH OF THE HOOP >300 mm A CROSS TIE SHALL BE PROVIDED AS SHOWN IN FIG- 7B DETAILED AS FIG – 7C IN IS 13920:1993
20. HOOP SPACING SHOULD NOT BE GREATER THAN HALF LATERAL

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DIMENSIONS OF THE COLUMN i.e 300/2=150mm
21. THE DESIGN SHEAR FORCE FOR COLUMN SHALL BE MAXIMUM OF (a) AND (b)
22. CalCULATE FACTOR SHEAR FORCE AS ANALSIS
23. i.e, TABLE
24. FACTORED SHEAR GIVE BY
25. $Vu=1.4[(Mu^{bL}{}_{lim}+ Mu^{bR}{}_{lim})]/h$
26. $Mu^{bL}{}_{lim}$and $Mu^{bR}{}_{lim}$ MOMENT OF RESISTANCE OF OPPOSITE SIGN OF BEAM AND bg IS THE STOREY HEIGHT
27. MOMENT OF RESISTANCE OF BEAM
28. d'/d = 0.10
29. $Mu/bd^2$=56/0.3x 0.3 =6.2 $N/mm^2$
30. FROM THE TABLE 50 FROM CODE SP-16:1980
31. Pt=2.045
32. Pt=1.146
33. Pt=2.045 X 300 X $412.5^2$
34. 2530.75 $mm^2$ (4@20 dia +4@ 22 dia = 2776.63 $mm^2$)
35. Pb=21.146x300x$412.5^2$=1418.17 $mm^{22}$ (2@16 dia + 4@ 22 dia = 1922$mm^2$)
36. REFFERING TABLE 2, SP 16:1980
37. $Mulim/bd^2$=1.45
38. Mulim(HOGGING MOMENT CAPACITY = 1.45 X 300 X $300^2$=391.5 KN-m
39. $Mulim/bd^2$=1.20
40. SAGGING MOMENT CAPACITY = 1.20 X 300 X $300^2$=324 KN-m
41. Vu = 1.4 (392 +324)/3 = 335.07 KN
42. Vc = Γcbd= 0.53 X 300 X (300-50) =39.750 KN
43. Γc= 1X 1.67
44. ð = 1+3 Pu/Agfck= 1+3X 1700X$10^2$/90X1000X25=3.36
45. As=Pbd/100
46. Chart -45
47. P/fck = 0.08 REINFORCEMENT IN %
48. = 0.08 X 25 = 2%
49. As=pbd/100=2x 300X 300/100 = 1800$mm^2$
50. 6@20 dia = 1885 $mm^2$
51. Ast=As/2=942
52. THERE FORE NOMINAL SHEAR REINFORCEMENT SHALL BE PROVIDED IN ACCORDSNCE WITH 26.5.16 OF IS 456:2000
53. USE 8mm dia TWO LEGGED STIRRUPS
54. Asv=2X 50.26= 100.52 $mm^2$
55. For minimum stirrups
56. Sv ≤ Asv 0.87fy/0.4 X b=300
57. THE SPACING SHALL BE LESSER OF
58. 0.75d= 0.75X 250 = 187.5 mm
59. 300 mm (7.3.1)
60. 302 mm AS CALCULATED
61. SPECIAL CONFINING REINFORCEMENT
62. SPECIAL CONFINING REINFORCEMENT WILL PROVIDE OVER A LENGTH
63. OF 10 TOWARDS THE MID SPAN COLUMN
64. b <≠ {DETH OF MEMBER =300mm {1/6(CLEAR SPAN = (3-0.45)/6X425 {450mm.
65. THE SPACING OF HOOP SHALL NOT EXCEED
66. $ð_{MAX}$ ≥≠ {1/4(MINIMUM MEMBER DIMENSION) SHOULD NOT BE LESS THAN 75mm (Sh), SHOULD NOT BE GREATER THAN 100mm.}
67. MINIMUM AREA OF CROSS SECTION OF THE BAR FORCING HOOP IN
68. Ash= 0.18X Sh X fck/fy(Ag/Ak-1.0)
69. 89.45 $mm^2$
70. JOINT FRAMES:
71. THE SPECIAL CONFINING REINFORCEMENT AS REQUIRED AT THE END OF COLUMN SHALL BE PROVIDED THROUGH THE JOINT AS WELL ENLESS THE JOINT IS CONFINED BY 8.2
72. A JOINT WHICH HAS BEAM FRAMES INTO ALL VERTICAL FACES OF IT AND BEAM WIDTH IS AT LEAST ¾ OF THE COLUMN WIDTH MAY BE PROVIDED WITHJ HALF THE SPECIAL CONFINING REINFORCEMENT REQUIRED AT THE END OF THE OF THE COLUMN THE SPACING OF HOOP SHALL NOT EXCEED 150mm
73. Ash= 89.45/2=44.72$mm^2$
74. Use 8mm dia bar (50.26$mm^2$) AT A USE 8mmdia BAR (50.26$mm^2$) AT A SPACING OF 94.8 X50.26/44.72=106.5mm.

## II. PUSHOVER ANALYSIS OF FRAME

**Pushover analysis of frames**

Pushover analysis is a static, nonlinear procedure in which the magnitude of the lateral loads is incrementally increased, maintaining a predefined distribution pattern along the height of the building. Pushover analysis can determine the behaviour of a building, including the ultimate load and the maximum inelastic deflection. Local nonlinear effects are modelled, and the structure is pushed until a collapse mechanism is developed. At each step, the base shear and the roof displacement can be plotted to generate the pushover curve.

**2.1Necessity of Non-Linear Stati Pushover Analysis(NLSA)**

The existing building can become seismically deficient since seismic design code requirements are constantly upgraded and advancement in engineering knowledge. Further, Indian buildings built over past two decades are seismically deficient because of lack of awareness regarding seismic behaviour of structures. The wide spread damage especially to RC buildings during earthquakes exposed the construction practices being adopted around the world, and generated a great demand for seismic evaluation and retrofitting of existing building stocks

**2.1.1Purpose of Push-Over Analysis**

The purpose of pushover analysis is to evaluate the expected performance of structural systems by estimating performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of static inelastic analysis and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter-story drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element connection forces (for elements and connections that cannot sustain inelastic deformations), The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces that no longer can be resisted within the elastic range of structural behaviour.

**2.1.2Non-Linear Static Analysis for buildings**

Seismic analysis of buildings can be categorized depending upon the sophistication of modelling adopted for the analysis. Buildings loaded beyond the elastic range can be analysed using Non-Linear static analysis, but in this method, one would not be able to capture the dynamic response, especially the higher mode effects. This is pushover analysis. There is no specific code for NLSA. This procedure leads to the capacity curve which can be compared with design spectrum/DCR of members and one can determine whether the building is safe or needs strengthening and its extent.

The capacity of structure is represented by pushover curve. The most convenient way to plot the load deformation curve is by tracking the base shear and the roof displacement. The pushover procedure can be presented in various forms can be used in a variety of forms for the use in a variety of methodologies. As the name implies it is a process of pushing horizontally, with a prescribed loading pattern, incrementally, until the structure reaches the limit state. There are several types of sophistication that can be used over for pushover curve analysis.

**Level-1**: It is generally used for single storey building, where at a single concentrated horizontal force equal to base shear applied at the top of the structure and displacement is obtained.

**Level-2**:In this level, lateral force in proportion to storey mass is applied at different floor levels in accordance with IS: 1893-2002 (Part-I) procedure, and story drift is obtained.

**Level-3**: In this method lateral force is applied in proportion to the product of storey masses and first mode shape elastic model of the structure. The pushover curve is constructed to represent the first mode response of structure based on the assumption that the fundamental mode of vibration is the predominant response of the structure. This procedure is valid for tall buildings with fundamental period of vibration upto 1 sec.

**Level-4**: This procedure is applied to soft storey buildings, wherein lateral force in proportion to product of storey masses and first mode of shape of elastic model of the structure, until first yielding, the forces are adjusted with the changing the deflected shape.

**Level-5**:This procedure is similar to level 3 and level 4 but the effect of higher mode of vibration in determining yielding in individual structural element are included while plotting the pushover curve for the building in terms of the first mode lateral forces and displacements. The higher mode effects can be determined by doing higher mode pushover analysis. For the higher modes, structure is pushed and pulled concurrently to maintain the mode shape.

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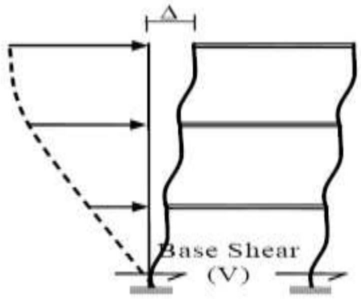
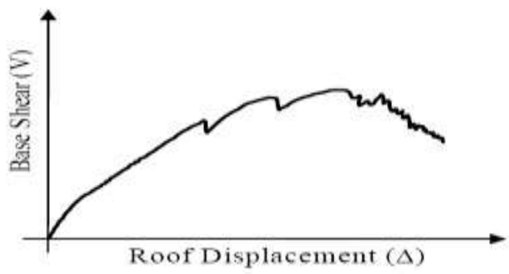



a) Building model




b) Pushover curve

**2.1.3 Capacity Spectrum method**

The nonlinear static pushover analysis is a comprehensive method of evaluating earth quake response of structures explicitly considering nonlinear behaviour of structure elements. The capacity spectrum method is on approach for implementing pushover analysis that compares structure capacity with ground shaking demand to determine peak response during an earthquake.

The capacity spectrum method estimates peak response by expressing both structure capacity and ground shaking demand in terms of spectral acceleration and displacement (hence the name capacity spectrum)

A capacity spectrum is the base shear versus roof displacement curve. When the demand spectrum is plotted along with the capacity spectrum in an Acceleration Displacement Response Spectrum (ADRS) format, the two curves may meet to give a performance point. The performance point represents the maximum deformation and the degree of damage that the building will sustain the applied static forces.

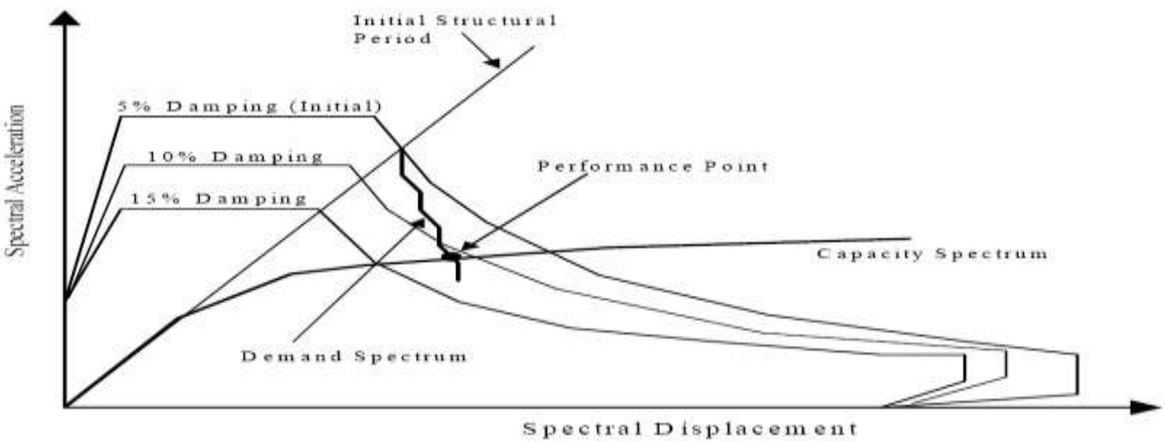



Fig -5 capacity spectrum curve

**2.2 Seismic load Distribution**

Pushover analysis requires the seismic load distribution with which the structure will be displaced incrementally. The load distribution is based on the first three mode shapes.

**2.3Different Hinge properties in Pushover Analysis**

There are three types of hinge properties in E-Tabs. They are

1) Default hinge properties,
2) User-defined hinge properties and
3) Generated hinge properties.

Only default hinge properties and user-defined hinge properties can be assigned to frame elements. When these hinge properties are assigned to a frame element, the program automatically creates a different generated hinge property for every hinge.

**2.4 Limitations of Pushover Analysis**

Although pushover analysis has advantages over elastic analysis procedures, underlying assumptions, the accuracy of pushover predictions and limitations of current pushover procedures must be identified. The estimate of target displacement, selection of lateral load patterns and identification of failure mechanisms due to higher modes of vibration are important issues that affect the accuracy of pushover results.

## III. MODELLING OF FRAME

### 3.1Modelling of frame

All the preliminary modelling was done in staad.pro and the modelled frame was imported into E-Tabs. A four-storey frame was modelled in STAAD Pro. and imported to E-Tabs. The main aim is to derive the difference in displacement & Base Shear.

### 3.2Member Properties

- ✓ All the beams in the frame were sized to 0.30m X 0.45m
- ✓ All the columns in the frame were sized to 0.3m X 0.3m in case-1
- ✓ The slab of 0.15m thickness was taken for the analysis purpose and assigned to each floor.
- ✓ Default M3hinge was assigned to beams.
- ✓ Default P-M-M hinge was assigned to columns.

### 3.3Member Loading

All the members were assigned the following loadings.

- ✓ Self-Weight
- ✓ External Wall Load--- 17.8 KN/m
- ✓ Internal Wall Load--- 14 KN/m
- ✓ Live Load----------- 2 KN/m
- ✓ Earth Quake Loading----- as per IS-code:1983-2002
- ✓ It was assumed that the wind force was not governing the frame efficiency.

### 3.4 Push over cases

Two pushover cases were defined for the analysis

- ✓ Push1 also known as gravity pish which is done for gravity loading (DL+LL) for which it is done in Load defined pattern.
- ✓ Push-2also known as lateral push in which the governing load is lateral load (EQ)for which it is done in displacement defined pattern.

### 4.Results and Discussions.

### 4.1Results

The results from the analysis are the deflected shape and the formation of hinges with increasing load and their performance levels.

The frames can be found from the displacement and base shear plots i.e., push-over curve. Capacity Spectrum curve can be drawn from the analysed plot.

From the capacity spectrum curve the existence of performance point can be noted. If the performance point doesn't exist, the structure fails to achieve the target performance level.

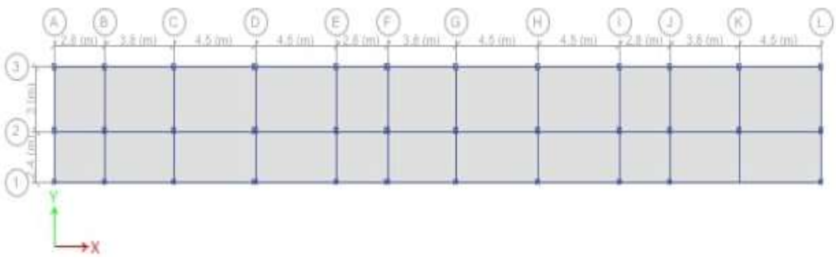


Fig-6 RCC Frame (Plan) in ETABS

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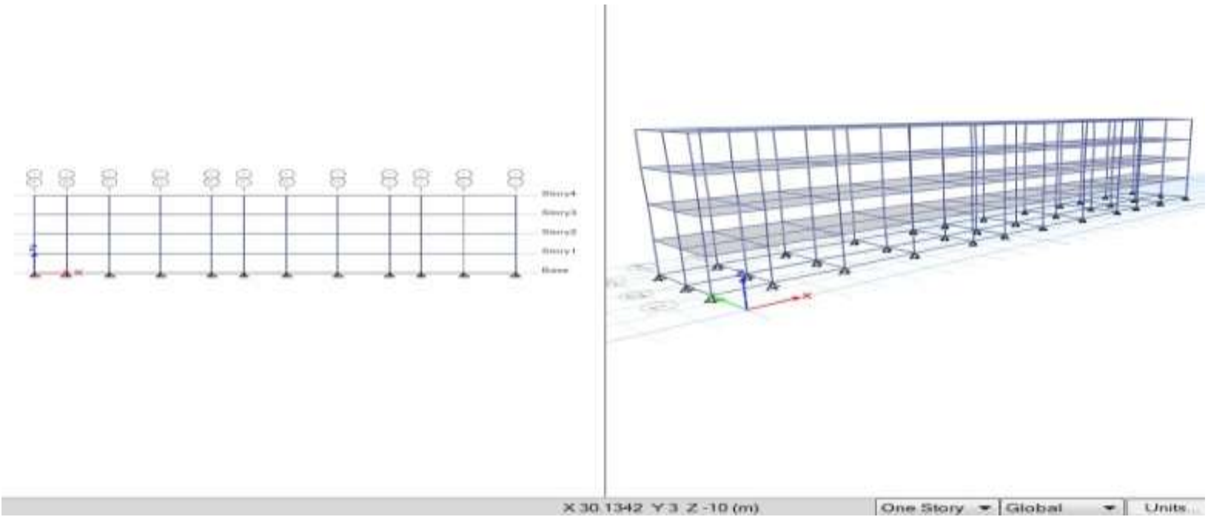
Fig-7 RCC frame (3D view)

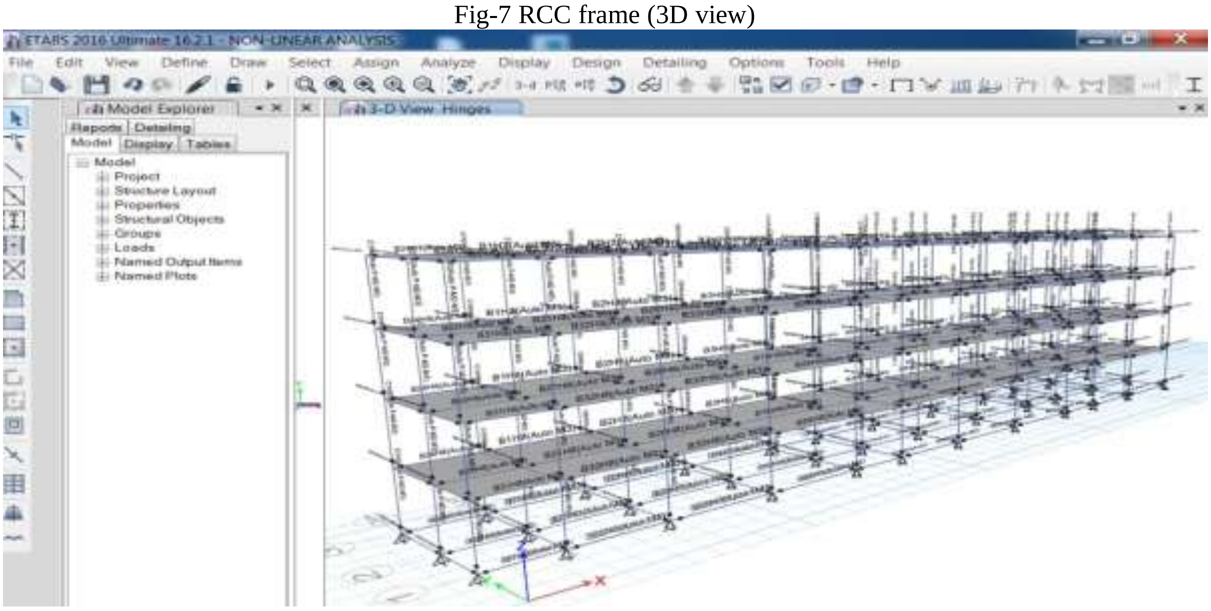

Fig-8 RCC frame with user defined Hinges

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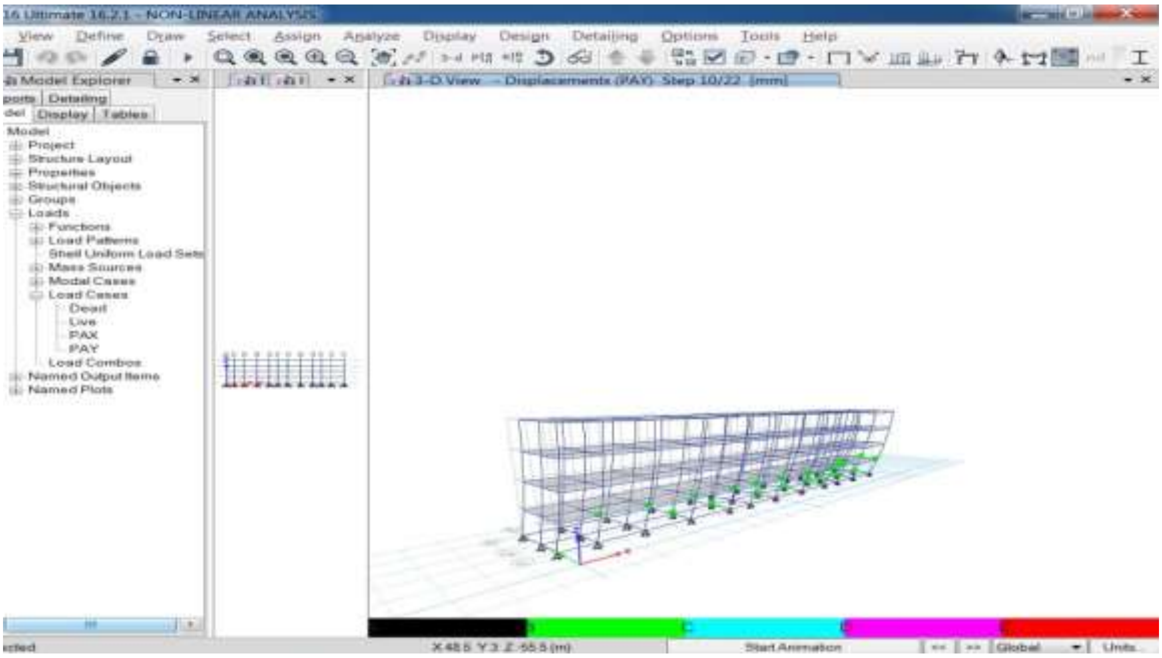

Fig-9 RCC frame deformed shape

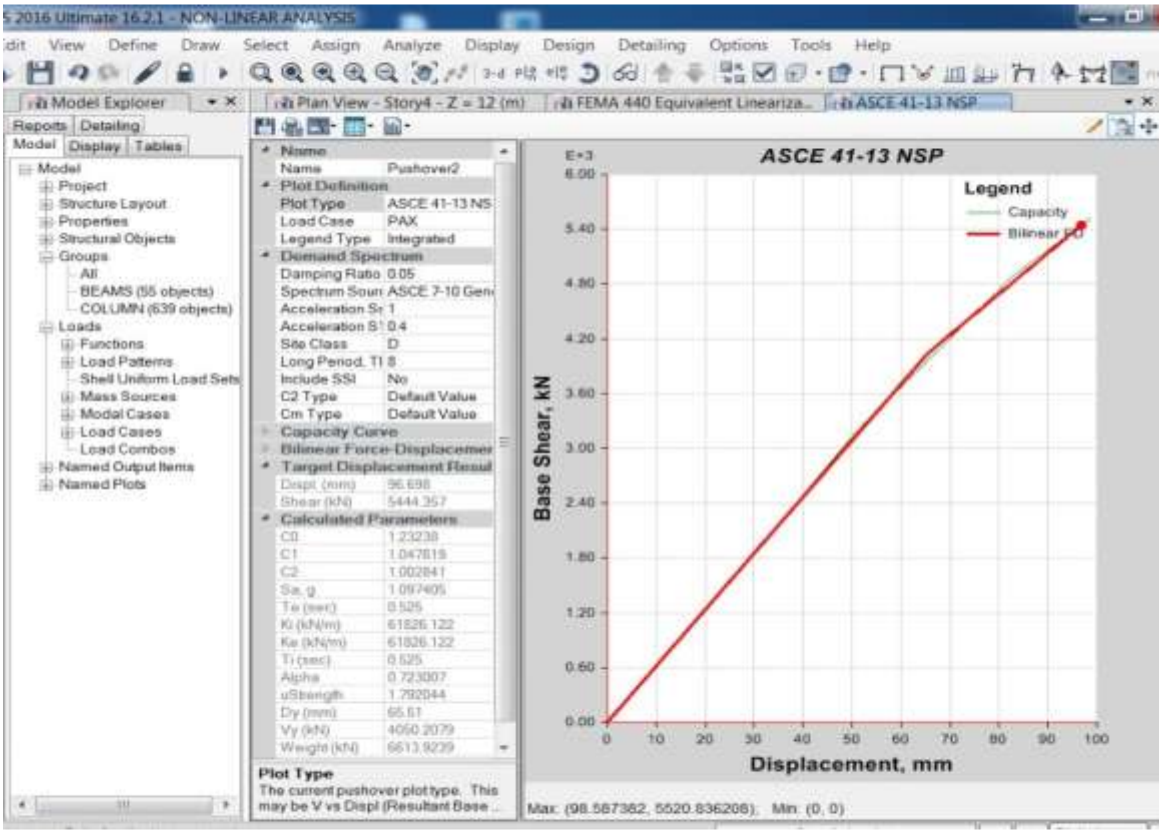


Table-4 RCC frame Pushover curve

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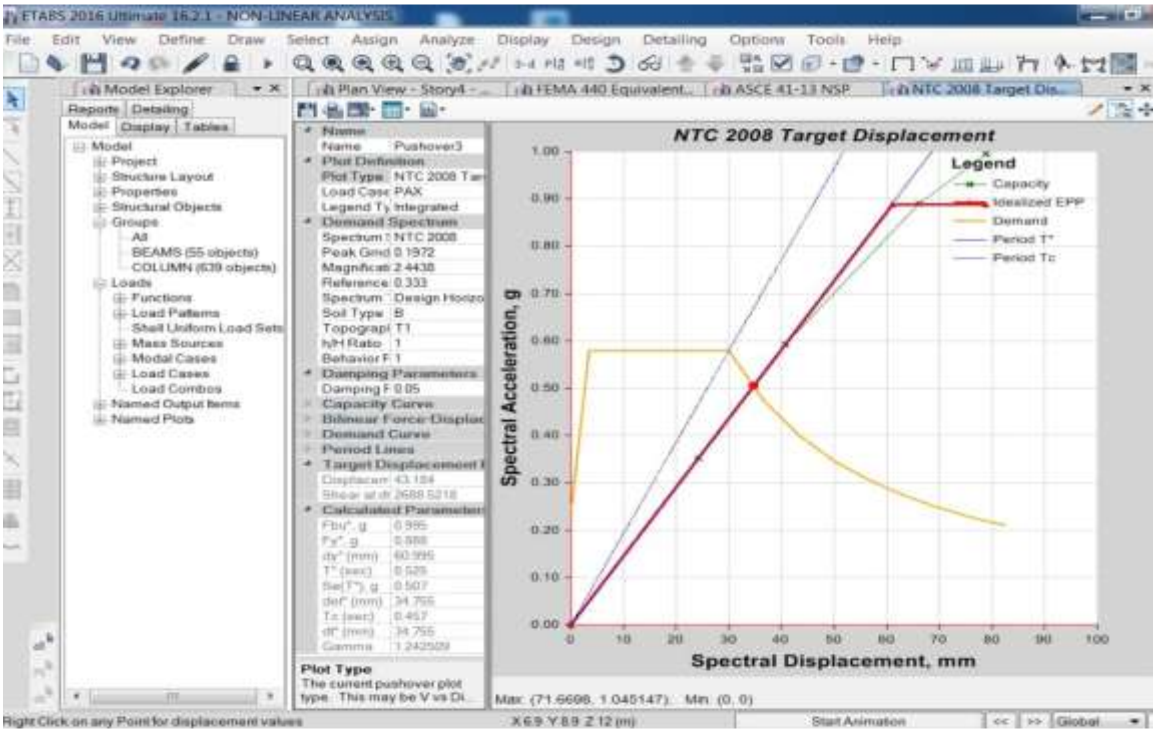

Table-5 RCC frame Capacity spectrum curve

PUSHOVER CURVE OF FRAME

**Table 6 Maximum Base shear and Roof displacement for the G+4-storey building**

| case | Base shear (kN) | Roof Displacement (mm) |
|---|---|---|
| Case-1 | 4050 | 65.51 |

**4.2 Summary and Conclusions**

Performance evaluation procedures aim at assessing the inelastic base shear and inelastic displacement capacity of existing building. Modelling of building for performance evaluation necessitates the knowledge about the section and reinforcement details of existing buildings.

In this thesis, the evolution of RC design procedure of limit state method as given in different versions of IS: 456 are discussed. Various provisions in detailing such as minimum and maximum compression / tension reinforcement, transverses reinforcement for flexural and compression members with appropriate spacing of rectangular stirrups are critically reviewed and tabulated. Design steps for Reinforced concrete beams and columns as per limit state method are presented. Spread sheets are developed for the design of RC beams and columns as per limit state method.

In this thesis one typical designs have been carried out as per present codes of practice. The nonlinear static analyses are carried out and the capacity curves are generated. The actual values of maximum base shear and roof displacement capacities for the frame are brought out clearly. The performance points are obtained, and the corresponding base shear and roof displacements are arrived for NTC 2008 Target Displacement. It is clearly found that the frame to meet the performance point.

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