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# Rehabilitation of a Beam & Column in R.C Structures by Using Steel Jacketing Technique

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***Abstract*: Earth quake is one of the greatest natural hazards to life on this planet. The effects of the earthquake are very sudden with little or no warning to make alert against damages and collapse of the buildings. The hazards of life in the case of earthquake are entirely associated with manmade structures like building, bridges and dams etc. Prevention of structure from this disaster has become increasingly important in recent years. Prevention of disaster includes the prevention of seismic risk through retrofitting of existing buildings in order to meet the seismic safety requirements. There are lot of building which are not designed for earthquake forces or many buildings which are designed for earthquake forces but later on due to change in earthquake code, these buildings are needed to be retrofied. The new construction can be built earth quake resistant easily by adopting proper design methodology and quality control in construction but old construction which is not design with code provisions posses' enormous seismic risk in particular to human life and historic monuments. Most of the losses of lives in previous earthquakes in different countries have occurred due to collapse of buildings, these buildings are generally non-engineered, those constructed without any concern with the engineer. The safety of the non-engineered buildings from the large earthquakes is a subject of highest priority because in view of the fact that in the moderate to severe seismic zones of the world more than 90% of the population still lives and works in such buildings. The risk to casualty is rapidly increasing due to increasing population, poverty, deficiency of modern building materials, as cement and steel, lack of awareness and necessary skills and technology, particularly in the developing countries. Most of the small and residential buildings are built rapidly with little or without engineering inputs. The main problem creating this situation is lack of awareness, knowledge, and poor mechanism of technology dissemination. So it is highly needed to increase its capacity to bear these forces caused due to earthquake. Many high rise buildings are highly vulnerable to earthquake due to more height and large occupancy. The Bhuj earthquake of 26 January- 2001 can be considered as an ill-fated event when this class of buildings feels catastrophic damage causing large scale casualty and property loss. These buildings need immediate attention and seismic retrofit of such buildings to get safe from upcoming severe problems. This thesis presents an attempt towards quantitative evaluation of seismic vulnerability of this particular type of buildings and proposes practical solutions to reduce it. The results, with and without strengthening measures, are compared to estimate the effectiveness of the various intervention options.**

***Keywords*: Seismic Retrofit, Seismic Vulnerability, Disaster, Human life, Increasing Population, Poverty, Deficiency of Modern Building Materials.**

## 1. INTRODUCTION

The present study investigates the structural behaviour of an RC frame (G+2 Commercial building) under the additional load in the form of seismic forces. The structure is analyzed for two load cases. In first case (Gravity load case) structure is analyzed for only gravity forces and no seismic force is considered in this analysis while in second case (Seismic load case) structure is analyzed with consideration of seismic forces along with gravity forces. The analysis is performed by using structural analysis software i.e. STAAD Pro. The analysis results of structure for gravity and seismic load cases are compared to evaluate the effect of seismic forces on the RC structure. Weak zones are detected by comparing the results and retrofitting technique is suggested for the structure. Two cases for the compare of structure are

Case 1:- Structure with gravity loads only (STR-GR)

Case 2:- Structure with earthquake loads of Zone III in addition to gravity loads (STR-EQ)

## 2. MODELLING

Modelling is done for the structure, the details of which is illustrated in table

**Table 2.1 Details of structure for modelling**

| | |
|---|---|
| Structure type | RCC commercial building |
| Storeys | G + 2 |
| Height of each storey | 3.5m |
| Building plan size | 21m x 12.5m |
| Building height | 10.5m |
| Depth of foundation | 1.5m below GL |
| Type of supports | Fixed |
| Slab thickness each | 150mm |
| Column size each | 300mm x 300mm |
| Beam size | 200mm x 400mm |
| Type of wall separation | Glazed |
| Dead load of wall taken | Consider brick wall load |
| Live load on each floor | 4 $KN/m^2$ |
| Live load on terrace | 1.5 $KN/m^2$ |
| Seismic zone | Zone III |
| Live load with seismic force | 50% (IS 1893:2002) |
| Steel grade | Fe 415 |
| Characteristic strength of concrete ($f_{ck}$) | 25 $N/mm^2$ |

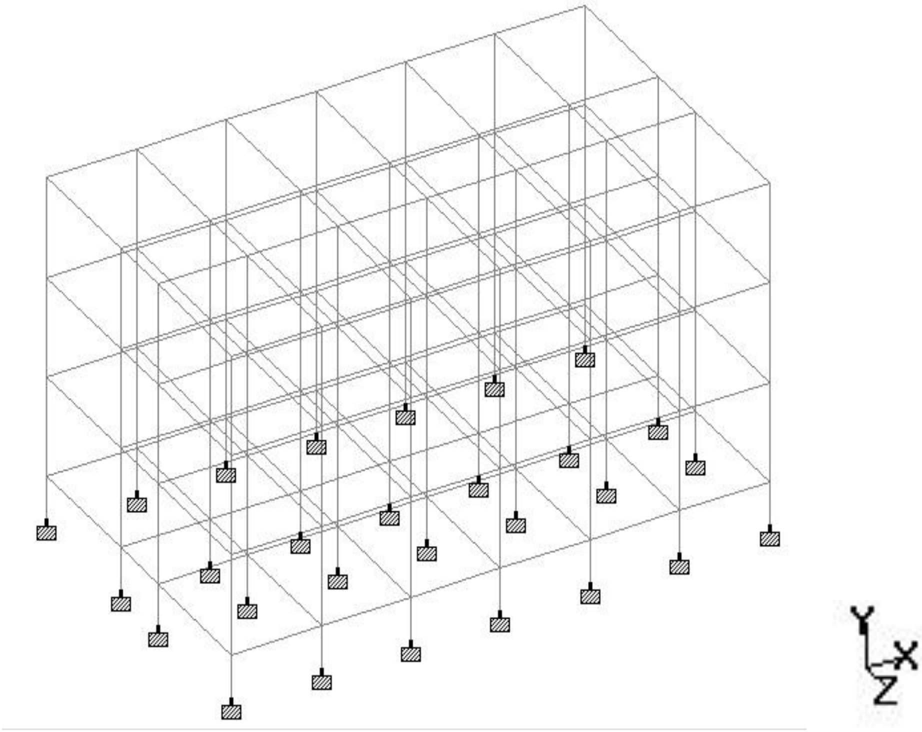


**Fig 2.1 Isometric view of proposed structure**

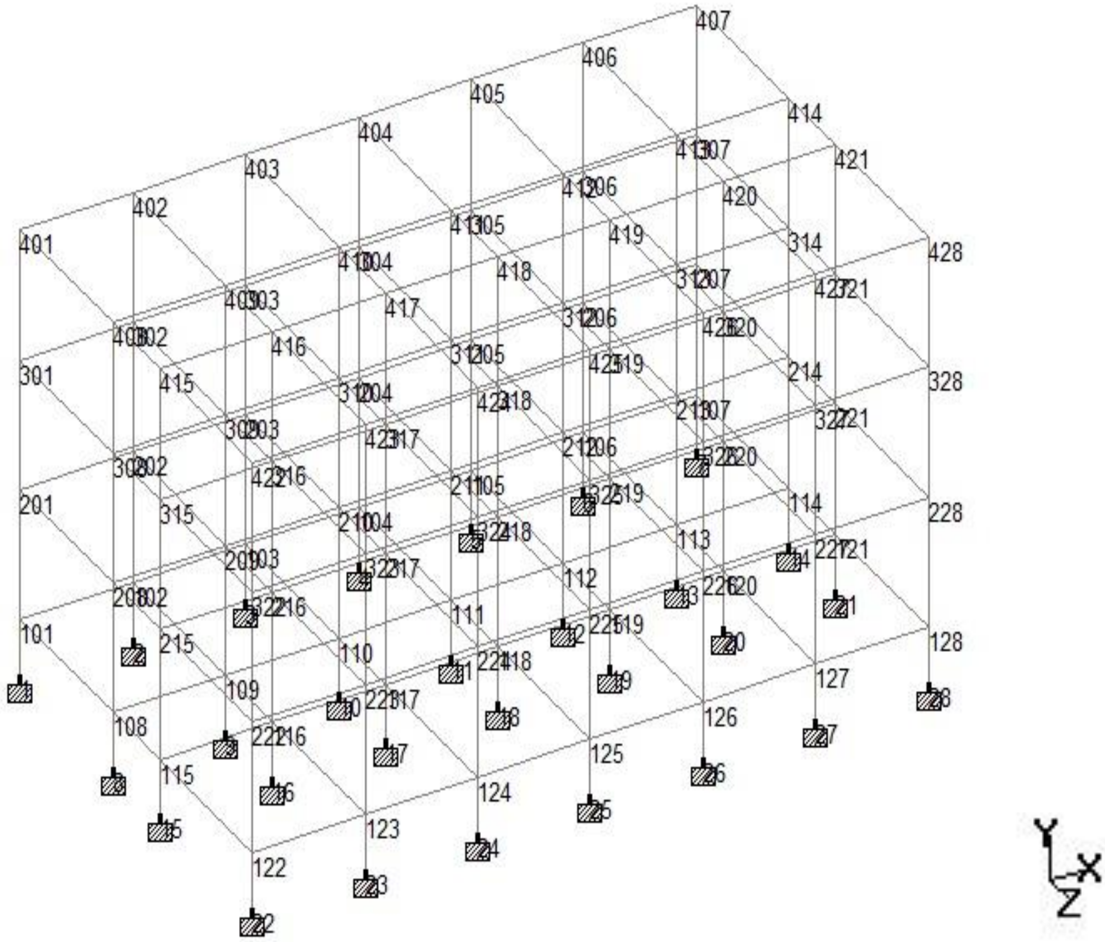


**Fig 2.2 Isometric view with node numbers**

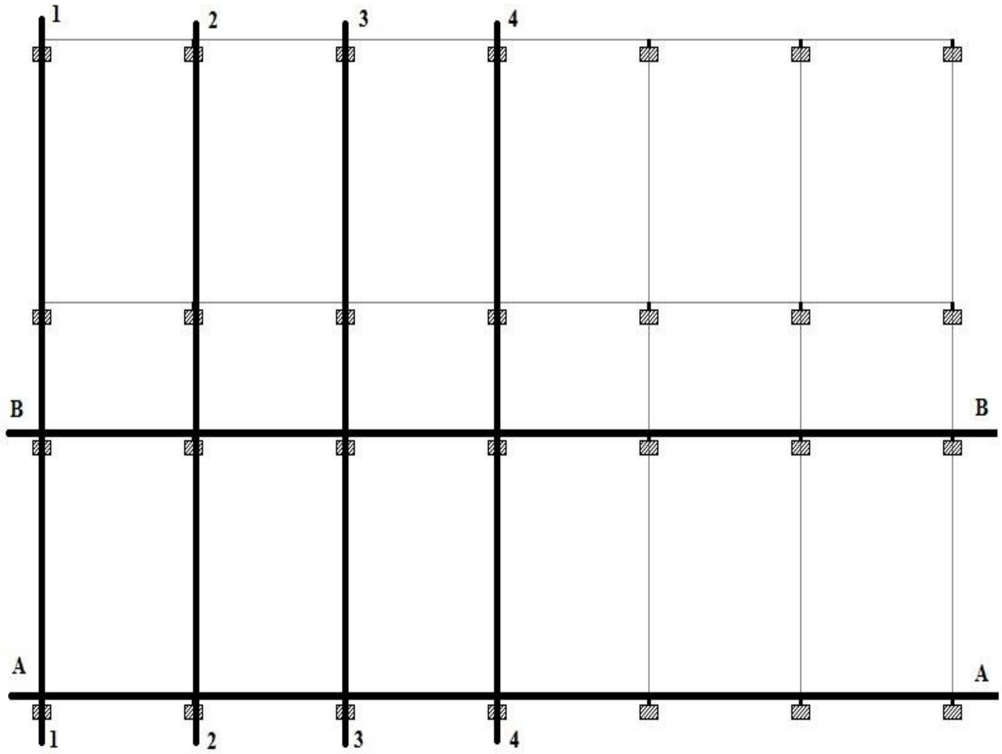


**Fig 2.3 Sections in plan where beams and columns are considered**

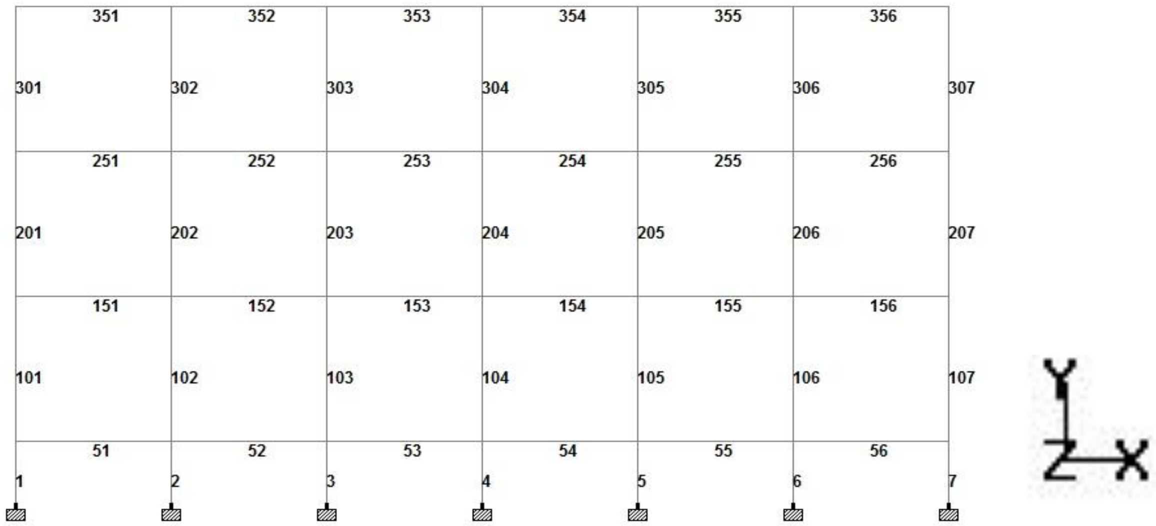


**Fig 2.4 Section A-A showing beam and column numbering**

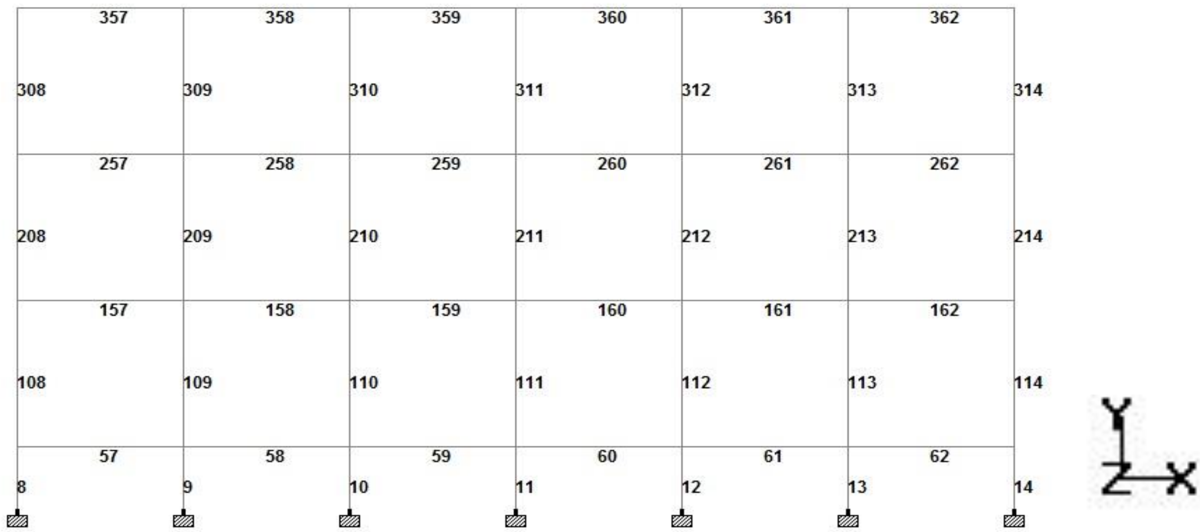


**Fig 2.5 Section B-B showing beam column numbering**

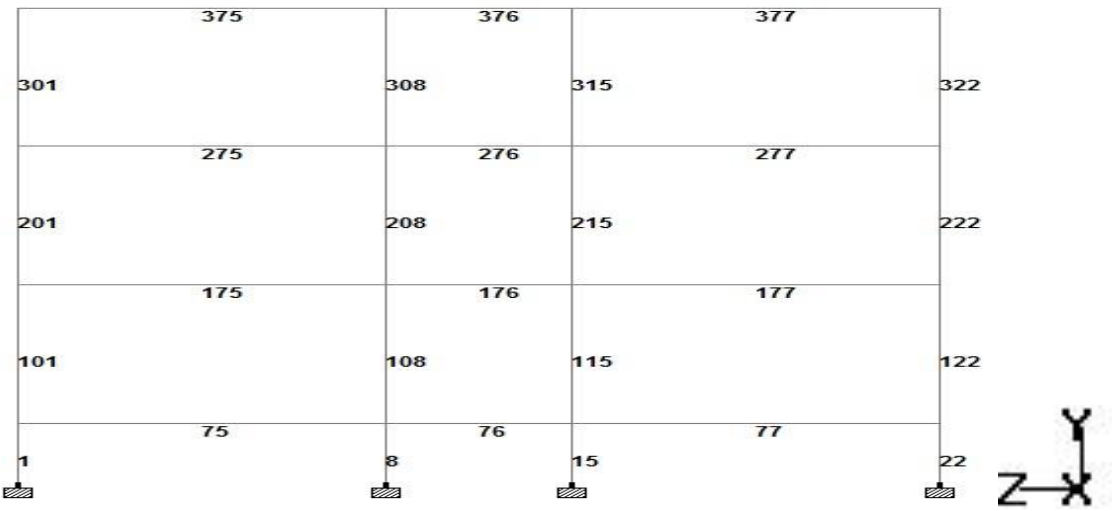


**Fig 2.6 Section 1-1 with showing beam column numbering**

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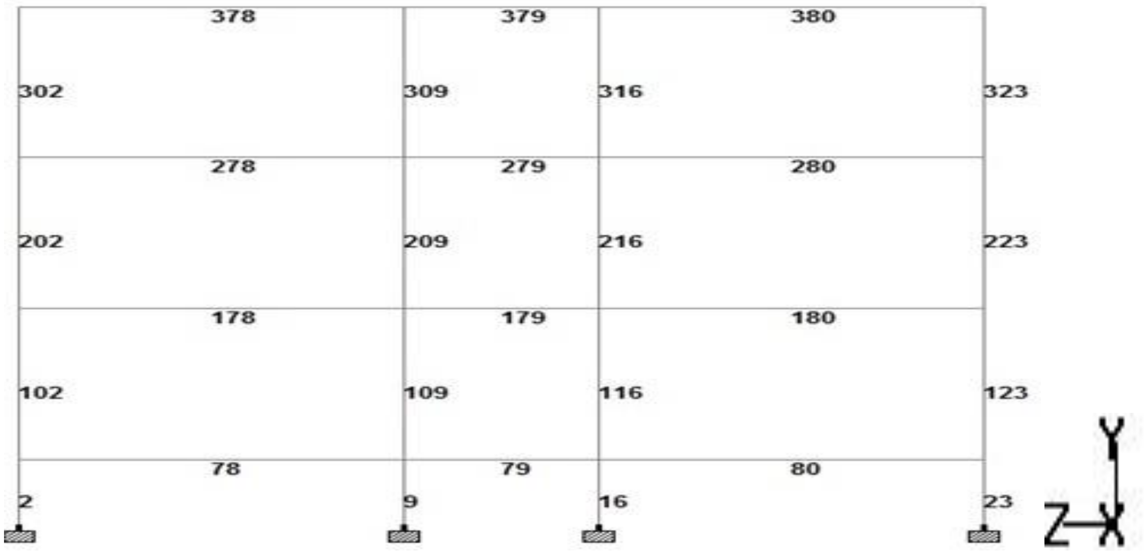


**Fig 2.7 Section 2-2 with showing beam column numbering**

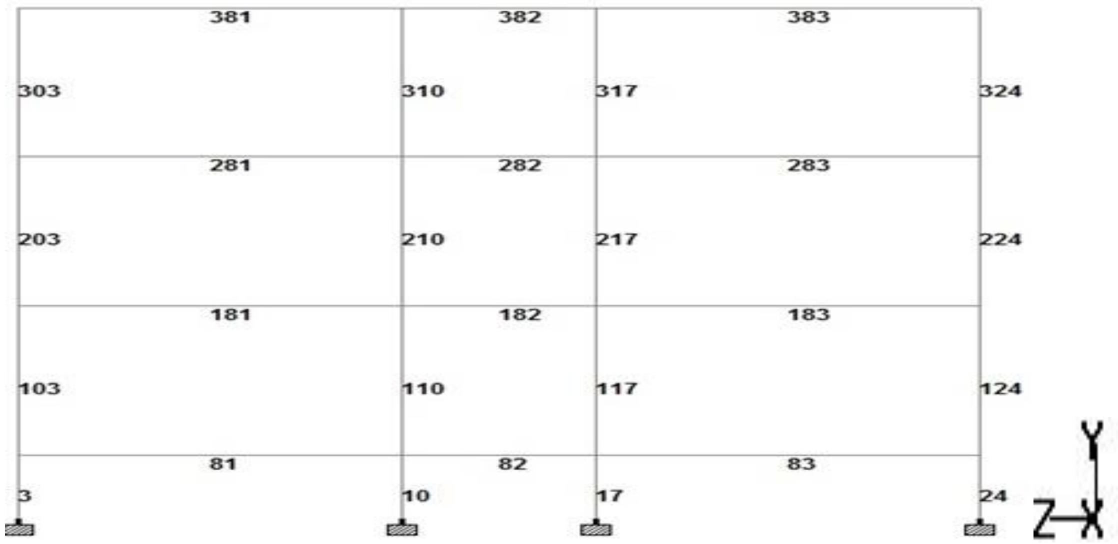


**Fig 2.8 Section 3-3 with showing beam column numbering**

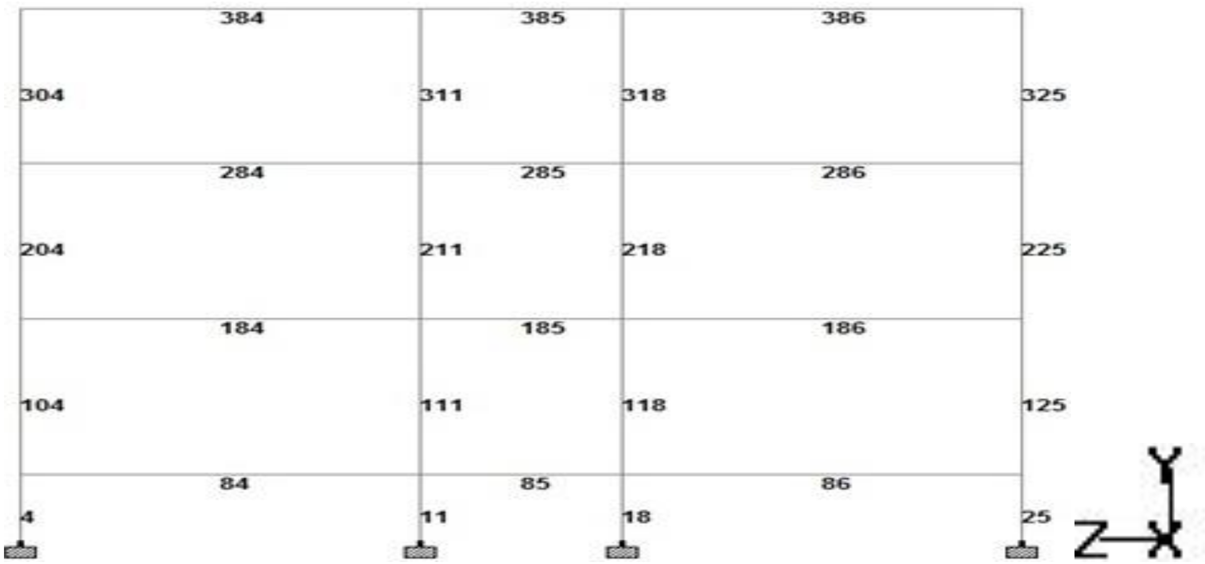


**Fig 2.9 Section 4-4 with showing beam column numbering**

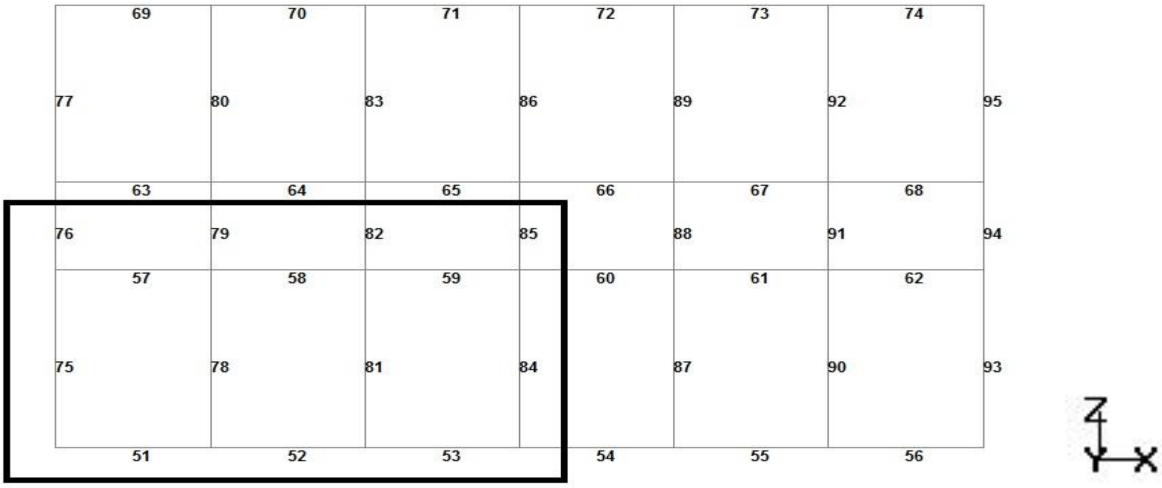


**Fig 2.10 Region from which Beams and columns are considered for comparison**

## 3. NOMENCLATURES

For the easy understanding of the geometry, numbers of nodes, beams and columns are given in a particular pattern. So that the location of the nodes, beams and columns can be identify easily.

**Table 3.1 Nomenclature of node numbers in structure**

| Level | Node no |
|---|---|
| Footing level | 1 to 28 |
| Plinth level | 101 to 128 |
| First floor level | 201 to 228 |
| Second floor level | 301 to 328 |
| Third floor level | 401 to 428 |

**Table 3.2 Nomenclature of column numbers in structure**

| Storey | Column no |
|---|---|
| Below ground | 1 to 28 |
| First storey | 101 to 128 |
| Second storey | 201 to 228 |
| Third storey | 301 to 328 |

**Table 3.3 Nomenclature of beam numbers in structure**

| Level | Node no |
|---|---|
| Plinth level | 51 to 95 |
| First floor level | 151 to 195 |
| Second floor level | 251 to 295 |
| Third floor level | 351 to 395 |

## 4. Load calculation

Dead load and live loads are calculated and tabulated below for the analysis of the structure.

**Table 4.1 Dead load and Live load on structure**

| Members | Load calculation | Load |
|---|---|---|
| Dead load of 200mm wall | 0.2 x 3.1 x 20 | 12.4 kN/m |
| Dead load of 100mm wall | 0.1 x 3.1 x 20 | 6.20 kN/m |
| Dead load of parapet wall of 100 mm | 0.1 x 1 x 20 | 2.00 kN/m |

| | | |
|---|---|---|
| Dead load of slab | 0.15 x 25 | 3.75 $kN/m^2$ |
| Live load on floors | By IS code | 4.00 $kN/m^2$ |
| Live load on roof | By IS code | 1.50 $kN/m^2$ |

Earth quake load

Determination of base shear

$V_B = A_h W$ …………………………………. (1)

$V_B$ = Base shear

$A_h$ = Horizontal seismic coefficient

W = Seismic weight of building

$A_h = (\frac{Z}{2})(\frac{I}{R})(\frac{S_a}{g})$ ……………………….. (2)

Z = Zone factor

I = Importance factor

R = Response reduction factor

$\frac{S_a}{g}$ = Spectral acceleration

$Q_i = V_B \frac{W_i h_i^2}{\sum_i^n W_i h_i^2}$ ……………………….. (3)

$Q_i$ = Design lateral force at floor i

$W_i$ = Seismic weight of floor i

$h_i$ = Height of floor i measured from base

n = Number of stories in the building is the number of levels at which masses are located

Time period

$T = 0.075\ h^{0.75}$

h = Height of the building

For our case

Z = zone III = 0.16

I = 1 (All general building)

R = 5 (Assume special RC moment resisting frame)

$\frac{S_a}{g}$ = 2.5 (both in x and z direction)

Period in X direction = 0.21

Period in Z direction = 0.27

## 5. LOAD COMBINATION: - ACCORDING TO IS 1893:2002

Basic loads

DL = Dead load

LL = Live load

EQ X = EQ in +X direction

EQ-X = EQ in -X direction

EQ Z = EQ in +Z direction

EQ-Z = EQ in-Z direction

Combination of loads according to IS 1893:2002

1) 1.5 DL + 1.5 LL
2) 1.2 DL + 1.2 LL + 1.2 EQ X
3) 1.2 DL + 1.2 LL + 1.2 EQ-X
4) 1.2 DL + 1.2 LL + 1.2 EQ Z
5) 1.2 DL + 1.2 LL + 1.2 EQ-Z
6) 1.5 DL + 1.5 EQ X
7) 1.5 DL + 1.5 EQ-X
8) 1.5 DL + 1.5 EQ Z
9) 1.5 DL + 1.5 EQ-Z
10) 0.9 DL + 1.5 EQ X
11) 0.9 DL + 1.5 EQ-X
12) 0.9 DL + 1.5 EQ Z
13) 0.9 DL + 1.5 EQ-Z

## 6. BOUNDARY CONDITIONS AND ANALYSIS

It is assumed that all the supports at foundation level are fixed. The effect of soil structural interaction is not considered in the analysis.

In present study structure is analysed for following two cases,

Case 1 Gravity load case (STR-GR)

The structure is analysed for dead load and live load applied on the structure.

Case 2 STR-EQ (Seismic load case)

The structure is analysed for seismic forces in addition to gravity load applied on the structure.

## 7. RESULTS AND DISCUSSION

In this chapter the effects of the earthquake forces on structure is studied in addition to gravity forces. The comparison of shear forces, axial forces, bending moments and reinforcement is done for two cases i.e. for STR-GR and STR-EQ structure and their differences are tabulated to estimate the strengthening requirement for the additional load. Floor wise results are discussed for different beams and columns. Subsequently the retrofitting method is used to strengthen the weak members.

In results STR-GR indicates the results of structure analysed with gravity forces only and STR-EQ indicates the results of structure analysed with earthquake force in addition to gravity forces.

### 7.1 Effects of Additional Seismic Force on Beams

The shear force, bending moment and area of reinforcing steel in beams and columns of different storeys are presented and compared for gravity and seismic load cases.

#### *7.1.1 Effect on shear force in beam*

The shear force in both the cases as for STR-GR and STR-EQ are compared for beams at each floor.

*a) Plinth beams*

The shear force in plinth beams for gravity and seismic load cases are discussed. The increase in shear force due to application of earthquake forces in addition to gravity forces are shown in table 7.1.

**Table 7.1 Comparison of Shear force Fy (kN) in plinth beams between gravity and seismic load case**

| Beam No | Shear force Fy | | Increase in Shear force | % increase in shear force |
|---|---|---|---|---|
| | STR-GR | STR-EQ | | |
| | (1) | (2) | (2)-(1) | |
| 51 | 39.10 | 57.53 | 18.43 | 47.14 |
| 52 | 37.57 | 52.87 | 15.30 | 40.72 |
| 53 | 37.51 | 52.94 | 15.43 | 41.14 |
| 57 | 21.61 | 41.19 | 19.58 | 90.61 |
| 58 | 21.25 | 37.31 | 16.06 | 75.58 |
| 59 | 21.24 | 37.48 | 16.24 | 76.46 |
| 75 | 54.34 | 65.30 | 10.96 | 20.17 |
| 76 | 26.78 | 56.28 | 29.50 | 110.16 |
| 78 | 30.43 | 42.24 | 11.81 | 38.81 |
| 79 | 15.16 | 46.57 | 31.41 | 207.19 |
| 81 | 30.41 | 42.67 | 12.26 | 40.32 |
| 82 | 15.16 | 47.80 | 32.64 | 215.30 |
| 84 | 30.40 | 42.80 | 12.40 | 40.79 |
| 85 | 15.16 | 48.17 | 33.01 | 217.74 |

From the above comparison it is revealed that there is an increase in shear force Fy in all the beams. The maximum increase in shear force is found to be 33.01 kN in beam no 85 with percentage increase of 217.74%.

*b) First floor beams*

The shear force in first floor beams for gravity and seismic load cases are discussed. Increase in shear force due to application of earthquake forces in addition to gravity forces are shown in table 7.2.

**Table 7.2 Comparison of Shear force (kN) in first floor beams between gravity and seismic load case**

| Beam No | Shear force Fy | | Increase In Shear force | % increase in shear force |
|---|---|---|---|---|
| | STR-GR | STR-EQ | | |
| 151 | 58.35 | 67.40 | 9.05 | 15.51 |
| 152 | 55.47 | 67.05 | 11.58 | 20.88 |
| 153 | 55.32 | 67.42 | 12.10 | 21.87 |
| 157 | 58.93 | 69.01 | 10.08 | 17.11 |
| 158 | 55.61 | 62.04 | 6.43 | 11.56 |
| 159 | 55.41 | 62.30 | 6.89 | 12.43 |
| 175 | 88.83 | 88.83 | 0.00 | 0.00 |
| 176 | 35.87 | 69.19 | 33.32 | 92.89 |
| 178 | 99.11 | 99.11 | 0.00 | 0.00 |
| 179 | 33.32 | 64.68 | 31.36 | 94.12 |
| 181 | 99.15 | 99.15 | 0.00 | 0.00 |
| 182 | 33.32 | 66.29 | 32.97 | 98.95 |
| 184 | 99.15 | 99.15 | 0.00 | 0.00 |
| 185 | 33.32 | 66.75 | 33.43 | 100.33 |

From the above comparison it is revealed that there is an increase in shear force Fy in all the beams. The maximum increase in shear force is found to be 33.43 kN in beam no 185 with percentage increase is 100.33%.

*c) Second floor beam*

The shear force in second floor beams for gravity and seismic load cases are discussed. Increase in shear force due to application of earthquake forces in addition to gravity forces are shown in table 7.3.

**Table 7.3 Comparison of Shear force (kN) in second floor beams between gravity and seismic load case**

| Beam No | Shear force Fy | | Increase in Shear force | % increase in shear force |
|---|---|---|---|---|
| | STR-GR | STR-EQ | | |
| 251 | 57.57 | 65.61 | 8.04 | 13.97 |
| 252 | 55.39 | 60.83 | 5.44 | 9.82 |
| 253 | 55.31 | 60.92 | 5.61 | 10.14 |
| 257 | 57.75 | 60.99 | 3.24 | 5.61 |
| 258 | 55.50 | 56.96 | 1.46 | 2.63 |
| 259 | 55.37 | 56.93 | 1.56 | 2.82 |
| 275 | 88.48 | 88.48 | 0.00 | 0.00 |
| 276 | 35.87 | 57.41 | 21.54 | 60.05 |
| 278 | 98.34 | 98.34 | 0.00 | 0.00 |
| 279 | 33.32 | 52.29 | 18.97 | 56.93 |
| 281 | 98.34 | 98.34 | 0.00 | 0.00 |
| 282 | 33.32 | 53.49 | 20.17 | 60.53 |
| 284 | 98.34 | 98.34 | 0.00 | 0.00 |
| 285 | 33.32 | 53.83 | 20.51 | 61.55 |

From the above comparison it is revealed that there is an increase in shear force Fy in all the beams. The maximum increase in shear force is found to be 21.54 kN in beam no 276 with percentage increase of 60.05%.

*d) Third floor beam*

The shear force in third beams for gravity and seismic load cases are discussed. Increase in shear force due to application of earthquake forces in addition to gravity forces are shown in table 7.4.

**Table 7.4 Comparison of Shear force (kN) in third floor beams between gravity and seismic load case**

| Beam No | Shear force Fy | | Increase In Shear force | % increase in shear force |
|---|---|---|---|---|
| | STR-GR | STR-EQ | | |
| 351 | 22.79 | 25.78 | 2.99 | 13.12 |
| 352 | 22.70 | 24.55 | 1.85 | 8.15 |
| 353 | 22.34 | 24.19 | 1.85 | 8.28 |
| 357 | 29.51 | 29.82 | 0.31 | 1.05 |
| 358 | 28.41 | 28.41 | 0.00 | 0.00 |
| 359 | 28.14 | 28.14 | 0.00 | 0.00 |
| 375 | 38.21 | 38.21 | 0.00 | 0.00 |
| 376 | 13.44 | 19.78 | 6.34 | 47.17 |
| 378 | 53.93 | 53.93 | 0.00 | 0.00 |
| 379 | 15.84 | 21.14 | 5.30 | 33.46 |
| 381 | 54.01 | 54.01 | 0.00 | 0.00 |
| 382 | 15.84 | 21.56 | 5.72 | 36.11 |
| 384 | 54.01 | 54.01 | 0.00 | 0.00 |
| 385 | 15.84 | 21.67 | 5.83 | 36.80 |

From the above comparison it is revealed that there is an increase in shear force Fy in all the beams. The maximum increase in shear force is found to be 6.34 kN in beam no 376 with percentage increase of 47.17%.

## 8. EFFECT ON BENDING MOMENT IN BEAM

Bending moment and corresponding reinforcement area of steel in beam are discussed. Sagging moment and hogging moment both are compared for the two cases as for STR-GR and STR-EQ. Maximum of two hogging moments from both ends are taken for the comparison.

***a) Plinth level beams:***

**Table 8.1 Comparison of bending moment Mz (kNm) and corresponding reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in beams at plinth level**

| Beam no | STR-GR | | | | STR-EQ (Zone III) | | | | Increase in moment/ reinforcement | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Hogging moment | Sagging moment | Ast Top | Ast Bottom |
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | (5-1) | (6-2) | (7-3) | (8-4) |
| 51 | -23.00 | 12.61 | 226 | 226 | -53.00 | 24.40 | 565 | 226 | -30.00 | 11.79 | 339 | 0 |
| 52 | -22.08 | 10.86 | 226 | 226 | -48.72 | 16.31 | 452 | 226 | -26.64 | 5.45 | 226 | 0 |
| 53 | -21.89 | 10.94 | 226 | 226 | -48.96 | 16.36 | 452 | 226 | -27.07 | 5.42 | 226 | 0 |
| 57 | -12.34 | 6.91 | 226 | 226 | -45.32 | 29.64 | 402 | 339 | -32.98 | 22.73 | 176 | 113 |
| 58 | -12.41 | 6.21 | 226 | 226 | -40.61 | 20.84 | 339 | 226 | -28.20 | 14.63 | 113 | 0 |
| 59 | -12.42 | 6.18 | 226 | 226 | -40.84 | 21.04 | 339 | 226 | -28.42 | 14.86 | 113 | 0 |
| 75 | -42.68 | 26.20 | 402 | 226 | -67.26 | 31.80 | 603 | 339 | -24.58 | 5.60 | 201 | 113 |
| 76 | -17.30 | 0.00 | 226 | 226 | -54.69 | 26.18 | 565 | 226 | -37.39 | 26.18 | 339 | 0 |
| 78 | -24.18 | 14.00 | 226 | 226 | -53.24 | 22.74 | 452 | 226 | -29.06 | 8.74 | 226 | 0 |
| 79 | -8.60 | 0.87 | 226 | 226 | -48.85 | 33.52 | 452 | 339 | -40.25 | 32.65 | 226 | 113 |
| 81 | -24.19 | 13.92 | 226 | 226 | -54.57 | 23.13 | 565 | 226 | -30.38 | 9.21 | 339 | 0 |
| 82 | -8.49 | 0.98 | 226 | 226 | -50.30 | 35.11 | 452 | 339 | -41.81 | 34.13 | 226 | 113 |
| 84 | -24.19 | 13.92 | 226 | 226 | -54.94 | 23.26 | 565 | 226 | -30.75 | 9.34 | 339 | 0 |
| 85 | -8.49 | 0.99 | 226 | 226 | -50.75 | 35.57 | 452 | 339 | -42.26 | 34.58 | 226 | 113 |

***b) First floor beams:***

**Table 8.2 Comparison of bending moment Mz (kNm) and corresponding reinforcement area Ast ($mm^2$ ) between Gravity and Seismic analysis in beams at first floor**

| Beam no | STR-GR | | | | STR-EQ (Zone III) | | | | Increase in moment/ reinforcement | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Hogging moment | Sagging moment | Ast Top | Ast Bottom |
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | (5-1) | (6-2) | (7-3) | (8-4) |
| 151 | -36.27 | 22.66 | 339 | 226 | -70.68 | 37.30 | 628 | 339 | -34.41 | 14.64 | 289 | 113 |
| 152 | -35.35 | 18.53 | 339 | 226 | -64.86 | 22.10 | 565 | 226 | -29.51 | 3.56 | 226 | 0 |
| 153 | -34.91 | 18.71 | 339 | 226 | -65.45 | 22.10 | 603 | 226 | -30.54 | 3.36 | 264 | 0 |
| 157 | -38.96 | 25.07 | 339 | 226 | -69.37 | 42.10 | 628 | 402 | -30.41 | 17.05 | 289 | 176 |
| 158 | -37.78 | 20.44 | 339 | 226 | -63.09 | 24.40 | 565 | 226 | -25.31 | 3.92 | 226 | 0 |
| 159 | -37.16 | 20.72 | 339 | 226 | -63.70 | 24.80 | 565 | 226 | -26.54 | 4.11 | 226 | 0 |
| 175 | -73.03 | 52.63 | 678 | 452 | -93.22 | 52.60 | 904 | 452 | -20.19 | 0.00 | 226 | 0 |
| 176 | -33.08 | 0.00 | 339 | 226 | -73.70 | 31.80 | 678 | 339 | -40.62 | 31.80 | 339 | 113 |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 178 | -86.00 | 64.96 | 804 | 565 | -99.69 | 65.00 | 942 | 565 | -13.69 | 0.00 | 138 | 0 |
| 179 | -37.49 | 0.00 | 339 | 226 | -75.02 | 36.40 | 791 | 339 | -37.53 | 36.44 | 452 | 113 |
| 181 | -85.99 | 65.07 | 804 | 565 | -100.90 | 65.10 | 981 | 565 | -14.91 | 0.00 | 177 | 0 |
| 182 | -37.65 | 0.00 | 339 | 226 | -77.07 | 38.40 | 791 | 339 | -39.42 | 38.43 | 452 | 113 |
| 184 | -85.99 | 65.08 | 804 | 565 | -101.30 | 65.10 | 981 | 565 | -15.26 | 0.00 | 177 | 0 |
| 185 | -37.66 | 0.00 | 339 | 226 | -77.66 | 39.00 | 791 | 339 | -40.00 | 39.01 | 452 | 113 |

***c) Second floor beam:***

**Table 8.3 Comparison of bending moment Mz (kNm) and corresponding reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in beams at second floor**

| Beam no | STR-GR | | | | STR-EQ (Zone III) | | | | Increase in moment/ reinforcement | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Hogging moment | Sagging moment | Ast Top | Ast Bottom |
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | (5-1) | (6-2) | (7-3) | (8-4) |
| 251 | -34.96 | 22.60 | 339 | 226 | -56.50 | 26.70 | 565 | 226 | -21.54 | 4.13 | 226 | 0 |
| 252 | -35.04 | 18.69 | 339 | 226 | -54.17 | 18.70 | 565 | 226 | -19.13 | 0.00 | 226 | 0 |
| 253 | -34.90 | 18.70 | 339 | 226 | -54.08 | 18.70 | 565 | 226 | -19.18 | 0.00 | 226 | 0 |
| 257 | -37.28 | 24.68 | 339 | 226 | -54.70 | 26.10 | 565 | 226 | -17.42 | 1.40 | 226 | 0 |
| 258 | -37.33 | 20.70 | 339 | 226 | -52.12 | 20.70 | 452 | 226 | -14.79 | 0.00 | 113 | 0 |
| 259 | -37.12 | 20.69 | 339 | 226 | -51.99 | 20.70 | 452 | 226 | -14.87 | 0.00 | 113 | 0 |
| 275 | -72.70 | 52.07 | 678 | 452 | -82.64 | 52.10 | 791 | 452 | -9.94 | 0.00 | 113 | 0 |
| 276 | -32.15 | 0.00 | 339 | 226 | -58.53 | 17.40 | 565 | 226 | -26.38 | 17.35 | 226 | 0 |
| 278 | -85.78 | 63.27 | 791 | 565 | -90.84 | 63.30 | 904 | 565 | -5.06 | 0.00 | 113 | 0 |
| 279 | -34.94 | 0.00 | 339 | 226 | -57.85 | 22.00 | 565 | 226 | -22.91 | 21.98 | 226 | 0 |
| 281 | -85.77 | 63.28 | 791 | 565 | -91.78 | 63.30 | 904 | 565 | -6.01 | 0.00 | 113 | 0 |
| 282 | -34.97 | 0.00 | 339 | 226 | -59.28 | 23.50 | 565 | 226 | -24.31 | 23.51 | 226 | 0 |
| 284 | -85.77 | 63.28 | 791 | 565 | -92.04 | 63.30 | 904 | 565 | -6.27 | 0.00 | 113 | 0 |
| 285 | -34.97 | 0.00 | 339 | 226 | -59.70 | 23.90 | 565 | 226 | -24.73 | 23.94 | 226 | 0 |

***d) Third floor beam:***

**Table 8.4 Comparison of bending moment Mz (kNm) and corresponding reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in beams at third floor**

| Beam no | STR-GR | | | | STR-EQ (Zone III) | | | | Increase in moment/ reinforcement | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Max. hogging moment | Max. Sagging moment | Ast Top | Ast Bottom | Hogging moment | Sagging moment | Ast Top | Ast Bottom |
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | (5-1) | (6-2) | (7-3) | (8-4) |
| 351 | -13.16 | 10.76 | 226 | 226 | -21.16 | 11.80 | 226 | 226 | -8.00 | 1.04 | 0 | 0 |
| 352 | -15.21 | 8.57 | 226 | 226 | -22.11 | 8.57 | 226 | 226 | -6.90 | 0.00 | 0 | 0 |
| 353 | -14.96 | 8.17 | 226 | 226 | -21.65 | 8.17 | 226 | 226 | -6.69 | 0.00 | 0 | 0 |
| 357 | -18.71 | 14.70 | 226 | 226 | -25.42 | 14.70 | 226 | 226 | -6.71 | 0.00 | 0 | 0 |
| 358 | -20.01 | 11.47 | 226 | 226 | -25.33 | 11.50 | 226 | 226 | -5.32 | 0.00 | 0 | 0 |
| 359 | -19.79 | 11.22 | 226 | 226 | -24.95 | 11.20 | 226 | 226 | -5.16 | 0.00 | 0 | 0 |
| 375 | -31.43 | 25.94 | 339 | 226 | -36.60 | 25.90 | 339 | 226 | -5.17 | 0.00 | 0 | 0 |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 376 | -16.10 | 0.00 | 226 | 226 | -23.63 | 2.03 | 226 | 226 | -7.53 | 2.03 | 0 | 0 |
| 378 | -46.49 | 39.59 | 402 | 339 | -46.88 | 39.60 | 402 | 339 | -0.39 | 0.00 | 0 | 0 |
| 379 | -23.54 | 0.00 | 226 | 226 | -29.54 | 0.00 | 339 | 226 | -6.00 | 0.00 | 113 | 0 |
| 381 | -46.49 | 39.78 | 402 | 339 | -47.42 | 39.80 | 402 | 339 | -0.93 | 0.00 | 0 | 0 |
| 382 | -23.82 | 0.00 | 226 | 226 | -30.28 | 0.30 | 339 | 226 | -6.46 | 0.30 | 113 | 0 |
| 384 | -46.48 | 39.79 | 402 | 339 | -47.57 | 39.80 | 402 | 339 | -1.09 | 0.00 | 0 | 0 |
| 385 | -23.83 | 0.00 | 226 | 226 | -30.44 | 0.00 | 339 | 226 | -6.61 | 0.00 | 113 | 0 |

Table 8.1 shows the bending moment and corresponding reinforcement area for plinth beams. Here the increase in hogging moment is maximum for beam no 85 as the value is increased by 42.26 kNm. Maximum increase in sagging moment is in the same beam with the value is increased by 34.58 kNm. The increase in reinforcement area for maximum increase in hogging moment at this level beams is 339 $mm^2$ in beam no 51, 76, 81, 84 and increase in reinforcement area for maximum increase in sagging moment at this level beam is 113 $mm^2$ in beam no (57, 75, 79, 82, 85).

Table 8.2 shows the bending moment and corresponding reinforcement area for first floor beams. Here the increase in hogging moment is maximum for beam no 176 as the value is increased by 40.62 kNm. Maximum increase in sagging moment is in beam no 185 with the value is increased by 39.01 kNm. The increase in reinforcement area for maximum increase in hogging moment at this floor beams is 452 $mm^2$ in beams no 179, 182, 185 and increase in reinforcement area for maximum increase in sagging moment at this level beam is 176 $mm^2$ in beam no 157.

Table 8.3 shows the bending moment and corresponding reinforcement area for second floor beams. Here the increase in hogging moment is maximum for beam no 276 as the value is increased by 26.38 kNm. Maximum increase in sagging moment is in beam no 285 with the value is increased by 23.94 kNm. The increase in reinforcement area for maximum increase in hogging moment at this floor beam is 226 $mm^2$ in beams no 251, 252, 253, 257, 276, 279, 282, 285 and there is no increase in reinforcement area for sagging moment in any beam.

Table 8.4 shows the bending moment and corresponding reinforcement area for third floor beams. Here the increase in hogging moment is maximum for beam no 351 as the value is increased by 8 kNm. Maximum increase in sagging moment is in beam no 376 with the value is increased by 2.03 kNm. The increase in reinforcement area for maximum increase in hogging moment at this floor beam is 113 $mm^2$ . In beams no 379, 382, 385 and there is no increase in reinforcement area for sagging moment in any beam.

## 9. EFFECTS OF ADDITIONAL SEISMIC FORCE ON COLUMNS

Axial force, bending moment in X and Y direction, and area of reinforcement steel are compared for both the cases as STR-GR and STR-EQ.

***a) Columns below plinth:***

**Table 9.1 Comparison of Axial force (kN), bending moment (kNm) and reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in columns below plinth level**

| Column No. | STR-GR | | | | STR-EQ Z III | | | | Increase in forces/reinforcement | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Max of Mz& My |
| 1 | 458.78 | 3.65 | 8.05 | 329 | 464.76 | 24.45 | 8.51 | 333 | 5.98 | 20.80 | 0.46 | 4 | 20.80 |
| 2 | 654.32 | 0.41 | 3.59 | 469 | 654.32 | 0.41 | 3.59 | 469 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |
| 3 | 647.38 | 0.09 | 3.44 | 464 | 647.38 | 0.09 | 3.44 | 464 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |
| 4 | 647.01 | 0.00 | 3.44 | 464 | 647.01 | 0.00 | 3.44 | 464 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |
| 8 | 572.27 | 2.01 | 5.52 | 411 | 572.27 | 2.01 | 5.52 | 411 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |
| 9 | 745.23 | 0.13 | 2.01 | 535 | 745.23 | 0.13 | 2.01 | 535 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |
| 10 | 738.10 | 0.02 | 1.90 | 530 | 738.10 | 0.02 | 1.90 | 530 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |
| 11 | 737.74 | 0.00 | 1.90 | 529 | 737.74 | 0.00 | 1.90 | 529 | 0.00 | 0.00 | 0.00 | 0 | 0.00 |

***b) Columns at first storey:***

**Table 9.2 Comparison of Axial force (kN), bending moment (kNm) and reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in columns at first storey**

| Column No. | STR-GR | | | | STR-EQ Z III | | | | Increase in forces/reinforcement | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Max of Mz& My |
| 101 | 365.31 | 8.82 | 20.24 | 262 | 342.58 | 37.51 | 22.48 | 860 | -22.73 | 28.69 | 2.24 | 598 | 28.69 |
| 102 | 542.68 | 0.28 | 17.27 | 389 | 236.61 | 41.13 | 12.01 | 877 | -306.07 | 40.85 | -5.26 | 488 | 40.85 |
| 103 | 537.47 | 0.08 | 17.19 | 386 | 242.34 | 40.08 | 12.00 | 835 | -295.13 | 40.00 | -5.19 | 449 | 40.00 |
| 104 | 536.98 | 0.00 | 17.19 | 385 | 241.77 | 40.19 | 12.00 | 834 | -295.21 | 40.19 | -5.19 | 449 | 40.19 |
| 108 | 465.54 | 7.01 | 12.99 | 334 | 320.86 | 7.59 | 58.76 | 1220 | -144.68 | 0.58 | 45.77 | 886 | 45.77 |
| 109 | 652.06 | 0.19 | 11.55 | 468 | 408.20 | 0.55 | 60.68 | 1230 | -243.86 | 0.36 | 49.13 | 762 | 49.13 |
| 110 | 645.27 | 0.03 | 11.52 | 463 | 400.01 | 0.05 | 62.54 | 1292 | -245.26 | 0.02 | 51.02 | 829 | 51.02 |
| 111 | 644.99 | 0.00 | 11.52 | 463 | 399.61 | 0.00 | 63.08 | 1319 | -245.38 | 0.00 | 51.56 | 856 | 51.56 |

***c) Columns at second storey:***

**Table 9.3 Comparison of Axial force (kN), bending moment (kNm) and reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in columns at second storey**

| Column No. | STR-GR | | | | STR-EQ Z III | | | | Increase in forces/reinforcement | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Max of Mz& My |
| 201 | 206.38 | 14.19 | 34.01 | 758 | 194.19 | 36.75 | 25.88 | 1288 | -12.19 | 22.56 | -8.13 | 530 | 22.56 |
| 202 | 323.96 | 0.52 | 41.40 | 488 | 245.18 | 38.02 | 23.93 | 1191 | -78.78 | 37.50 | -17.47 | 703 | 37.50 |
| 203 | 322.21 | 0.02 | 41.64 | 509 | 248.91 | 29.27 | 31.69 | 1139 | -73.30 | 29.25 | -9.95 | 630 | 29.25 |
| 204 | 321.48 | 0.00 | 41.65 | 507 | 248.28 | 29.29 | 31.69 | 1139 | -73.20 | 29.29 | -9.96 | 632 | 29.29 |
| 208 | 266.77 | 14.89 | 22.03 | 212 | 194.02 | 9.30 | 57.62 | 1436 | -72.75 | -5.59 | 35.59 | 1224 | 35.59 |
| 209 | 393.95 | 0.68 | 27.74 | 283 | 254.18 | 0.51 | 60.39 | 1300 | -139.77 | -0.17 | 32.65 | 1017 | 32.65 |
| 210 | 391.20 | 0.01 | 27.90 | 281 | 249.92 | 0.04 | 62.11 | 1371 | -141.28 | 0.03 | 34.21 | 1090 | 34.21 |
| 211 | 390.57 | 0.00 | 27.90 | 280 | 249.52 | 0.00 | 62.60 | 1387 | -141.05 | 0.00 | 34.70 | 1107 | 34.70 |

***d) Columns at third storey:***

**Table 9.4 Comparison of Axial force (kN), bending moment (kNm) and reinforcement area Ast ($mm^2$) between Gravity and Seismic analysis in columns at third storey**

| Column No. | STR-GR | | | | STR-EQ Z III | | | | Increase in forces/reinforcement | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Axial force Pu | Bending moment Mz | Bending moment My | Area of steel As | Max of Mz& My |
| 301 | 68.57 | 12.84 | 29.78 | 902 | 52.28 | 8.99 | 33.60 | 947 | -16.29 | -3.85 | 3.82 | 45 | 3.82 |
| 302 | 105.52 | 0.11 | 36.41 | 670 | 74.57 | 19.10 | 25.20 | 931 | -30.95 | 18.99 | -11.21 | 261 | 18.99 |
| 303 | 94.75 | 0.24 | 35.70 | 683 | 75.23 | 18.99 | 25.47 | 934 | -19.52 | 18.75 | -10.23 | 251 | 18.75 |
| 304 | 94.06 | 0.00 | 35.73 | 685 | 74.65 | 18.71 | 25.49 | 936 | -19.41 | 18.71 | -10.24 | 251 | 18.71 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 308 | 89.43 | 14.04 | 18.83 | 551 | 59.77 | 9.18 | 33.63 | 926 | -29.66 | -4.86 | 14.80 | 375 | 14.80 |
| 309 | 138.16 | 0.02 | 23.95 | 253 | 94.16 | 0.41 | 38.98 | 780 | -44.00 | 0.39 | 15.03 | 527 | 15.03 |
| 310 | 137.53 | 0.18 | 24.11 | 262 | 92.48 | 0.13 | 40.22 | 820 | -45.05 | -0.05 | 16.11 | 558 | 16.11 |
| 311 | 137.02 | 0.00 | 24.11 | 262 | 92.32 | 0.00 | 40.52 | 826 | -44.70 | 0.00 | 16.41 | 564 | 16.41 |

Table 9.1 showing the comparison of axial force (Pu), bending moment in both direction (Mz& My) & area of reinforcement steel (Ast) in the columns below plinth level for STR-GR and STR-EQ. The maximum increase in axial force is found in column no 1 as 5.98 kN. The maximum increase in bending Mz is found in same column as 20.80 kNm. The maximum increase in My is found in same column as 0.46 kNm. The maximum increase in As is found in same column as $4\text{mm}^2$.

Table 9.2 showing the comparison of axial force (Pu), bending moment in both direction (Mz& My) & area of reinforcement steel (Ast) in the columns at ground storey for STR-GR and STR-EQ. In this storey axial force is decreasing in all the columns. So it needs not to compare. The maximum increase in bending Mz is found in column no 102 as 40.85 kNm. The maximum increase in My is found in column no 111 as 51.56 kNm. The maximum increase in As is found in column no 108 as 886 $\text{mm}^2$.

Table 9.3 showing the comparison of axial force (Pu), bending moment in both direction (Mz& My) & area of reinforcement steel (Ast) in the columns of first storey for STR-GR and STR-EQ. In this storey axial force is decreasing in all the columns. So it needs not to compare. The maximum increase in bending Mz is found in column no 202 as 37.50 kNm. The maximum increase in My is found in column no 208 as 35.59 kNm. The maximum increase in As is found in column no 208 as 1224 $\text{mm}^2$.

Table 9.4 showing the comparison of axial force (Pu), bending moment in both direction (Mz& My) & area of reinforcement steel (Ast) in the columns of second storey for STR-GR and STR-EQ. In this storey axial force is decreasing in all the columns. So it needs not to compare. The maximum increase in bending Mz is found in column no 302 as 18.99 kNm. The maximum increase in My is found in column no 311 as 16.41 kNm. The maximum increase in As is found in column no 311 as 564 $\text{mm}^2$.

## 10. STRENGTHENING OF BEAMS

Strengthening of beams is done for the flexure and shear, to reach the strength of the structural member up to the require strength.

***10.1 Strengthening of beams for flexure:***

Retrofitting is done for beams by adding steel plate of equivalent area of reinforced bars. Plate is designed for the additional area of steel required.

Equivalent mild steel area:-

The additional area of reinforcement bars are found by the comparison of analysis of both cases, the obtained additional required steel is of tor steel. But for the retrofitting mild steel plate is needed, the area of equivalent mild steel plate can be found by force equilibrium.

Force in Tor steel = Force in mild steel

$A_{st_1}$ x $f_{y_1}$ = $A_{st_2}$ x $f_{y_2}$

$A_{st_1}$ = Area of Tor steel

$f_{y_1}$ = yield stress of Tor steel

$A_{st_2}$ = Area of Mild steel

$f_{y_2}$ = yield stress of Mild steel

The above formula is used in the table itself to find the equivalent area of reinforcing steel.

For tor steel (Fe 415 $\text{N/mm}^2$ ) area up to 400 $\text{mm}^2$

$A_{st_1} = 400\ mm^2$

$f_{y_1} = 415\ N/mm^2$

$f_{y_2} = 250\ N/mm^2$

$A_{st_2}$ = Area of Mild steel

So $A_{st_2} = (\frac{415}{250}) \times 400 = 664\ mm^2$

Similarly equivalent area of mild steel, as given in table below

Design of steel plate for required additional reinforcement

Select different range from the tables for additional Ast ($mm^2$) of $f_y = 250\ N/mm^2$

**Table 10.1 Plate sizes showing for different range of equivalent mild steel area**

| Serial number | Additional reinforcement area (mm2) required (Fe 415) | Corresponding mild steel area (mm2) required (Fe 250) | Plate size used (mm x mm) |
|---|---|---|---|
| 1 | Up to 400 | 664 | 100 x 8 |
| 2 | 400-600 | 996 | 100 x 10 |
| 3 | 600 -800 | 1328 | 100 x 12 |

***10.2 Design of shear connector for flexure:***

Shear connector has to be design for every beam column joints for the maximum moment in that beam. Shear connector will transfer the additional force coming at existing reinforcement level to the outer plate which is designed for different beams. So the force which is to be transfer to the outer plate is to be calculated. These connectors are used for either top plate for hogging moment or bottom plate for sagging moment. As every beam will have different additional moment, the force for which shear connector will design will be different. Here shear connector is designed for the maximum moment developed among all the beams of the structure.

So for this, we have

Force = $\frac{\text{moment}}{\text{Lever arm}}$ ……………………………….……….$eq^n$ 1

Here lever arm L.A. = $(d-0.42x_u)$ ………………….$eq^n$ 2

But for $x_u$ ,

$M = 0.36\ f_{ck}\ x\ b\ x_u\ x\ (d-0.42x_u)$ ……………….…….$eq^n$ 3

Maximum additional moment = 42.26 kNm

Calculation of force for this maximum additional moment is given below,

Finding $x_u$ for max of sagging and hogging moment by $eq^n$ 3

Max hogging moment = -42.26 kNm

Therefore we have,

$42.26 \times 10^6 = 0.36 \times 25 \times 200\ x_u \times (367 - 0.42x_u)$

$42.26 \times 10^6 = 660600x_u - 756\ x_u^2$

$x_u = 69.50$ mm

Put this $x_u$ in $eq^n$ 2

L.A. = 367 – 0.42 x 69.50

L.A. = 337.81 mm

Now additional force which is to be carried by stud

$F = \frac{M}{L.A.}$

$F = \frac{42.26 \times 10^6}{337.81}$

F = 125099.91 N

Therefore,

F = 125.10 kN

Now designing the shear connector for the above force using IS 11384:1985 code

From table 1, we have

For 22 mm diameter of stud, 100 mm height and for M25 concrete

Strength of Shear connector F = 77.5 kN

Provide 2 shear connectors to resist the design shear force.

***10.3 Strengthening of beams for shear force***

Plates are used at side face of the beams for resist additional shear force.

The maximum force is taken among all the beams and from all the floors as 33.01 kN.

Take mild steel plate as Fe 250. Permissible stress for mild steel plate in shear is 140 $N/mm^2$

Area of steel plate = $\frac{\text{Force}}{\text{Permissible stress in plate}}$

So As = $\frac{33010}{140}$ = 235.79 $mm^2$

Assume depth of the plate is 200 mm

So thickness of plate will be $\frac{235.79}{200}$ = 1.179 mm≈ 2 mm

But for the practical purpose take plate of size 200mm x 4mm.

***10.4 Design of shear connector for shear force***

To transfer the shear stresses from existing shear reinforcement to outer plate, Shear connectors are used according to IS: 11384-1985.

As the maximum additional shear force among all the beams and from all the floors is 33.01 kN. So for this,

By table 1 of IS: 11384-1985 gives the Design strength of shear connectors for different concrete strengths.

Strength of shear connector for 12mm dia. and 62mm height used in M25 is 25.50 kN. So, two shear connectors are needed to resist shear force of 33.01 kN.

## 11. STRENGTHENING OF COLUMNS

Strengthening of columns is explained below for the additional moment due to seismic forces in addition to gravity forces.

The maximum additional moment, Mz or My is chosen from the columns of the structure. For these maximum moments, steel sections are designed. Assume an I section for the calculation of moment of resistance of column.

As if any column has axial load and moment simultaneously, it is designed by the beam bending theory. So,

According to the bending theory,

$M = f \times (\frac{I}{y})$ ………......................… (1)

Where

f= Maximum stress at extreme fibre

I=Moment of inertia of section

y =Distance of extreme fibre from CG of section

We know

$I = \frac{B \times D^3}{12} + (A \times h^2)$ …………………. (2)

Where

B= width of the section

D= depth of the section

A=area of the section whose inertia is to be determined

h=Distance between the CG of the whole section and CG of the section whose inertia is to be determined,

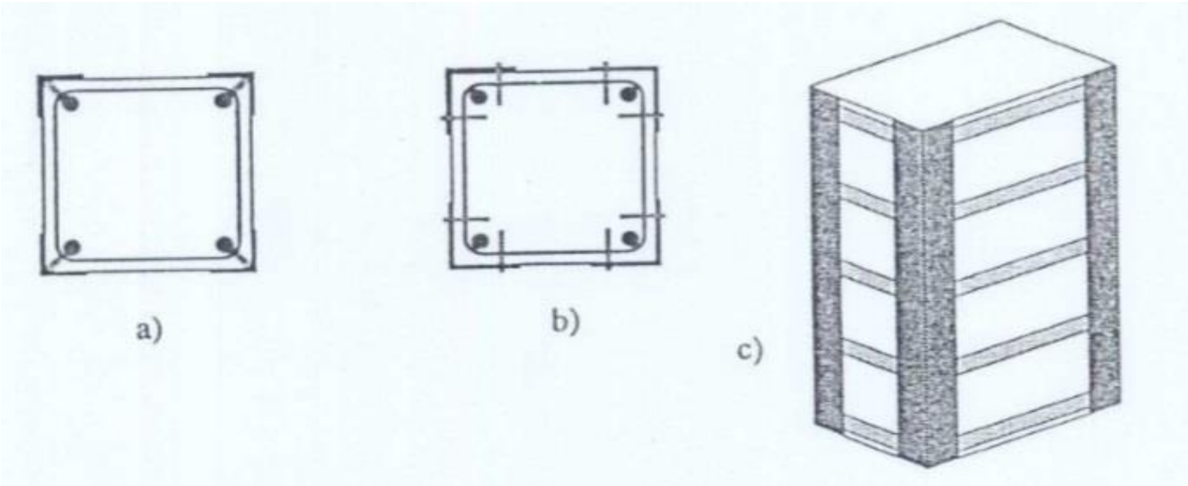


**Fig 11.1 Strengthening of columns by angles and battens**

Angles are placed at every corner of column with the help of epoxy glue and battens are placed at a spacing of 500 mm c/c, to reduce the slenderness ratio and to place the angles at fixed location. Strength of battens is not taken into account.

***11.1 Design of angle to resist additional moment***

Choose different ranges of Max of Mz and My (kNm) from tables

Assume angles to find the moment of resistance.

i) Take an angle section ISA 40404

Thickness = 4 mm

$I = 4.5 \times 10^4 mm^4$

$A = 307 \ mm^2$

CG = 11.2 mm

I for four angles

$I = 4[4.5 \times 10^4 + 307(150 + 4 - 11.2)^2]$

$I = 25.22 \times 10^6 mm^4$

By eq. 1

$M = 150 \times \frac{25.22 \times 10^6}{(150+4)}$

M = 24.57 kNm> 15 kNm

If the additional moment in column is less than 24.57 kNm, the angle section ISA 40404 can be used at that column at each corner.

Similarly moment of resistance of different angles are tabulated below in table 11.1

**Table 11.1: Moment of resistance of different angle sections**

| Angle section | Thickness (mm) | Area (mm2) | Centre of gravity (mm) | Moment of inertia (mm4) | Moment of resistance (kNm) |
|---|---|---|---|---|---|
| ISA 35x35 | 3 | 203 | 9.50 | 2.3 x 104 | 16.48 |
| | 4 | 266 | 10.0 | 2.9 x 104 | 21.60 |
| | 5 | 327 | 10.4 | 3.5 x 104 | 26.60 |
| | 6 | 386 | 10.8 | 4.1 x 104 | 31.46 |
| ISA 40x40 | 3 | 234 | 10.6 | 3.4 x 104 | 18.74 |
| | 4 | 307 | 11.2 | 4.5 x 104 | 24.57 |
| | 5 | 378 | 11.6 | 5.4 x 104 | 30.30 |
| | 6 | 447 | 12.0 | 6.3 x 104 | 36.89 |
| ISA 45x45 | 3 | 264 | 12.0 | 5.0 x 104 | 20.78 |
| | 4 | 347 | 12.5 | 6.5 x 104 | 27.32 |
| | 5 | 428 | 12.9 | 7.9 x 104 | 33.76 |
| | 6 | 507 | 13.3 | 9.2 x 104 | 40.06 |
| ISA 50x50 | 3 | 295 | 13.2 | 6.9 x 104 | 22.88 |
| | 4 | 388 | 13.7 | 9.1 x 104 | 30.11 |
| | 5 | 479 | 14.1 | 11.0 x 104 | 37.24 |
| | 6 | 568 | 14.5 | 12.9 x 104 | 44.24 |
| ISA 55x55 | 5 | 527 | 15.3 | 14.7 x 104 | 40.38 |
| | 6 | 628 | 15.7 | 17.3 x 104 | 48.21 |
| | 8 | 818 | 16.5 | 22.0 x 104 | 63.03 |
| | 10 | 1001 | 17.2 | 26.3 x 104 | 77.61 |
| ISA 60x60 | 5 | 575 | 16.5 | 19.2 x 104 | 43.44 |
| | 6 | 684 | 16.9 | 22.6 x 104 | 51.77 |
| | 8 | 896 | 17.7 | 29.0 x 104 | 68.08 |
| | 10 | 1100 | 18.5 | 34.8 x 104 | 83.90 |

Above table shows the values of moment of resistance of different angles. So according to the additional moment at different members in case of STR-EQ these angles can be used.

***11.2 DESIGN OF BRACKET:***

For the maximum moment and maximum shear force among all the beams design a bracket with Welded connection.

Maximum additional moment in structure is

$M_{max}$ = 42.26 kNm

Maximum additional shear force in the structure is

$F_{max}$ = 33.43 kN

Minimum size of weld = 5 mm

Use 8 mm weld on each side of bracket plate.

Throat thickness, t = 0.7 x 8 = 5.6 mm

Resistance of weld = $f_{wd} = \frac{f_u}{\sqrt{3}} x \frac{1}{r_{mw}}$

$f_u$ = maximum force carried by weld or the plate

$r_{mw}$ = 1.25 for shop weld

$r_{mw}$ = 1.50 for field weld

$f_{wd} = \frac{250}{\sqrt{3}} x \frac{1}{1.25} = 115.47\ N/mm^2$

Depth of weld required to resist bending alone = $h' = \sqrt{\frac{6M}{2tf_{wd}}}$

$h' = \sqrt{\frac{6x\ 42.26x10^6}{2x5.6x115.47}} = 442.79$ mm

About 10% extra depth is to be provided.

Let h = 1.1 x 442.79 mm

So h = 487.07 mm≈ 490 mm.

6.5.1 Check for stress:

Direct shear stress

$q = \frac{F}{2\ x\ t\ x\ h}$

$q = \frac{33.43\ x\ 1000}{2\ x\ 5.6\ x\ 490}$

$q = 6.09\ N/mm^2$

Bending stress $f = \frac{M}{Z} = \frac{Pe}{Z}$

$f = \frac{6\ Pe}{2\ x\ t\ x\ h^2}$

$f = \frac{6\ x\ 42.26\ x\ 10^6}{2\ x\ 5.6\ x\ 490^2} = 94.29\ N/mm^2$

$\therefore \sqrt{f^2 + 3q^2} = 94.88\ N/mm^2 < 115.47\ N/mm^2$

Hence design is safe.

## 12. CONCLUSIONS

The present study investigates the structural behaviour of an RC frame (G+2 Commercial building) under the additional load in the form of seismic forces. The structure is analyzed for two load cases. In first case (Gravity load case) structure is analyzed for only gravity forces and no seismic force is considered in this analysis while in second case (Seismic load case) structure is analyzed with consideration of seismic forces along with gravity forces. The seismic forces cause substantial change in columns and beams forces in the structure.

## REFERENCES

[1] Singh V., bansal P. P., Kumar M. and Kaushik S.S (2014), “Experimental studies on strength and ductility of CFRP jacketed reinforced concrete-beam joint” in construction and building materials volume 55, page no. 194 – 201.

[2] Belal M.F., Mohamed H.M. and Morad S.A. (2014), “Behaviour of reinforced concrete columns strengthened by steel jacket” in HBRC journal.

[3] Ruano G., Facundo I, Pedraza R. I., Sfer D. and Luccioni B. (2013), “Shear retrofitting of reinforced concrete beams with steel fibre reinforced concrete” in construction and building materials volume 54, page no. 646 – 658.

[4] Su R. and lingzhi L.I. (2013), “Strengthening of Reinforced Concrete Structures by Bolting of Steel Plates” in Hong Kong Concrete Institute.

[5] Sabu D.J., Pajgade P.S.(2012),“Seismic Evaluation of Existing Reinforced Concrete Building” in International Journal of Scientific & Engineering Research Volume 3, ISSN 2229-5518.

[6] Obaidat Y. T. (2011), "structural retrofitting of concrete beams using frp - Debonding Issues" in department of construction science structural mechanics, ISSN 0281-6679.

[7] Obaidat Y.T., Heyden S., Dahlblom O., Abu-Farsakh G. and Yahia A.J. (2011), "Retrofitting of Reinforced Concrete Beams Using Composite Laminates", in Construction & Building Materials, volume 25, page no. 591-597.

[8] Vijayakumar A. and Venkatesh B. (2011), "A survey of methods and techniques used for Seismic retrofitting of RC buildings" in international journal of civil and structural engineering, ISSN 0976 – 4399, volume 2, page no. 56-66.

[9] Obaidat Y. T., Dahlblom O., Heyden S. (2010), "Nonlinear FE Modelling of Shear Behaviour in RC Beam Retrofitted with CFRP", in proceedings of Computational Modelling of Concrete Structures (EURO-C), ISBN 978-0-415-5879-1.

[10] S. Kothandaraman and G. Vasudevan (2010), "Flexural retrofitting of RC beams using external bars at soffit level- An experimental study" in construction and building materials volume 24, page no. 2208–2216.

[11] Duggal S.K. (2010) "Limit state Design of steel structures" in Tata McGraw hill education private limited New Delhi, ISBN no. 978-0-07-070023-9.

[12] Bhavikatti S.S. (2010) "Design of steel structures" in I.K. International Publishing House Pvt. Limited, ISBN no 938057813X, 9789380578132.

[13] Shrestha H.D. and Krishna S., Pribadi (2009), "Retrofitting of existing vulnerable school buildings – assessment to retrofitting" in construction Quality & Technical Assistance (CQTA) Center for Disaster Mitigation - Institute of Technology Bandung (CDM –ITB).

[14] Williams R.J., Gardoni P. and Bracci J. M. (2009), "Decision analysis for seismic retrofit of structures" in Zachry Department of Civil Engineering, Texas A&M University, College Station, USA volume 31, page no. 188-196.

[15] Shrestha H. And Dixit A.M. (2008) "Identifying seismic retrofitting measures for common reinforced concrete frame buildings in Nepal" in The 14th World Conference on Earthquake Engineering, Beijing, China.

[16] Riyadh A.A. and Riyadh A.M. (2006), "Coupled flexural – shear interaction of RC beams using CFRP straps" in 13th International Conference of Composite Structures, Melbourne, Australia volume 75, page no. 457 – 464.

[17] Lakshmanan N. (2006), "Seismic evaluation and retrofitting of buildings and structures" in ISET Journal of Earthquake Technology, Volume 43, Paper No. 469.

[18] Agarwal P. and Shrikhande M. (2006) "Earthquake resistant design of structure" in prentice hall, of India private limited, New Delhi, ISBN no. 81-203-2892-2.

[19] Arlekar J.N. and Murty C. V. R. (2004), "Shear moment interaction for design of steel beam-to-column connections" in 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada, Page no. 635.

[20] Richard D., Sheikh S.A. and Bayrak O. (2003) "Retrofit of Square Concrete Columns with Carbon Fibre-Reinforced Polymer for Seismic Resistance" in ACI structural journal, title no. 100-S81.

[21] Lee H.S., Kage T., Noguchi T. and Tomosawa F. (2002), "An experimental study on the retrofitting effects of reinforced concrete columns damaged by rebar corrosion strengthened with carbon fibre sheets", in Cement and Concrete Research volume 33, page no. 563–570.

[22] Sakino K. and Sun Y. (2000), "Steel jacketing for improvement of column strength and ductility" in 12th world Conferences on Earthquake Engineering (WCEE).

[23] Pajgade P.S., Jawade S.S., Gaulkar M.P. and Kulkarni S.S. (2000), "Evaluation and retrofitting of buildings damaged due to Jabalpur (India) earthquake of may 22nd 1997" in 12th world Conferences on Earthquake Engineering (WCEE).

[24] Parretti R. and Nanni A. (2000), "Axial testing of concrete columns confined with carbon FRP" effect of fibre orientation.

[25] Geng Z.J., Chajes M.J., Chou T.W. and Pan D.Y.C. (1998), "The retrofitting of reinforced concrete" in composite sciences and technology, volume 58, page no. 1298-1305.

[26] Gomes A. and Appleton J. (1997), "Strengthening design of Concrete beam by addition of steel plates" Department of civil engineering, IST, Technical university of Lisbon, Portugal.

[27] R.P. Johnson and R.J. Buckby (1994), "Composite structures of steel and concrete.