SEISMIC DESIGN OF MULTISTORIED AND MULTI BAY STEEL BUILDING FRAME

A thesis submitted in the partial fulfillment of the requirements for the

Degree of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

BY

BELLAM PRATHEEK (109CE0454)

RISHIT KAR (109CE0444)

UNDER THE GUIDANCE OF

Prof. ASHA PATEL

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NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA

2013

CERTIFICATE

This is to certify that the Project Report entitled, "**SEISMIC DESIGN AND ANALYSIS OF MULTISTORIED AND MULTIBAY STEEL FRAME**" submitted by **BELLAM PRATHEEK (Roll-109CE0454)** and **RISHIT KAR (Roll-109CE0444)** in partial fulfillment for the requirements for the award of the Degree of Bachelor of Technology in Civil Engineering at National Institute of Technology, Rourkela is an authentic work carried out by them under my supervision and guidance. To the best of my knowledge, the matters embodied in the thesis have not been submitted to any other university/Institute for the award of any Degree or Diploma.

Prof. Asha Patel

Department Of Civil Engineering

Place: NIT Rourkela

National Institute of Technology

Date: 11th May 2013

Rourkela-769008, Orissa (India)

Acknowledgement

At this moment we feel grateful and loyal to our supervisor, Prof. Asha Patel for her enthusiastic suggestions and kind help towards us in achieving success of our project. Her constant encouragement and invaluable share of her valuable time with us made such a difficult task so easy. We take this opportunity to express our heartfelt gratitude to all those who helped us in various ways during our project at NIT Rourkela. We also express our sincere gratitude to Prof. N. Roy, Head of the Department, Civil Engineering, and Prof. Ramakar Jha for providing valuable departmental facilities. We would like to use this opportunity to express our gratitude to all the faculty members of the Civil Engineering Department who played a vital role in bringing us to this level.

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Abstract:

Steel is one of the most widely used material for building construction in the world .The inherent strength, toughness and high ductility of steel are characteristics that are ideal for seismic design. To utilize these advantages for seismic applications, the design engineer has to be familiar with the relevant steel design provisions and their intent given in codes.

The seismic design of building frame presented in this project is based on IS 1893-2002 and IS 800 .The aim of the present work is to analyze a multistory and multi bay (G+5) moment resisting building frame for earthquake forces following IS 1893 and then design it as per IS 800:2007 .The frame consists of six story's and has three bays in horizontal direction and five bays in lateral direction. The selection of arbitrary sections have been done following a standard procedure. The two methods that have been used for analysis are Equivalent static load method and Response Spectrum method .A comparative study of the results obtained from both these methods have been made in terms of story displacement ,inter story drift and base shear .

The frame has also been further checked for P-Δ analysis and required correction in moments have been done following IBC code .Then the steel moment resisting frame has been designed following IS-800:2007 based on these methods of analysis. In the process of design the section has undergone numerous iterations till all the criteria mentioned in the IS 800 have been satisfied. The designed frame was again analyzed and results were compared in terms of sections used. The cost efficiency of both the methods have been compared .Finally the design of connection of an interior joint and an exterior joint of the frame have been done and the calculations have been shown. Also the design of the foundation which consists of the base plate has been done according to IS 800:2007.Relevant calculations have been shown and the figures have been drawn.

The software used for analysis and design is STAAD PRO .Both during design and analysis sufficient manual calculations have been made and compared.

LIST OF FIGURES

LIST OF TABLES

NOMENCLATURE:

A	Cross-sectional area of the structural element
A_h	Design horizontal seismic coefficient for a structure
A_g	gross area of cross-section
α	Imperfection Factor
δ	Deflection of the section.
DL	Dead load
EQ	Earthquake Load
F_y	Yield strength of Steel
F_{yb}	Yield strength of beams
F_{yc}	Yield strength of columns
E	Young's modules
G_{floor}	Dead weight of the floor
G_{walls}	Dead weight of the walls
I_{zz}	Moment of inertia about z-z axis.
KL/R	Effective Slenderness Ratio
LL	Live Load
M_c	Moment of columns
M_g	Moment of beams
R_{zz}	Radius of gyration about z-z axis.
R	Bolt value
λ	Non dimensional effective Slenderness ratio
ϴ	Inter storey drift coefficient
$ϴ_x$	Rotation in x- axis.
Δ_x	Drift of storey x
Z	Seismic zone factor

CHAPTER-1

INTRODUCTION

1.1 INTRODUCTION

Seismic Analysis is a subset of structural analysis and is the calculation of the response of a building structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent.

The most important earthquakes are located close to the borders of the main tectonic plates which cover the surface of the globe. These plates tend to move relative to one another but are prevented by doing so by friction until the stresses between plates under the epicenter point become so high that a move suddenly takes place. This is an earthquake. The local shock generates waves in the ground which propagate over the earth's surface, creating movement at the bases of structures. The importance of waves reduces with the distance from the epicenter. Therefore, there exists region of the world with more or less high seismic risk, depending on their proximity to the boundaries of the main tectonic plates

Besides the major earthquakes which take place at tectonic plate boundaries, others have their origin at the interior of the plates at fault lines. Called 'intra plates' earthquakes, these less energy, but can still be destructive in the vicinity of the epicenter

The action applied to a structure by an earthquake is a ground movement with horizontal and vertical components. The horizontal movement is the most specific feature of earthquake action because of its strength and because structures are generally better designed to resist gravity than horizontal forces. The vertical component of the earthquake is usually about 50% of the horizontal component, except in the vicinity of the epicenter where it can be of the same order.

Steel structures are good at resisting earthquakes because of the property of ductility. Experience shows that steel structures subjected to earthquakes behave well. Global failures and huge numbers of casualties are mostly associated with structures made from other materials. This may be explained by some of the specific features of steel structures. There are two means by which the earthquake may be resisted:

Option 1 structures made of sufficiently large sections that they are subject to only elastic stresses

Option 2 structures made of smaller sections, designed to form numerous plastic zones.

A structure designed to the first option will be heavier and may not provide a safety margin to cover earthquake actions that are higher than expected, as element failure is not ductile. In this case the structure's global behavior is 'brittle' and corresponds for instance to concept a) in a Base Shear V- Top Displacement diagram. In a structure designed to the second option selected parts of the structure are intentionally designed to undergo cyclic plastic deformations without failure, and the structure as a whole is designed such that only those selected zones will be plastically deformed.

The structure's global behavior is 'ductile' and corresponds to concept b) in the Base Shear V- Top Displacement d. The structure can dissipate a significant amount of energy in these plastic zones, this energy being represented by the area under the V-d curve. For this reason, the two design options are said to lead to 'dissipative' and 'non-dissipative' structures.

A ductile behavior, which provides extended deformation capacity, is generally the better way to resist earthquakes. One reason for this is that because of the many uncertainties which characterize our knowledge of real seismic actions and of the analyses we make, it may be that the earthquake action and/ or its effects are greater than expected. By ensuring ductile behavior, any such excesses are easily absorbed simply by greater energy dissipation due to plastic deformations of structural components. The same components could not provide more strength (a greater elastic resistance) when option 1 is adopted. Furthermore, a reduction in base shear V(V reduced < V elastic) means an equal reduction in forces applied to the foundations, resulting in lower costs for the infrastructure of a building.

Steel structures are particularly good at providing an energy dissipation capability, due to:

.. The ductility of steel as a material

.. The many possible ductile mechanisms in steel elements and their connections

.. The effective duplication of plastic mechanisms at a local level

.. Reliable geometrical properties

.. Relatively low sensitivity of the bending resistance of structural elements to the presence of coincident axial force

Variety of possible energy dissipation mechanisms in steel structures, and the reliability of each of these possibilities, are the fundamental characteristics explaining the excellent seismic behavior of

steel structures. Furthermore, steel structures tend to have more reliable seismic behavior than those using other materials, due to some of the other factors that characterize them:

- guaranteed material strength, as result a of controlled production
- designs and constructions made by professional

CHAPTER-2

PROBLEM STATEMENT

The structure consisting of six stories with three bays in horizontal direction and six bays in lateral direction is taken and analyzed it by both equivalent static method and response spectrum analysis and designed.

The storey height is 3 meters and the horizontal spacing between bays is 8 meters and lateral spacing of bays is 6 meters

The seismic parameters of building site are as follows

- Seismic zone: 3
- Zone factor ‘Z’: 0.16
- Building frame system: steel moment resisting frame designed as per SP 6
- Response reduction factor: 5
- Importance factor:1.5
- Damping ratio: 3%

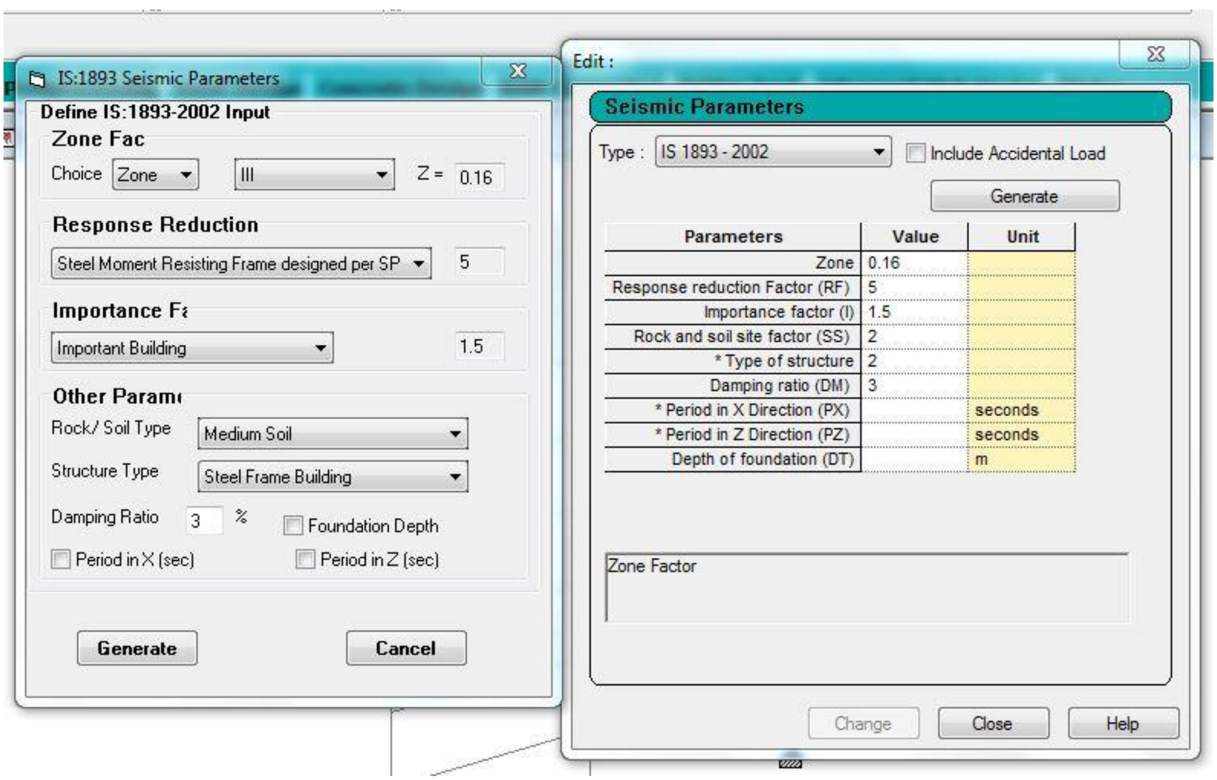


FIG 2.1 : STAAD input of seismic parameters

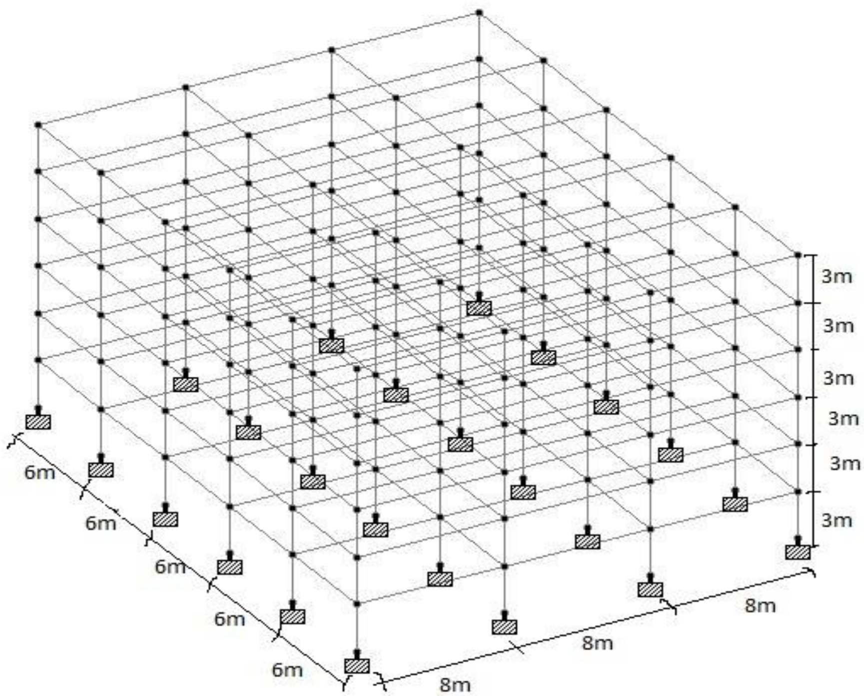


FIG 2.2 : 3-dimensional view of the steel building frame

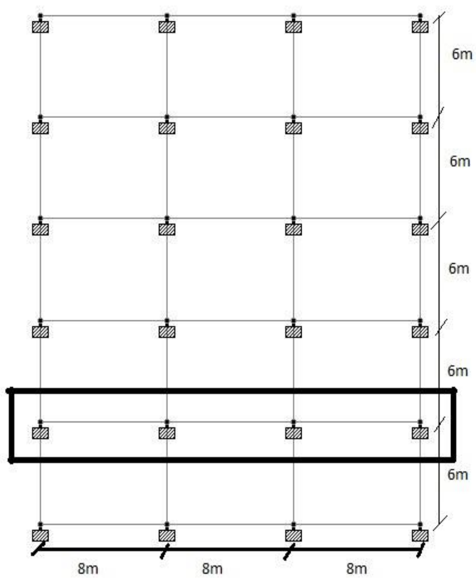


FIG 2.3: Plan of the building frame

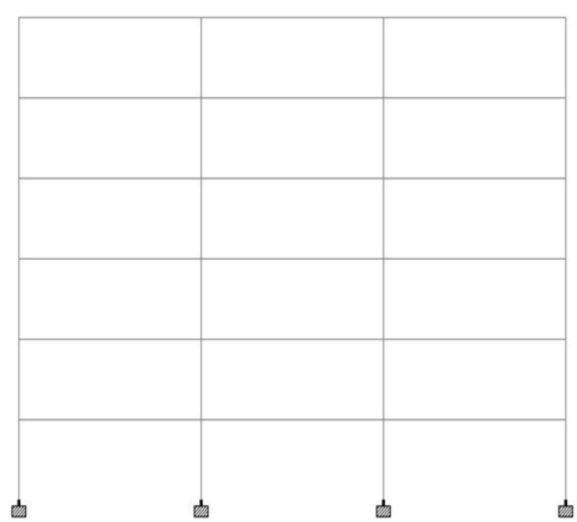


FIG 2.4 : Elevation of the building frame

LOAD PARAMETERS:

dead load is taken as = $5KN/m^2$ and live load is taken as 3 KN/m^2

Load Calculation

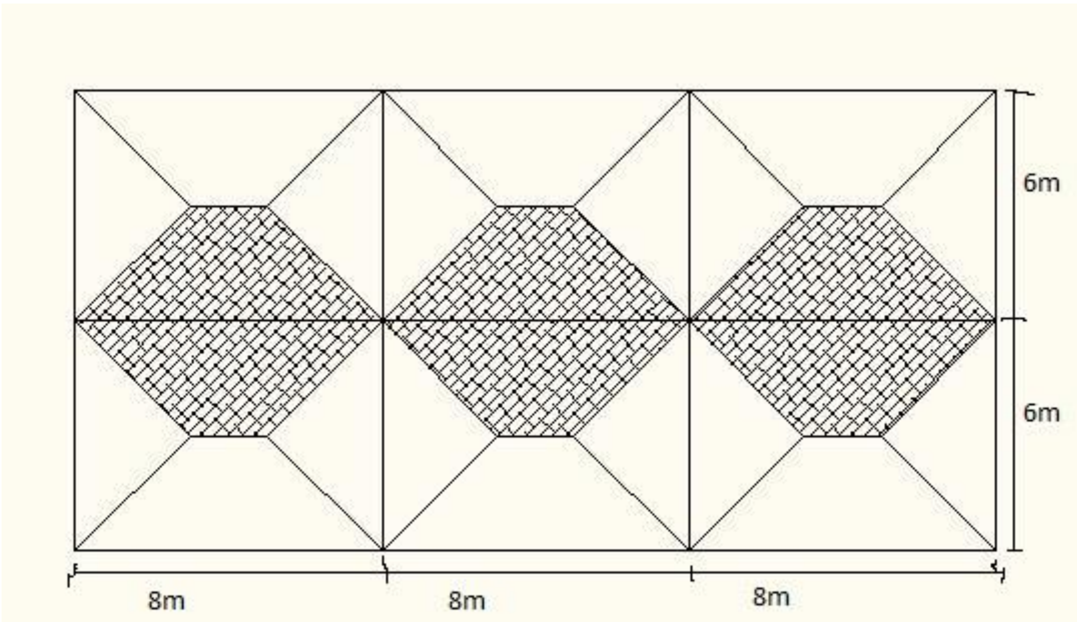


FIG 2.5 : Load distribution diagram

Load on beam along horizontal direction

1.	Dead Load	=	$30m^2 \times 5KN/m^2$	=	150KN
	Uniformly Distributed Load	=	150/8	=	18.75KN/m
2.	Live Load	=	30×3	=	90KN
	Uniformly Distributed Load	=	90/8	=	11.75KN/m

Load combinations as per IS1893-2002 :

- 1.7(DL+LL)
- 1.7(DL+EQ)
- 1.7(DL-EQ)
- 1.3(DL+LL+EQ)
- 1.3(DL+LL-EQ)

CHAPTER-3

METHODOLOGY

METHODOLOGY:

The initial step is preliminary design of building frame. The procedure involved are selection of sections of members of the frame. Since the dynamic action effects are a function of member stiffness, the process unavoidably involves much iteration.

The example considered here involves a building in which seismic resistance is provide by moment resisting frames (MRF), in both x and y directions. Moment resisting frames (MRF) are known to be flexible structures. Thus their design is often governed by the need to satisfy deformation criteria under service earthquake loading, or limitation of P-Δ effects under design earthquake loading. For this reason rigid connections are preferred. The Preliminary design consists of following steps:

- Defining beam sections, checking deflection and resistance criteria under gravity loading.
- Following an iterative process, going through the following steps until all design criteria are fulfilled.

  The iterative process can make use either of lateral force method or the spectral response modal superposition method.

1. Selection of Beam Sections.
2. Definition of Column Sections checking the ‘weak beam strong column criteria’.
3. Check compression /buckling at ground floor level under gravity loading.
4. Calculation of seismic mass.
5. Static analysis of one plane frame under lateral loads.
6. Static analysis under gravity loading.
7. Stability check using P-Δ effects (parameter Θ) in the seismic loading situation.
8. Deflection check under earthquake loading.
9. For Response spectrum analysis step 5 is replaced by response spectrum analysis of one plane frame to evaluate earthquake action effects.

CALCULATION:

Selection of Beam Sections.

The beams to be used for the section are determined initially by using two checks: Moment Resistance check and Deflection criteria.

Checking deflection limits of Beams in x- direction.

Selection of Beam Section

Total Dead load + Live load =51 KN/m = gravity load

Deflection of a section loaded with uniformly distributed load.

$$\delta = \frac{Pl^4}{384EI}$$

Now the code specifies maximum deflection limit as $\frac{l}{300}$ Where l is the effective length of the section.

So, $\frac{l}{300} = \frac{Pl^4}{384EI}$

$$I_{Required} = \frac{300\,Pl^3}{384\,E}$$

$$= \frac{300 \times 51 \times 8^3}{384 \times 2.1 \times 10^8}$$

$$= 9714.3 cm^4$$

So section selected is **ISMB 350**

I_{zz}=13630cm^4

Area 66.7cm^2

Depth of section= 350 mm

Breadth of flange =140 mm

Thickness of flange=14.2 mm

Thickness of Web=8.1 mm

Definition of Column Sections checking the 'weak beam strong column criteria'

M_c : moment of column

M_g : moment of beam

$\Sigma M_c = M_{c1} + M_{c2}$

$\Sigma M_g = M_{g1} + M_{g2}$

$\Sigma M_c \geq 1.2 \Sigma M_g$(as per IS 800:2007)

$\sum f_{yc} \times Z_{column} > 1.2 \sum f_{yb} \times Z_{beam}$

So,

$2\times 250 \times Z_{req.} = 1.2 \times 250 \times 1094.8 \times 1000$

$Z_{req.} = 656.88\ cm^3$

So, therefore the section selected is **<u>I80012B50012.</u>**

There by calculating the moment using the above equations the section of column is decided to be: I80012B50012

Check compression /buckling at ground floor level under gravity loading.

Relevant loaded area=$8\times6=48m^2$.

Floor weight is $5Kn/m^2$,all included.

$G_{floor}=48\times5=240KN/storey$

$G_{walss}=(8+6)\times3=42$ KN/storey

$G_{frame}=18.5$ KN/storey

$Q=3\ KN/m^2 \times 48=144$ KN

$1.35 \times G + 1.5\times Q = 1.35\times300.5 + 1.5\times 144=622$ KN/storey

Compression in column at basement level: $6 \times 622=3732$ KN.

Approximate buckling length=3.0 m

(equal to storey height)

Now for the column section of 180012B50012

Sectional area=387 cm^2

And I_{ZZ}=494454 cm^4

$R_{zz}=\sqrt{\frac{I_{ZZ}}{A}}$ =35.744 cm

$\lambda=\sqrt{\frac{f_{y\times\frac{KL}{r}}}{\pi^2\times E}}$ =.48

$\chi=\frac{1}{\phi+\sqrt{\phi^2-\lambda^2}}$ =.85

$F_{cd}=\chi\, f_{y/}\gamma_{mo}$=.85×250/1.1=193.18 N/$mm^2$

$P_d=F_{cd}\times A$=193.18×38700=7476.136KN.>3732 KN

Where F_{cd} is design compressive stress.

Where P_d is design compressive strength.

Calculation of seismic mass

For the steel frame considered, the seismic mass is calculated in terms of joint weight and the member weight of the members of the steel frame

Dead load = 5KN/m^2 , live load= 3KN/m^2

Area load contributed to each beam is 30m^2, and there are 3 beams per each storey. Therefore total DL +LL per each storey is calculated to be = $3\times 30\times(5+3)$ =720KN

Nodal loads of 144KN is put on both interior nodes and a nodal load of 72KN is applied on the exterior nodes. Therefore total nodal load contribution for the seismic mass is: =$144\times 2+72\times 2$ = 432KN

Weight of wall is also contributes to the seismic mass. Weight of the wall is 3KN/m. therefore total wall weight per storey= 3×24 =72KN

Therefore total seismic mass per storey is given by= 720+432+72 = 1224KN

CHAPTER-4

ANALYSIS PROCEDURE

4.1 LATERAL FORCE METHOD:

The seismic load of each floor is calculated at its full dead load and imposed load. The weight of columns and walls in any storey should be appropriately divided to the floors above and below the storey. Buildings designed for the storage purposes are likely to have large percentages of service load present at the time of the earthquake. The imposed load on the roof is not considered.

In the equivalent static method which accounts for the dynamics of the buildings in approximate manner, the design seismic base shear is determined by $V_B = A_h \times W$

The following assumptions are involved in the equivalent static method procedure

- Fundamental mode of building makes the most significant contribution to the base shear
- The total building mass is considered against the modal mass that would be used in dynamic procedure. And both of these assumptions are valid for low and medium rise buildings which are regular

After the base shear is determined, it should be distributed along the height of the building using the following expression

$Qi = V_b\left(\frac{W_i h_i^k}{\Sigma W_j h_j^k}\right)$

Where V_b is total design lateral force.

W_i is the seismic weight of floor i

H_i is the height of floor measured from base

The approximate fundamental natural period of vibration in seconds, for a moment resisting frame without brick infill panels is given by:

$T_a = 0.085h^{0.75}$

for all other buildings including moment resisting frame buildings with brick infill, $T_a = \frac{0.09h}{\sqrt{d}}$

where ‘d’ is base dimension of the building at the plinth level for buildings with concrete and masonry shear walls,

Ta= $\frac{0.075h^{0.75}}{\sqrt{A_w}}$

Aw is the total effective area of the walls in the first storey of the building in square meters

In our case the value of T_a=0.09×18/√24=0.33 Hz.

After obtaining the seismic forces acting at different levels, the forces and moments in different members can be obtained by using any standard computer program for various load combinations specified in the code. The structure must also be designed to resist the overturning effects caused by seismic forces. And also storey drifts, member forces and moment due to P-delta effect must be determined. IS 1893 stipulates that the storey drift in any storey due to the minimum specified lateral loads , with a partial load factor of 1.0 should not exceed 0.004 times the storey height.

Table 4.1 : Analysis by lateral force method

Storey no.	Absolute displacement of storey D_i (m)	Design inter storey drift D_r (m)	Storey lateral force V_{tot} (KN)	Shear at storey P_{tot} (KN)
1	0.003869	0.003869	1.969	179.201
2	0.012595	0.008726	7.951	177.232
3	0.023837	0.011242	17.83	169.281
4	0.035892	0.012055	31.657	151.451
5	0.047566	0.011674	49.212	119.794
6	0.058123	0.010557	70.582	70.582

4.2 RESPONSE SPECTRUM ANALYSIS:

In the field of seismic analysis this is one of the most popular methods. The design spectrum diagram is used to perform it.The response spectrum method uses the idealization of a multi storey shear building by a basic assumption. The assumption used is that the mass is lumped at the roof diaphragm levels and at the floor levels. The diaphragrams are assumed as infinitely rigid and the column axially inextensible but laterally flexible. The dynamic response of the spectrum is represented in the form of lateral displacements of the lumped mass with the degrees of dynamic freedom(or modes of vibration n) being equal to the number of masses. The undamped analysis of the building can be done following standard methods of mechanics using appropriate masses and elastic stiffness of the structural system, and the natural period (T) and mode shapes (ϕ) of the modes in vibration can be obtained. The distribution of mass and the stiffness of the building determine the mode shapes.

As the ground motion is applied at the base of the multi mass system, the deflected shape is but a combination of all mode shapes, which otherwise can be obtained by superposition of the vibrations of each individual lumped mass. A modal analysis procedure is utilized in determining the dynamic response of multi-degree-of-freedom system. Modal analysis as suggested by IS 1893 is discussed herewith.

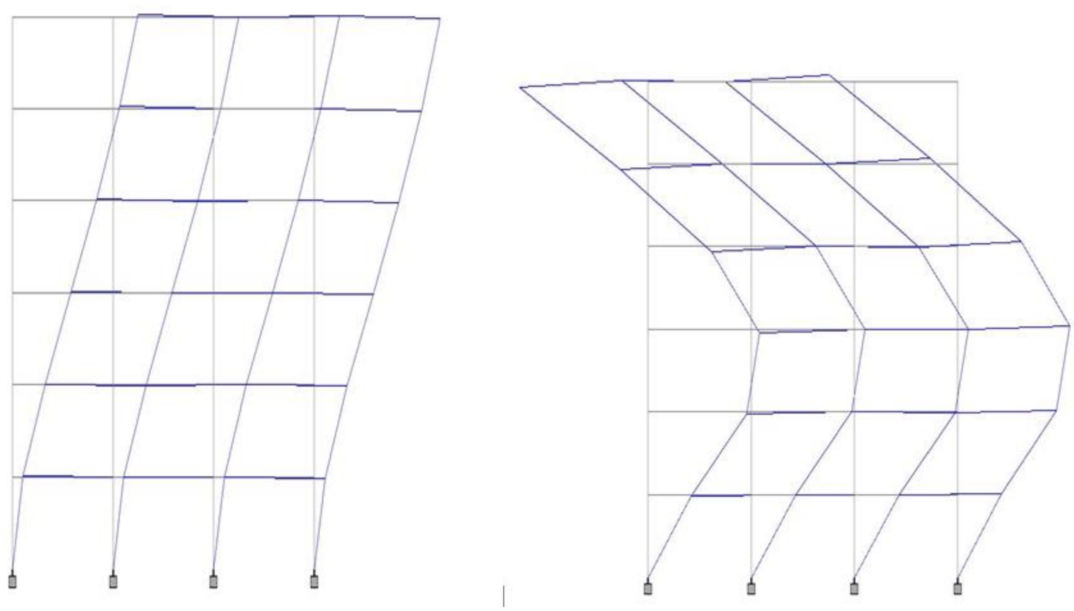
Each individual mode of vibration has its unique period of vibration (with its own shape called mode shape formed by locus of points of the deflected masses.)

Response is obtained by using different modal combination methods such as square-root-of-sum-of-squares method(SRSS)or the complete quadratic method (CQC) which are used when natural periods of the different modes are well separated (when they differ by 10% of the lower frequency and the damping ratio does not exceed 5%.The CQC is a method which can account for modal coupling methods suggested by IS 1893.

Table 4.2 : Analysis by response spectrum method.

Storey no.	Absolute displacement of storey D_i (m)	Design inter storey drift D_r (m)	Storey lateral force V_{tot} (KN)	Shear at storey P_{tot} (KN)
1	0.00491	0.00491	1.877	120.981
2	0.0115	0.0066	6.112	119.104
3	0.0161	0.0046	10.651	112.992
4	0.0196	0.0035	17.331	102.341
5	0.0219	0.0023	29.98	85.01
6	0.0234	0.0015	55.03	55.03

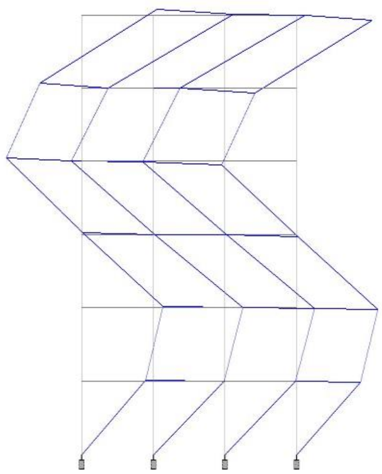
MODE SHAPES: fig (4.1)

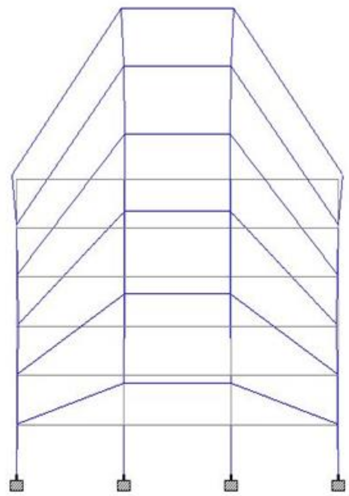


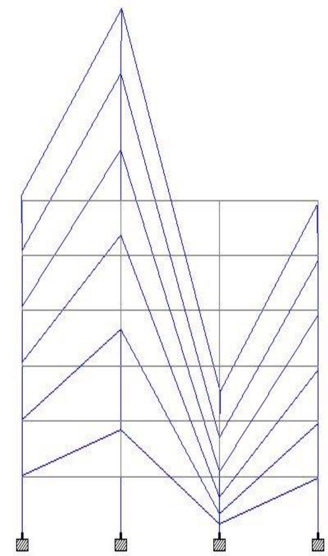
Mode 1 (1.592hz), **mode 2 (5.224hz),**

Modal Participation factor

MPF= 85.33 **MPF=8.13**







Mode 3 (9.525hz) **mode 4 (12.796hz)** **mode 5 (13.294hz)**

MPF 0 **MPF 0.01** **MPF 2.04**

Table 4.3 :Base shear and mass participation factor

MODE	BASE SHEAR(KN)	Mass participation factor
1	252.75	85.33
2	27.8	8.13
3	12.1	3.54
4	0	0
5	0.02	0.01
6	5.85	2.04

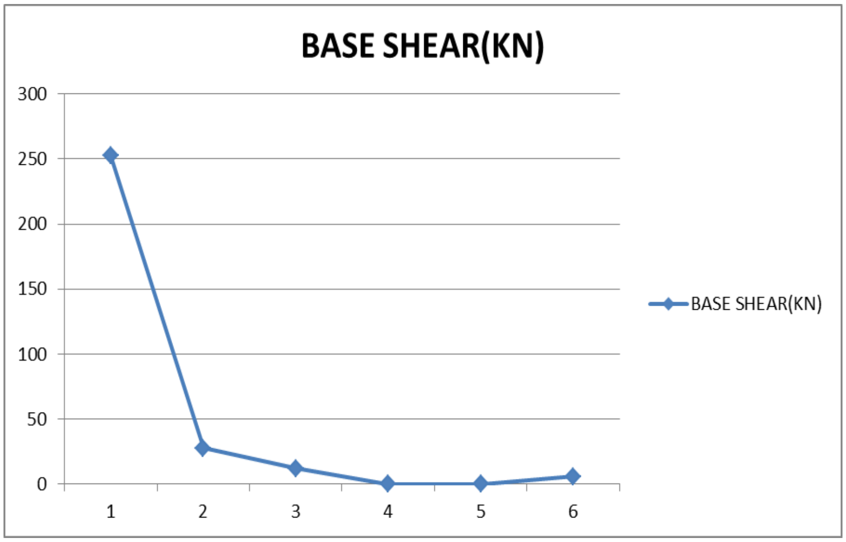
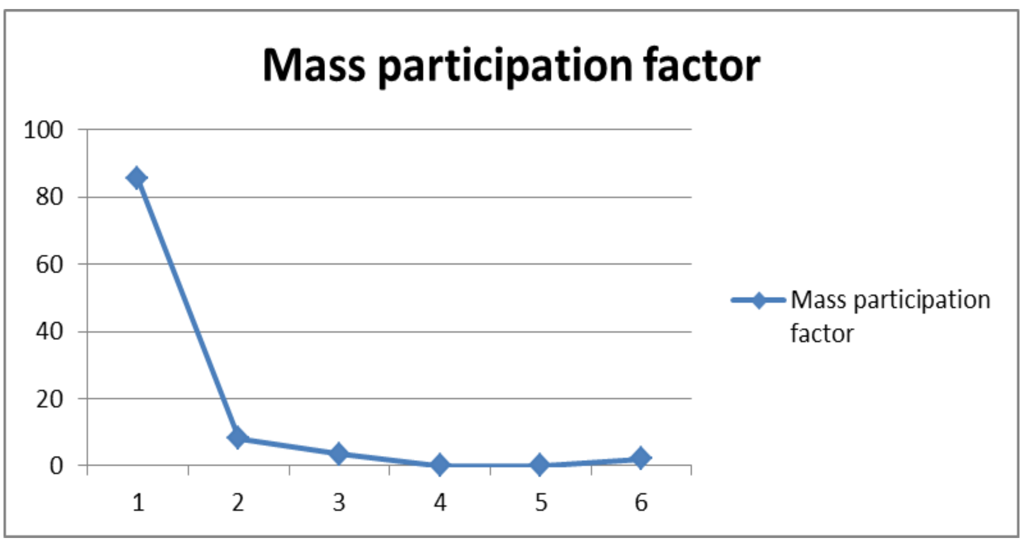


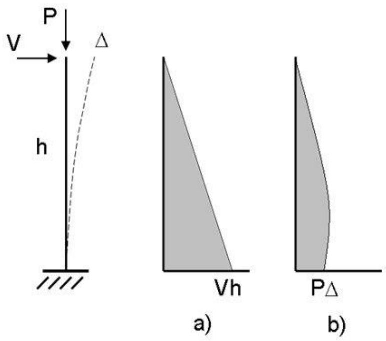
Fig (4.2) graph of modes Vs base shear



Fig(4.3) Graph of mass participation factor

4.3 P-Δ ANALYSIS:

The P-Δ effect refers to the additional moment produced by the vertical loads and the lateral deflection of the column or other elements of the building resting lateral forces.



(Fig 4.4) figure showing P-Δ effect

- Due to this load, the column undergoes a relative displacement or drift Δ. In this case P-Δ effect results in a secondary moment M_s=PΔ. Which is resisted by an additional shear force $\frac{P\Delta}{L}$ in the column. This secondary moment M_s=PΔ will further increase the drift in the column and consequently will produce an increment of the secondary moment and shear force in the column.
- To calculate the final drift Δ_{tx} add additional drift resulting from the incremental overturning moment i.e. $M_x\theta_x$, $(M_x \theta_x)\theta_x$, $((M_x\theta_x)\theta_x)\theta_x$ . . . . . to the primary drift Δx i.e.
- $\Delta_{tx}=\Delta_x+ \Delta_x\theta_x + \Delta_x\theta_x^2 + \ldots$
- Which is equal to = $\Delta_x(\frac{1}{1-\theta x})$ where $\theta x= \frac{Mx'}{Mx} = \frac{Px\Delta x}{VxHx}$
- M_x'= secondary moment
- P_x=total weight (DL+LL) at level X & above
- Δ_x= drift of storey X
- V_x= shear force of storey X
- H_x=height of storey X
- The code (uniform building code UBC) stipulates that the P-Δ need not be evaluated when the ratio of the secondary moment M_x' to the primary moment M_x at each level of the building is less than 0.1

Table 4.5:Correction for P-Δ effect (lateral force method)

Storey no:	Absolute displacement of the storey D_i (m)	Design inter storey drift D_r(m)	Storey lateral forces	Shear at storey V_{tot}(KN)	Total cumulative gravity load at storey P_{tot} (KN)	Storey height: H_i(m)	Inter storey drift sensitivity coefficient: (θ)
1	0.003869	0.003869	1.969	179.201	7344	3	0.05285
2	0.012595	0.008726	7.951	177.232	6120	3	**0.10043***
3	0.023837	0.011242	17.83	169.281	4896	3	**0.10838***
4	0.035892	0.012055	31.657	151.451	3672	3	0.09742
5	0.047566	0.011674	49.212	119.794	2448	3	0.07951
6	0.058123	0.010557	70.582	70.582	1224	3	0.06102

Table 4.6: Correction for P-Δ effect, (response spectrum analysis)

Storey no:	Absolute displacement of the storey D_i (m)	Design inter storey drift D_r(m)	Storey lateral forces	Shear at storey V_{tot}(KN)	Total cumulative gravity load at storey P_{tot} (KN)	Storey height: H_i(m)	Inter storey drift sensitivity coefficient: (θ)
1	0.00491	0.00491	1.877	120.981	7344	3	0.09935
2	0.0115	0.0066	6.112	119.104	6120	3	**0.11304***
3	0.0161	0.0046	10.651	112.992	4896	3	0.06644
4	0.0196	0.0035	17.331	102.341	3672	3	0.04186
5	0.0219	0.0023	29.98	85.01	2448	3	0.02207
6	0.0234	0.0015	55.03	55.03	1224	3	0.01112

*Beams in this storey failed to satisfy P-Δ effect

From the above table checks are made on the limitation of P-Δ effects with the results from the **lateral force method**. The value of resultant base shear is: 179.201KN

ϴ <0.1 at storeys 1,4,5,6. bending moment and other action effects found from the analysis at storeys 2 and 3 have to be increased by 1/(1-ϴ (1.11at storey 2 and 1.12 at storey 3)

The maximum bending moment is at storey 2 : 230.172KNm

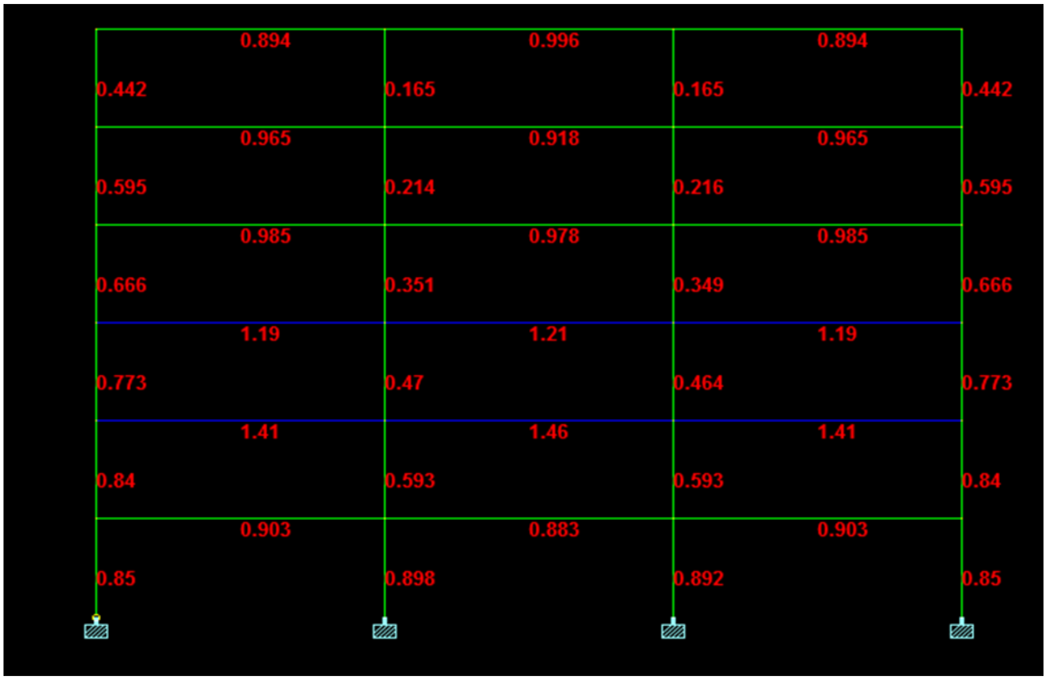
With the 1/(1-ϴ) increase: 1.11164×230.172=255.868KNm

Beams are ISMB350: $M=Z_x \times f_y$=877×250=219.25KNm And 219.25<230.172

(**So beams are failing**)

CHAPTER-5

DESIGN



Fig(5.1) Diagram showing failed members

BEAM AND COLUMN DESIGN

Staad pro is used for designing all members of frame following IS 800-2007

IS 800:2007 CLAUSE 7.1.2

DESIGN STRENGTH

7.1.1 Common hot rolled and built-up steel members (section: I80012B50012, member 17) used for carrying axial compression, usually fail by flexural buckling. The buckling strength of these members is affected by residual stresses, initial bow and accidental eccentricities of load. To account for all these factors, the strength of members subjected to axial compression is defined by buckling class a, b, c, or d as given Table 7.

7.1.2 The design compressive strength Pd, of a member is given by:

$P<P_d$ where $P_d = A_e \, x f_{Cd}$

where

A_e = effective sectional area as defined in 7.3.2, and

f_{cd} = design compressive stress, obtained as per 7.1.2.1.

7.1.2.1 The design compressive stress, f_{cd}, of axially loaded compression members shall be calculated using the following equation:

$f_{cd} = (f_y / y_{mo})/(\varphi + [\varphi^2 - \lambda^2]^{0.5}) = \chi f_y / y_{mo}$

where

$\varphi = 0.5 [1 + \alpha (\lambda - 0.2) + \lambda^2]$

λ= non-dimensional effective slenderness ratio

$= f_y / f_{cc} = \sqrt{(} f_y (KL/r)^2 / \prod^2 E)$

f_{cc} =Euler buckling stress=$\prod^2 E/(KL/r)^2$

KL/r =effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration, r

χ =stress reduction factor (see Table 8) for different buckling class, slenderness ratio and yield stress

$=1/(\varphi + [\varphi^2 - \lambda^2]^{0.5})$

Υ_{m0} =partial safety factor for material strength.

α=Imperfection factor from Table 7

IS 800:2007 CLAUSE 6.2 Design Strength Due to Yielding of Gross Section

The design strength of members(section ISMB350, member 5) under axial tension, T_d, as governed by yielding of gross section, is given by

$T_{dg} = A_g f_y / \Upsilon_{mo}$

where

f_y= yield stress of the material,

A_g= gross area of cross-section, and

Υ_{mo} = partial safety factor for failure in tension by yielding (see Table 5).

Table 5.1:Table of members failed and modified sections (by lateral force method)

Sl no.	Failed member no:	Failed section	Critical condition	Staad design section(passed)
1	1	ISMB350	IS 6.2	ISWB500
2	3,8,11,14,15	ISMB350	IS 6.2	ISLB550
3	10,12,17	ISMB350	IS 7.1.2	ISWB600
4	13	ISMB350	IS 6.2	ISHB450A
5	4,5,6,7,9,16,18	ISMB350	IS 7.1.2	ISWB600A
6	2	ISMB350	IS 6.2	ISHB450

Table 5.2:Table of members failed and new modified sections(by response spectrum analysis)

Sl no.	Failed member no:	Failed section	Critical condition	Staad design section(passed)
1	1,13	I80012B50012	IS 7.1.2	I80012B50016
2	2,14	I80012B50012	IS 7.1.2	I0012B55012
3	3,15	I80012B50012	IS 7.1.2	ISWB550
4	7,8,9,40,42	ISMB350	IS 6.2	I100012B50012
5	21	I80012B50012	IS 7.1.2	I100012B50012
6	27	I80012B50012	IS 7.1.2	ISWB600A
7	41	ISMB350	IS 6.2	ISMB600

5.1 CONNECTION DESIGN:

By considering node 16 ,the connection is made between the sections ISWB600A and I80012B50012 as described below

Welding connection of the steel plate to the flanges of the column is done by using full penetration butt weld, whose permissible stress may be, P_b=150Mpa and thickness of weld 't'=10mm

Length of weld required= $\sqrt{\frac{6\times M}{t\times \mathrm{Pb}}}$= $\sqrt{\frac{6\times 278.682\times 1000\times 1000}{10\times 150}}$ =1055mm

Maximum bending stress in the weld P_b =$\frac{M\times y}{I}$ = $\frac{6\times M}{t\times \mathrm{d2}}$ = $\frac{6\times 278.682\times 300\times 10^6}{850\times 300^3}$ =21.85Mpa

Shear stress at weld =$\frac{W}{d\times t}$= $\frac{120\times 10^3}{10\times 1294}$ =9.27Mpa

Final condition: P_e =$\sqrt{Pb2+3Ps2}$ = $\sqrt{21.85^2+3\times 9.27^2}$ =27.114Mpa< 225Mpa

Design is OK (so assumed steel plate of 824×850 mm is welded to the flanges of the column)

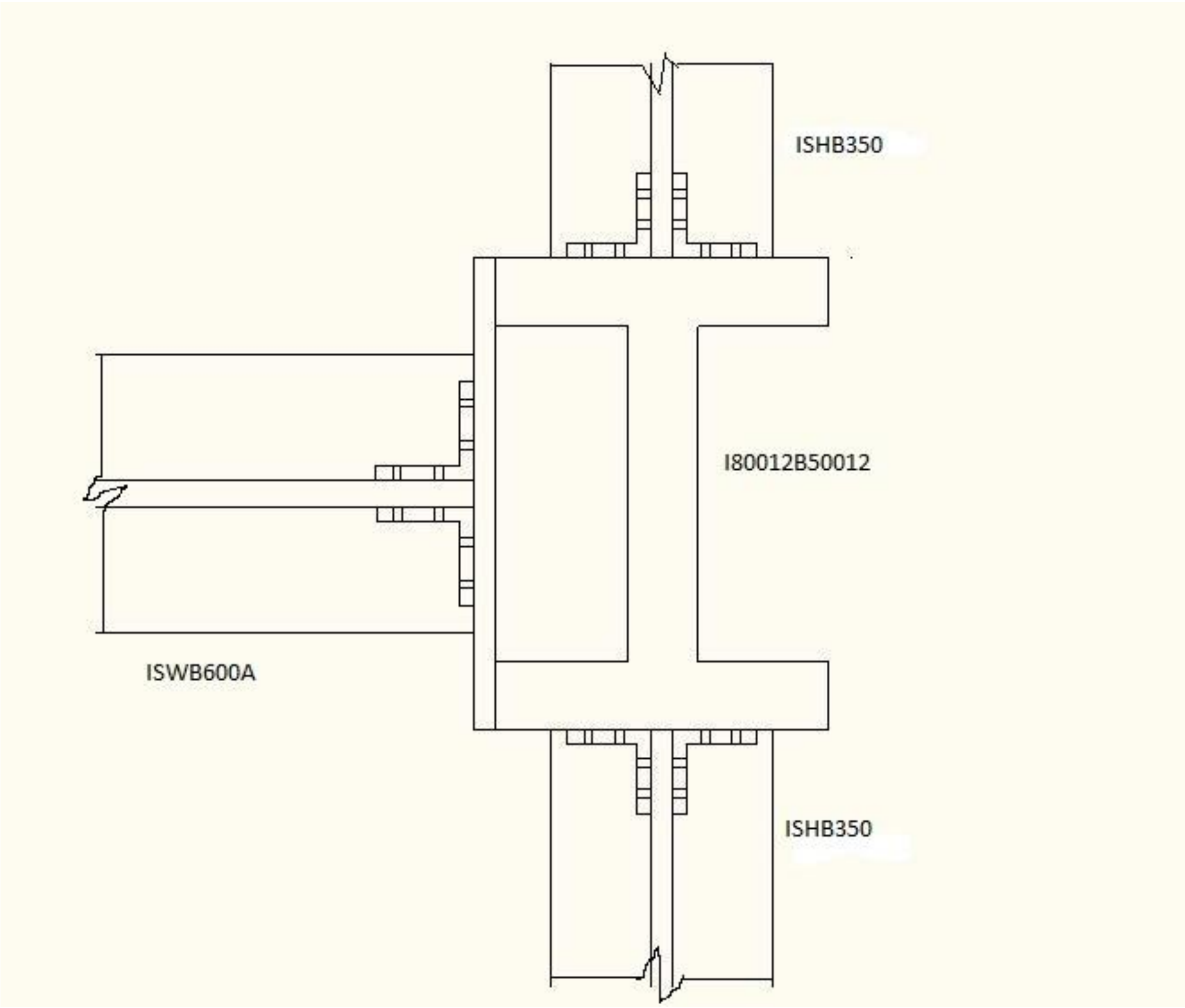


Fig 5.2 : Top view of exterior connection joint

CONNECTION OF BEAM TO THE STEEL-PLATE:

Consider 2 angle sections of ISA 100×100×8 and 20mm dia close tolerance turned bolts

n= $\sqrt{\frac{6\times M}{m\times p\times R}}$ (no. of bolts required) = $\sqrt{\frac{6\times 278.682}{4\times 0.06\times 108.915}}$ = 7.997 ≈ 8 bolts

R=bolt value (area× σ_{tf})

m= 4 lines, M= 278.682KN, W=120KN

After calculation n= 7.997 ≈ **8 no.s bolts per line**

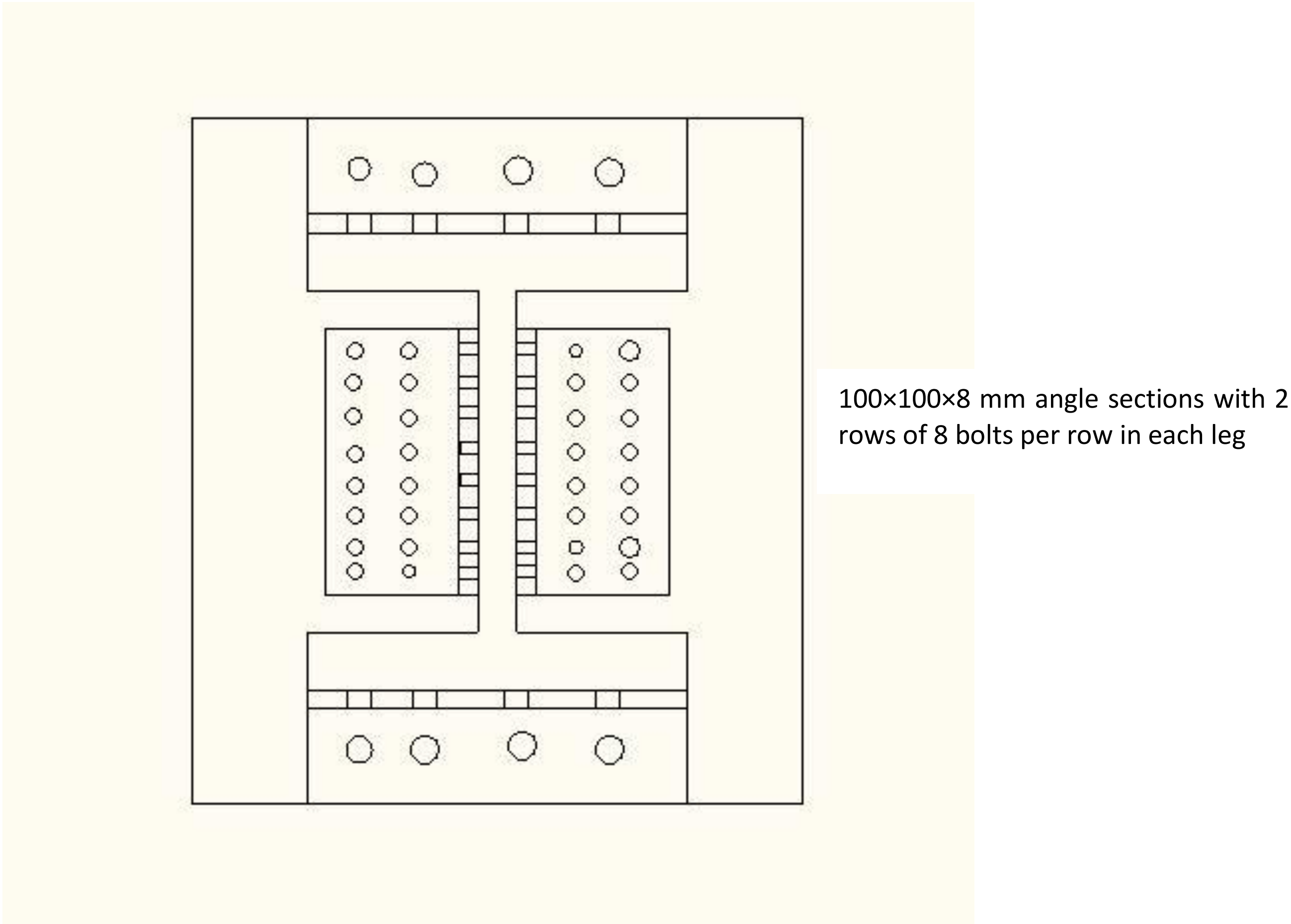


Fig 5.3 : Front view of connection between plate and beam using angle sections

Check for stresses:

σ_{tf} cal= $\frac{6M}{m\times p\times n2\times Ab}$= $\frac{6\times 278.682\times 100}{4\times 6\times 8^2\times\frac{\Pi}{4}\times 21.5^2}$ =299.8 Mpa

Shear stress on each bolt= $\frac{W}{m\times n\times Ab}$ = $\frac{120\times 10^3}{32\times\frac{\Pi}{4}\times 21.5^2}$ =1.03Mpa

Permissible combined shear and tensile stress :

$\frac{\tau vf\ cal}{\tau vf} + \frac{\sigma tf\ cal}{\sigma tf} \leq 1.4$

$= \frac{299.8}{300}+\frac{1.03}{100}$

$1.00963 \leq 1.4$ (OK)

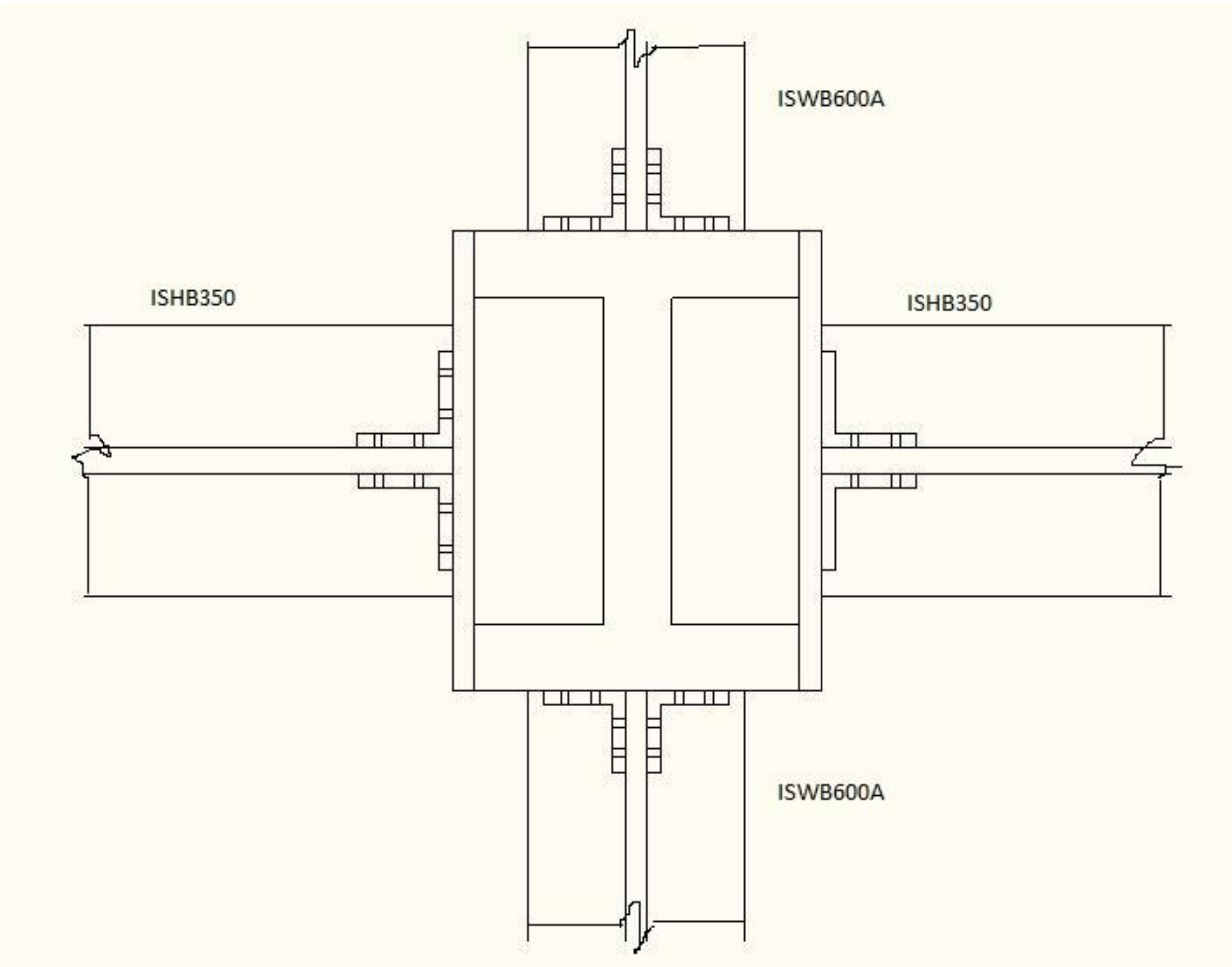


Fig 5.4: Top view of interior joint

UNSTIFFENED SEAT CONNECTION:

Assume 2 angle sections ISA 150×115×8

Strength of bolts in single shear = $\frac{300}{1000}\times\frac{\pi}{4}\times(21.5)2$= 108.9KN

Strength of bolts in bearing 12mm plate= $\frac{300}{1000}\times 21.5\times 12 = 77.4KN$

No. of bolts= $\frac{120}{77.4} = 1.55 \approx 2$

Bearing length a= $\frac{R}{\mathrm{tw}\times\sigma\mathrm{p}} - h_2\sqrt{3}$ = $\frac{120\times10^3}{11.8\times187.5}$ - 46.05× $\sqrt{3}$=-25.52 (negative)

But bearing length a≥ $\frac{R}{2\times\mathrm{tw}\times\sigma\mathrm{p}}$ = $\frac{120\times10^3}{11.8\times187.5}$ = 27.11mm

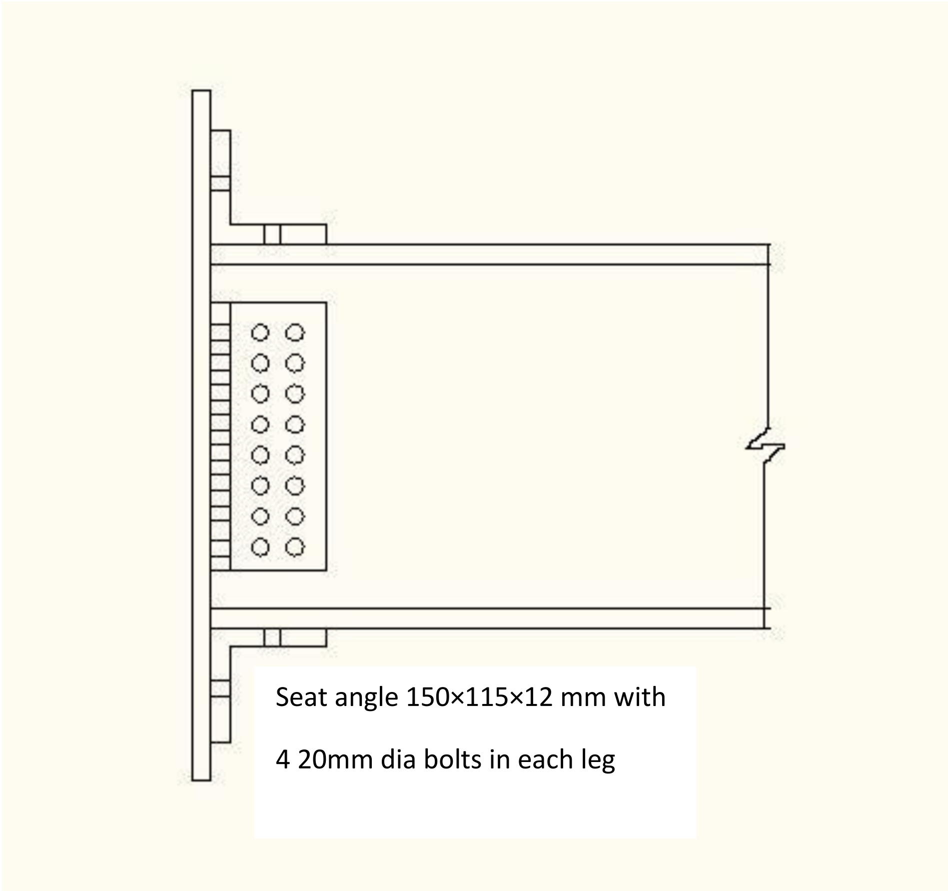


Fig 5.5: Unstiffened seat connection

Check for thickness

Max bending moment at critical section ;;

R $(12+\frac{a}{2}-t-r) = 120\times 10^3(12+\frac{27.11}{2}-8-11) = 7.86\times10^5$ N mm

At critical section, moment of resistance= $\sigma_{bt} \times\frac{l\times t2}{6} = 165\times 250\times\frac{t^2}{6} = 6875\ t^2$

Equating bending moment and moment of resistance

$6875t^2 = 7.86\times10^5$, $t=\sqrt{\frac{78.6\times10^5}{6875}} = 10.69$mm

t=10.69mm ≈ 12mm

Therefore take 12mm thickness

i.e. 150×115×12 is used for seat angle

5.2 BASE PLATE DESIGN:

For I80012B50012 column

The base plate has to carry additional stress due to bending moment in the column as it is eccentrically loaded. Stress due to vertical load ‘P’ and moment ‘M’ at the extreme fibre is given by $\frac{P}{b\times l} \pm \frac{6M}{b\times l2}$

σ_{bs} = 185Mpa

In single shear = $\boldsymbol{\tau}_{tf}\times\frac{\pi}{4}\times d^2 = \frac{100}{1000}\times\frac{\pi}{4}\times 25.52 = 51.05KN$

Therefore bolt value= 51.05KN

Force due to moment= $\frac{moment}{lever\ arm\ between\ flanges} = \frac{480093}{824}$ =582.63KN

Assume 50% of the axial load is transferred by direct bearing and 50% on both flanges. If the end is faced for complete bearing on base plate.

Total load transferred through one flange to angle section= $\frac{1}{2}\times\frac{1963.83}{2}$+ 582.63 = 1073.59KN

No. of bolts= $\frac{1073.59}{51.07}$= 21.02 ≈22 bolts

Provide 22 of 24mm diameter bolts to connect column to the base plate

Take length of base plate= 824+300 =112mm, take 1150mm

Since, $\frac{M}{P} = \frac{480093}{1963.83}$ =244.46mm > $\frac{1150}{6}=\frac{l}{6}$

Therefore the stress diagram will be

$\approx\frac{x}{3} = \frac{l}{2} - \frac{M}{P} = \frac{824}{2} - 244.46$

X=502.62mm

The extreme fibre stress σ is negative and a part of the base plate is lifted up. The resultant upward pressure R should be equal to the downward load P

Therefore R=$\frac{\sigma\times x\times b}{2}$ =P

b= $\frac{P\times 2}{\sigma\times x}$= $\frac{2\times 1963.83\times 1000}{8\times 502.62}$= 976.796 ≈980mm

Assume = **1150×980mm base plate**

Thickness of base plate:

Bending moment at section Z-Z

$Mz = 5.33\times(151\times980)\times\frac{151}{2}+\frac{8-5.33}{2}\times(151\times980)\times\frac{2}{3}\times151 = 79436403.9$

Equating bending moment to the moment of resistance

$(\frac{1}{6})\times980\times t^2\times185 = 79436403.9$

T= 51.27mm

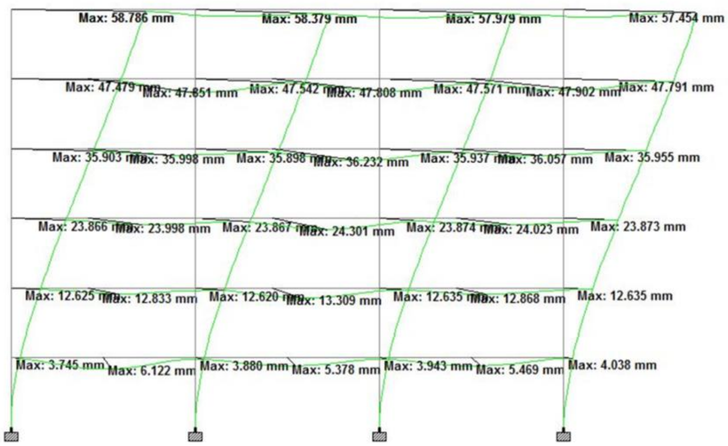
Thickness of base plate= t- thickness of angle= 51.27 – 12= 39.27 ≈**40mm**

CHAPTER-6

RESULTS AND DISCUSSIONS

RESULTS OF LATERAL FORCE METHOD:

Maximum bending moment, shear force etc. are obtained for load combination 1.7(EQ+DL)



FIG(6.1) Displacement diagram for load combination 1.7(EQ+DL)

The inter storey drift as seen from above diagram is within the limits of deflection of the code i.e. it is within .004 of storey height= 0.004X3000= 12mm.

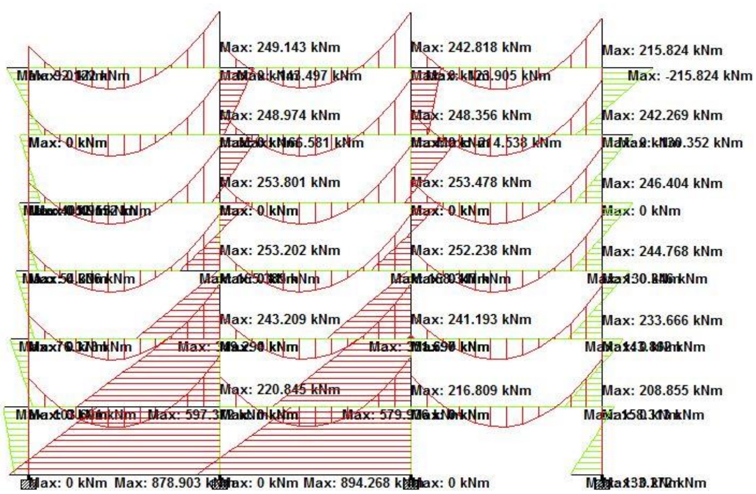
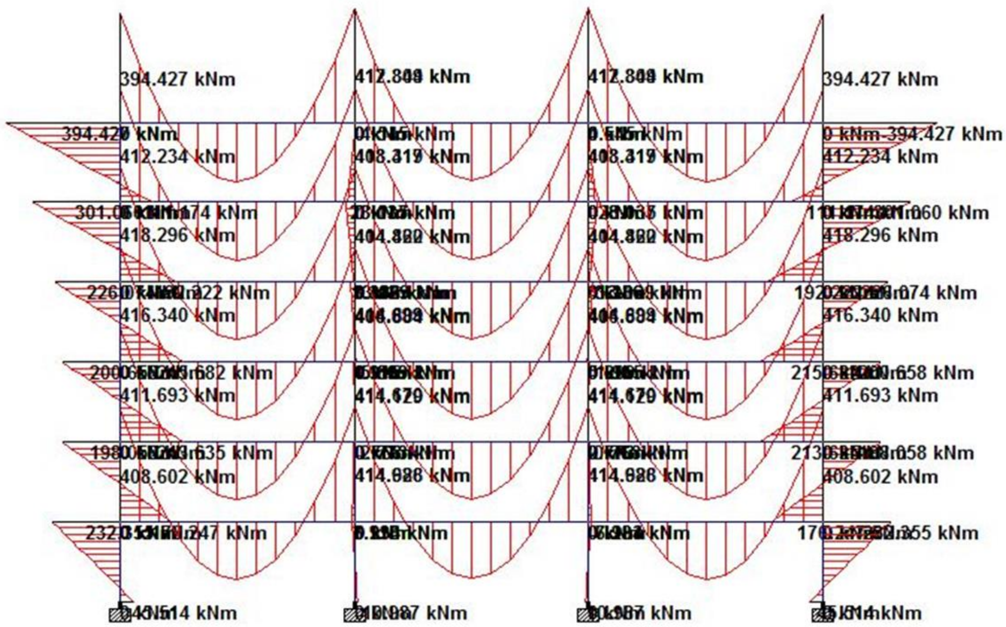


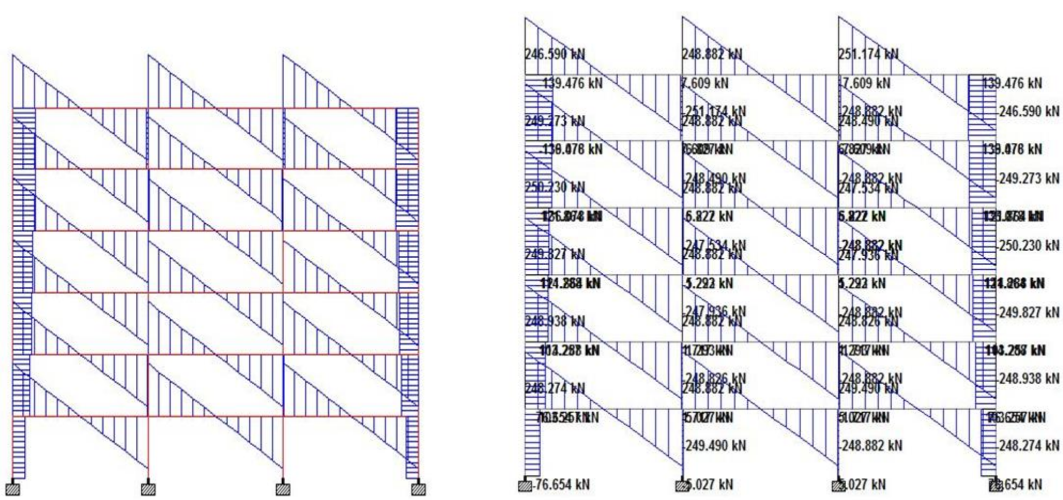
FIG (6.2) Bending moment diagram for load combination 1.7(EQ+DL)

RESULTS OF RESPONSE SPECTRUM ANALYSIS:

Maximum bending moment, shear force etc. are obtained for load combination 1.3(DL+LL+EQ)



Fig(6.3) Bending moment diagram for load combination 1.3(DL+LL+EQ)

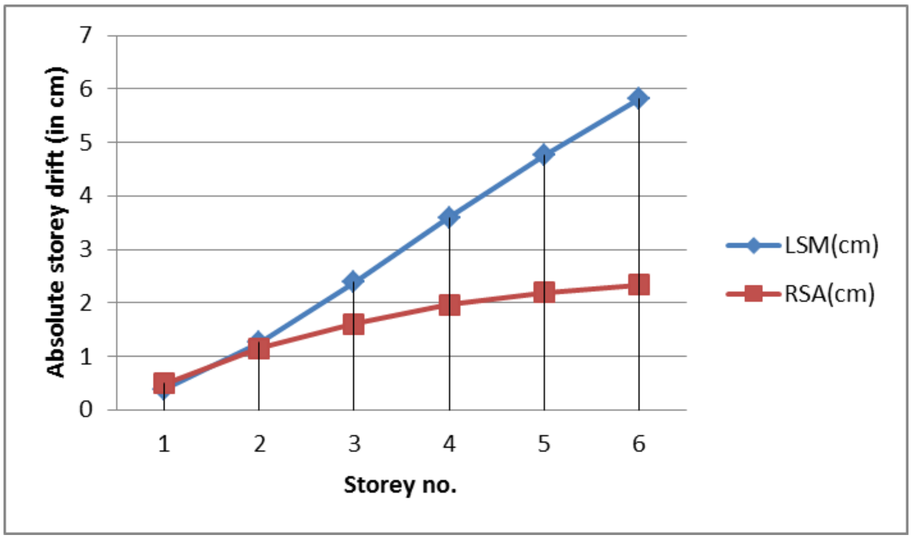


Fig(6.4) shear force diag.in X-axis shear force diag. in Y-axis

Load combination is same in both cases-Load case 1.3(DL+LL+EQ).

Comparison of absolute storey drift in both methods: (table 6.1)

Storey no.	Storey height	LSM(cm)	RSA(cm)
1	3	0.3869	0.491
2	6	1.2595	1.15
3	9	2.3837	1.61
4	12	3.5892	1.96
5	15	4.7566	2.19
6	18	5.8123	2.34



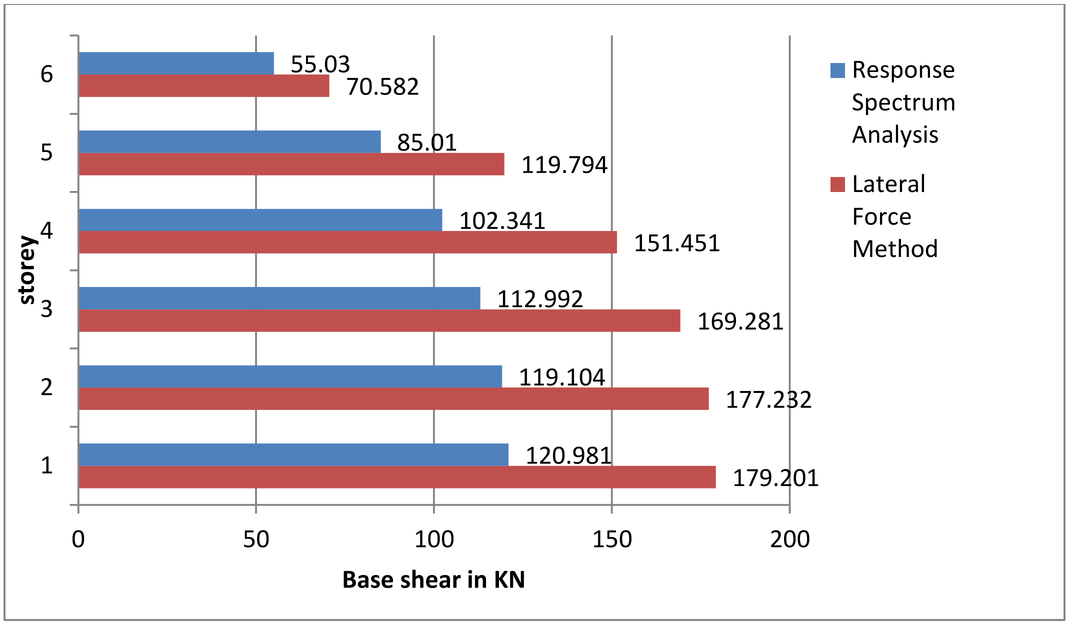
Fig(6.5) Graph of comparison of absolute storey drift

<u>Table(6.2) Comparison of storey shear: (using both LSM and RSA)</u>

Storey no.	Storey height	LSM (KN)	RSA (KN)	Difference in %
1	3	179.201	120.981	28.91
2	6	177.232	119.104	32.79
3	9	169.281	112.992	33.25
4	12	151.451	102.341	32.42
5	15	119.794	85.01	28.99
6	18	70.582	55.03	22.033

It is found that the difference storey shear by both these methods are about 29.73 %at an average per storey.

Fig(6.6) Graph of comparison of storey shear

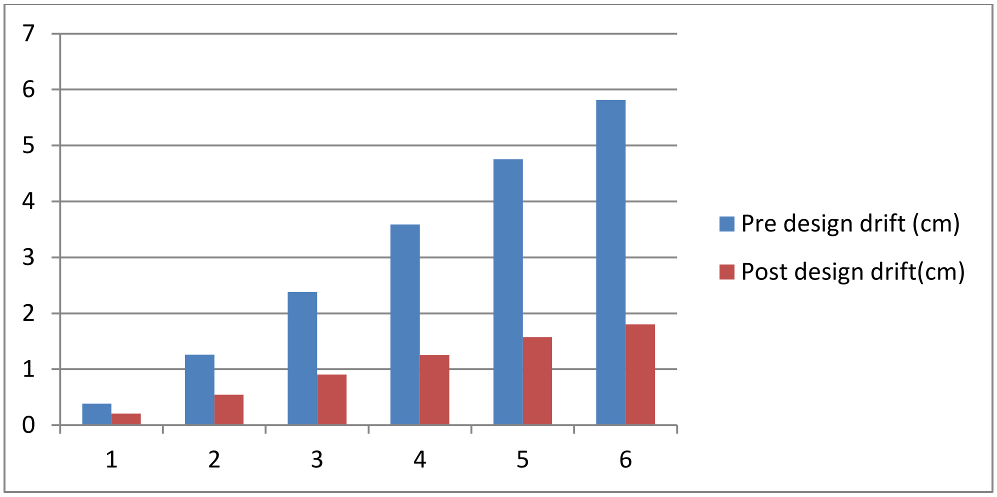


Final results compared with initial design result:

Table (6.3)Drift: By Lateral Force Method

Storey no.	Pre design drift (cm)	Post design drift(cm)	Difference in %
1	0.3869	0.2056	46.85
2	1.2595	0.5472	56.55
3	2.3837	0.9052	68.11
4	3.5892	1.2561	65
5	4.7566	1.5729	66.93
6	5.8123	1.8012	69.05

It is observed that the difference in drift in post and pre design is almost as high as 62.08% at an average per storey.



Fig(6.7) Graph of storey drift for final and initial design results

Response Spectrum Method:

Participation factor:

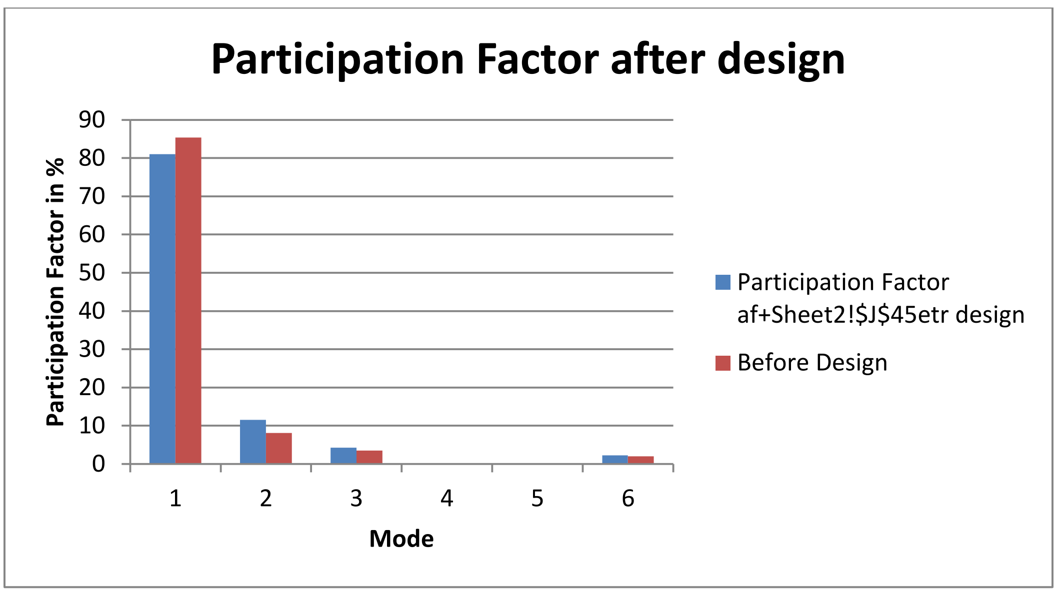


Fig. (6.8) graph of mode participation for final and initial design results

Total amount of steel required in the form of connections and member sections are more for analysis and design based on response spectrum method than lateral force method

CHAPTER-7

CONCLUSION

CONCLUSION

1. Inter storey drift was found out using lateral force method and response spectrum method and it was found that the displacements of response spectrum method was less than that of lateral force method.
2. Storey shear found by response spectrum method is less than that found by lateral force method.
3. The difference in results of response spectrum and lateral force method are attributed to certain assumptions prevalent in the lateral force method. They are:

a. The fundamental mode of the building makes most significant contribution to the base shear.
b. The total building mass is considered as against the modal mass that is used in dynamic procedure. Both the assumptions are valid for low and medium rise buildings which are regular.

4. As observed in the above results the values obtained by following dynamic analysis are smaller than those of lateral force method. This is so because the first mode period by dynamic analysis is 0.62803 is greater than the estimated 0.33 s of lateral force method.
5. The analysis also shows that the first modal mass is 85.33% of total seismic mass. The second modal mass is 8.13% of the total seismic mass m and the time period is 0.19s.
6. In the post design analysis the inter storey drift and base shear both have decreased significantly owing to heavier member sections leading to safe design. For example the initially used sections (eg:-**ISMB 350**) have failed and Staad Pro has redesigned and adopted higher section(eg:-**ISWB 600 A)**
7. The steel take off or the cost of steel used (which is directly proportional to the amount of steel used)is less in lateral force method as compared to the response spectrum method. This is so because the response spectrum method, being dynamic in nature, is a more accurate method taking into account many more parameters like mode shape, mass participation factors to calculate the seismic vibration results. Response spectrum method is more realistic method of analysis and design of steel building frame and from the present work it is found that lateral force method leads to more cost effective of seismic design of steel frame.
8. The amount of steel required for seismic design by using lateral force method is found to be 19.73% less than that by using response spectrum analysis
9. Because of the heavier sections used in response spectrum method the absolute displacement, storey drift are less than lateral force method
10. It is found that the inter storey drift sensitivity coefficient θ does not differ much in both the methods of analysis
11. The values of resultant base shear in lateral force method is 49.33 % more than that of response spectrum method

CHAPTER-8

ANNEXURE

CODE AND REPORT:

LATERAL STATIC METHOD

X-FRAME

```
CODE
FOR LATERAL FORCE METHOD X-FRAME
STAAD PLANE
START JOB INFORMATION
ENGINEER DATE 04-May-13
END JOB INFORMATION
INPUT WIDTH 79
UNIT METER KN
JOINT COORDINATES
1 0 0 0; 2 8 0 0; 3 16 0 0; 4 24 0 0; 5 0 3 0; 6 8 3 0; 7 16 3 0; 8 24 3 0;
9 0 6 0; 10 8 6 0; 11 16 6 0; 12 24 6 0; 13 0 9 0; 14 8 9 0; 15 16 9 0;
16 24 9 0; 17 0 12 0; 18 8 12 0; 19 16 12 0; 20 24 12 0; 21 0 15 0; 22 8 15 0;
23 16 15 0; 24 24 15 0; 25 0 18 0; 26 8 18 0; 27 16 18 0; 28 24 18 0;
MEMBER INCIDENCES
1 5 6; 2 6 7; 3 7 8; 4 9 10; 5 10 11; 6 11 12; 7 13 14; 8 14 15; 9 15 16;
10 17 18; 11 18 19; 12 19 20; 13 21 22; 14 22 23; 15 23 24; 16 25 26; 17 26 27;
18 27 28; 19 1 5; 20 2 6; 21 3 7; 22 4 8; 23 5 9; 24 6 10; 25 7 11; 26 8 12;
27 9 13; 28 10 14; 29 11 15; 30 12 16; 31 13 17; 32 14 18; 33 15 19; 34 16 20;
35 17 21; 36 18 22; 37 19 23; 38 20 24; 39 21 25; 40 22 26; 41 23 27; 42 24 28;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2.05e+008
POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
```

```
DAMP 0.03

END DEFINE MATERIAL

MEMBER PROPERTY INDIAN

19 TO 42 TABLE ST I80012B50012

1 TABLE ST ISWB500

3 8 11 14 15 TABLE ST ISLB550

10 12 17 TABLE ST ISWB600

13 TABLE ST ISHB450A

4 5 6 7 9 16 18 TABLE ST ISWB600A

2 TABLE ST ISHB450

CONSTANTS

BETA 90 MEMB 19 22 23 26 27 30 31 34 35 38 39 42

MATERIAL STEEL ALL

SUPPORTS

1 TO 4 FIXED

DEFINE 1893 LOAD

ZONE 0.16 RF 5 I 1.5 SS 2 ST 2 DM 3

SELFWEIGHT 1

JOINT WEIGHT

5 TO 28 WEIGHT 144

MEMBER WEIGHT

1 TO 18 UNI 30

CHECK SOFT STOREY

LOAD 1 LOADTYPE Seismic  TITLE EQ

1893 LOAD X 1

LOAD 2 LOADTYPE Dead  TITLE DL

SELFWEIGHT Y -1 LIST 1 TO 42

MEMBER LOAD

1 TO 18 UNI GY -18.75

LOAD 3 LOADTYPE Live  TITLE LL
```

MEMBER LOAD

1 TO 18 UNI GY -11.25

LOAD COMB 4 GENERATED INDIAN CODE STEEL_PLASTIC 1

2 1.7 3 1.7

LOAD COMB 5 GENERATED INDIAN CODE STEEL_PLASTIC 2

2 1.7 1 1.7

LOAD COMB 6 GENERATED INDIAN CODE STEEL_PLASTIC 3

2 1.7 1 -1.7

LOAD COMB 7 GENERATED INDIAN CODE STEEL_PLASTIC 4

2 1.3 3 1.3 1 1.3

LOAD COMB 8 GENERATED INDIAN CODE STEEL_PLASTIC 5

2 1.3 3 1.3 1 -1.3

LOAD COMB 9 GENERATED INDIAN CODE STEEL_PLASTIC 6

2 1.7

LOAD COMB 10 GENERATED INDIAN CODE STEEL_PLASTIC 7

2 1.3 3 1.3

PERFORM ANALYSIS PRINT ALL

PRINT STORY DRIFT

PARAMETER 1

CODE IS800 LSD

CHECK CODE ALL

PARAMETER 2

CODE IS800 LSD

STEEL TAKE OFF LIST 1 TO 42

PARAMETER 3

CODE IS800 LSD

SELECT ALL

FINISH

STOREY DRIFT:

STORY	HEIGHT (METE)	LOAD	AVG. DISP (CM) X	AVG. DISP (CM) Z	DRIFT(CM) X	DRIFT(CM) Z	RATIO
BASE=	0.00						
1	0.00	1	0.0000	0.0000	0.0000	0.0000	L /999999
		2	0.0000	0.0000	0.0000	0.0000	L /999999
		3	0.0000	0.0000	0.0000	0.0000	L /999999
		4	0.0000	0.0000	0.0000	0.0000	L /999999
		5	0.0000	0.0000	0.0000	0.0000	L /999999
		6	0.0000	0.0000	0.0000	0.0000	L /999999
		7	0.0000	0.0000	0.0000	0.0000	L /999999
		8	0.0000	0.0000	0.0000	0.0000	L /999999
		9	0.0000	0.0000	0.0000	0.0000	L /999999
		10	0.0000	0.0000	0.0000	0.0000	L /999999
2	3.00	1	0.1208	0.0000	0.1208	0.0000	L / 2482
		2	0.0001	0.0000	0.0001	0.0000	L /999999
		3	0.0001	0.0000	0.0001	0.0000	L /999999
		4	0.0003	0.0000	0.0003	0.0000	L /999999
		5	0.2056	0.0000	0.2056	0.0000	L / 1459
		6	-0.2053	0.0000	0.2053	0.0000	L / 1461
		7	0.1573	0.0000	0.1573	0.0000	L / 1907
		8	-0.1569	0.0000	0.1569	0.0000	L / 1912
		9	0.0002	0.0000	0.0002	0.0000	L /999999
		10	0.0002	0.0000	0.0002	0.0000	L /999999

3	6.00	1	0.3218	0.0000	0.2009	0.0000 L /	1493
		2	0.0001	0.0000	0.0000	0.0000 L /	999999
		3	0.0001	0.0000	0.0000	0.0000 L /	999999
		4	0.0004	0.0000	0.0001	0.0000 L /	999999
		5	0.5472	0.0000	0.3416	0.0000 L /	878
		6	-0.5467	0.0000	0.3415	0.0000 L /	878
		7	0.4186	0.0000	0.2613	0.0000 L /	1148
		8	-0.4180	0.0000	0.2611	0.0000 L /	1149
		9	0.0003	0.0000	0.0001	0.0000 L /	999999
		10	0.0003	0.0000	0.0001	0.0000 L /	999999
4	9.00	1	0.5322	0.0000	0.2105	0.0000 L /	1425
		2	0.0002	0.0000	0.0001	0.0000 L /	999999
		3	0.0001	0.0000	0.0000	0.0000 L /	999999
		4	0.0006	0.0000	0.0002	0.0000 L /	999999
		5	0.9052	0.0000	0.3579	0.0000 L /	838
		6	-0.9044	0.0000	0.3577	0.0000 L /	839
		7	0.6924	0.0000	0.2738	0.0000 L /	1096
		8	-0.6914	0.0000	0.2734	0.0000 L /	1097
		9	0.0004	0.0000	0.0001	0.0000 L /	999999
		10	0.0005	0.0000	0.0002	0.0000 L /	999999
5	12.00	1	0.7385	0.0000	0.2063	0.0000 L /	1454
		2	0.0004	0.0000	0.0002	0.0000 L /	999999

STAAD PLANE -- PAGE NO. 12

STORY	HEIGHT (METE)	LOAD	AVG. DISP(CM) X	AVG. DISP(CM) Z	DRIFT(CM) X	DRIFT(CM) Z	RATIO
BASE=	0.00						
		3	0.0002	0.0000	0.0001	0.0000	L /999999
		4	0.0010	0.0000	0.0004	0.0000	L /715283
		5	1.2561	0.0000	0.3509	0.0000	L / 855
		6	-1.2548	0.0000	0.3504	0.0000	L / 856
		7	0.9608	0.0000	0.2685	0.0000	L / 1117
		8	-0.9593	0.0000	0.2678	0.0000	L / 1120
		9	0.0006	0.0000	0.0003	0.0000	L /999999
		10	0.0008	0.0000	0.0003	0.0000	L /935370
6	15.00	1	0.9244	0.0000	0.1858	0.0000	L / 1614
		2	0.0009	0.0000	0.0005	0.0000	L /561227
		3	0.0005	0.0000	0.0003	0.0000	L /947767
		4	0.0025	0.0000	0.0014	0.0000	L /207350
		5	1.5729	0.0000	0.3168	0.0000	L / 947
		6	-1.5699	0.0000	0.3150	0.0000	L / 952
		7	1.2035	0.0000	0.2427	0.0000	L / 1236
		8	-1.1998	0.0000	0.2405	0.0000	L / 1247
		9	0.0015	0.0000	0.0009	0.0000	L /330133
		10	0.0019	0.0000	0.0011	0.0000	L /271150
7	18.00	1	1.0581	0.0000	0.1338	0.0000	L / 2243
		2	0.0014	0.0000	0.0005	0.0000	L /592800
		3	0.0008	0.0000	0.0003	0.0000	L /992852
		4	0.0038	0.0000	0.0014	0.0000	L /218341

5	1.8012	0.0000	0.2283	0.0000 L / 1314
6	-1.7964	0.0000	0.2265	0.0000 L / 1324
7	1.3785	0.0000	0.1749	0.0000 L / 1715
8	-1.3726	0.0000	0.1728	0.0000 L / 1736
9	0.0024	0.0000	0.0009	0.0000 L /348706
10	0.0029	0.0000	0.0011	0.0000 L /285523

STEEL DESIGN:

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER TABLE RESULT/ CRITICAL COND/ RATIO/ LOADING/

FX MY MZ LOCATION

===

1 ST ISWB500 (INDIAN SECTIONS)

PASS IS-6.2 0.889 4

27.73 T 0.00 299.40 8.00

2 ST ISHB450 (INDIAN SECTIONS)

PASS IS-6.2 0.991 4

16.24 T 0.00 280.03 8.00

3 ST ISLB550 (INDIAN SECTIONS)

PASS IS-6.2 0.960 4

27.44 T 0.00 298.80 0.00

4 ST ISWB600A (INDIAN SECTIONS)

PASS IS-7.1.2 0.924 7

14.56 C 0.00 340.97 8.00

5 ST ISWB600A (INDIAN SECTIONS)

PASS IS-7.1.2 0.916 5

0.40 C 0.00 342.62 8.00

6 ST ISWB600A (INDIAN SECTIONS)

PASS IS-7.1.2 0.923 8

14.07 C 0.00 340.84 0.00

7 ST ISWB600A (INDIAN SECTIONS)

PASS IS-7.1.2 0.917 7

1.94 C 0.00 342.53 8.00

8 ST ISLB550 (INDIAN SECTIONS)

PASS IS-6.2 0.886 4

2.79 T 0.00 280.12 8.00

9 ST ISWB600A (INDIAN SECTIONS)

PASS IS-7.1.2 0.916 8

1.77 C 0.00 342.29 0.00

STAAD PLANE -- PAGE NO. 20

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER TABLE RESULT/ CRITICAL COND/ RATIO/ LOADING/

FX MY MZ LOCATION

===

10 ST ISWB600 (INDIAN SECTIONS)

PASS IS-7.1.2 0.813 8

0.06 C 0.00 266.52 0.00

11 ST ISLB550 (INDIAN SECTIONS)

PASS IS-6.2 0.890 4

	8.48 T	0.00	280.07	8.00
12 ST ISWB600		(INDIAN SECTIONS)		
	PASS	IS-7.1.2	0.810	5
	0.43 C	0.00	265.44	8.00
13 ST ISHB450A		(INDIAN SECTIONS)		
	PASS	IS-6.2	0.980	4
	18.04 T	0.00	284.97	8.00
14 ST ISLB550		(INDIAN SECTIONS)		
	PASS	IS-6.2	0.884	4
	1.76 T	0.00	279.54	8.00
15 ST ISLB550		(INDIAN SECTIONS)		
	PASS	IS-6.2	0.909	4
	18.02 T	0.00	284.30	0.00
16 ST ISWB600A		(INDIAN SECTIONS)		
	PASS	IS-7.1.2	0.946	4
	99.37 C	0.00	320.23	8.00
17 ST ISWB600		(INDIAN SECTIONS)		
	PASS	IS-7.1.2	0.954	4
	82.71 C	0.00	285.28	8.00
18 ST ISWB600A		(INDIAN SECTIONS)		
	PASS	IS-7.1.2	0.943	4
	98.47 C	0.00	319.60	0.00

STAAD PLANE -- PAGE NO. 21

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER TABLE RESULT/ CRITICAL COND/ RATIO/ LOADING/

```
                    FX           MY            MZ       LOCATION
=======================================================================

   19 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.610         4
              1293.18 C       107.12          0.00       3.00
   20 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.550         5
              1764.08 C         0.00        471.64       0.00
   21 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.550         6
              1767.27 C         0.00       -471.31       0.00
   22 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.609         4
              1291.80 C      -106.75          0.00       3.00
   23 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.648         4
              1090.94 C      -127.47          0.00       0.00
   24 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.398         4
              2244.23 C         0.00         17.15       0.00
   25 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.398         4
              2244.74 C         0.00        -16.36       0.00
   26 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.647         7
               933.40 C       134.80          0.00       0.00
   27 ST   I80012B50012            (INDIAN SECTIONS)
                  PASS     IS-7.1.2          0.624         8
```

738.29 C -137.75 0.00 0.00

STAAD PLANE -- PAGE NO. 22

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER TABLE RESULT/ CRITICAL COND/ RATIO/ LOADING/

FX MY MZ LOCATION

===

28 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.313 4

1791.76 C 0.00 4.65 0.00

29 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.313 4

1792.21 C 0.00 -4.64 0.00

30 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.625 7

740.26 C 137.92 0.00 0.00

31 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.594 8

531.32 C 139.30 0.00 3.00

32 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.238 4

1344.35 C 0.00 8.24 0.00

33 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.237 4

1344.86 C 0.00 -7.43 0.00

34 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.594 7

533.23 C -139.17 0.00 3.00

35 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.521 4

416.17 C 124.66 0.00 3.00

36 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.183 6

579.96 C 0.00 160.01 3.00

STAAD PLANE -- PAGE NO. 23

ALL UNITS ARE - KN METE (UNLESS OTHERWISE NOTED)

MEMBER TABLE RESULT/ CRITICAL COND/ RATIO/ LOADING/

FX MY MZ LOCATION

===

37 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.186 5

576.96 C 0.00 -166.40 3.00

38 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.523 7

352.34 C 128.17 0.00 0.00

39 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.628 4

194.24 C 164.87 0.00 3.00

40 ST I80012B50012 (INDIAN SECTIONS)

PASS IS-7.1.2 0.139 5

```
              280.04 C        0.00       -175.71       3.00
     41 ST   I80012B50012           (INDIAN SECTIONS)
                   PASS     IS-7.1.2          0.139        6
              280.33 C        0.00        174.78       3.00
     42 ST   I80012B50012           (INDIAN SECTIONS)
                   PASS     IS-7.1.2          0.627        4
              194.31 C     -164.82          0.00       3.00
```

STEEL TAKEOFF:

PROFILE	LENGTH(METE)	WEIGHT(KN)
ST ISWB500	8.00	7.448
ST ISHB450	8.00	6.828
ST ISLB550	40.00	33.801
ST ISWB600A	56.00	79.542
ST ISWB600	24.00	31.416
ST ISHB450A	8.00	7.246
ST I80012B50012	72.00	214.050

TOTAL = 380.330

Y- FRAME: **CODE**

STAAD PLANE

START JOB INFORMATION

ENGINEER DATE 30-Apr-13

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 6 0 0; 3 12 0 0; 4 18 0 0; 5 24 0 0; 6 30 0 0; 7 0 3 0; 8 6 3 0;

```
9 12 3 0; 10 18 3 0; 11 24 3 0; 12 30 3 0; 13 0 6 0; 14 6 6 0; 15 12 6 0;
16 18 6 0; 17 24 6 0; 18 30 6 0; 19 0 9 0; 20 6 9 0; 21 12 9 0; 22 18 9 0;
23 24 9 0; 24 30 9 0; 25 0 12 0; 26 6 12 0; 27 12 12 0; 28 18 12 0; 29 24 12 0;
30 30 12 0; 31 0 15 0; 32 6 15 0; 33 12 15 0; 34 18 15 0; 35 24 15 0;
36 30 15 0; 37 0 18 0; 38 6 18 0; 39 12 18 0; 40 18 18 0; 41 24 18 0;
42 30 18 0;
MEMBER INCIDENCES
1 37 31; 2 25 31; 3 25 19; 4 19 13; 5 7 13; 6 7 1; 7 37 38; 8 38 32; 9 32 31;
10 32 26; 11 26 25; 12 26 20; 13 20 19; 14 14 20; 15 14 13; 16 14 8; 17 8 7;
18 8 2; 19 38 39; 20 39 40; 21 40 41; 22 41 42; 23 42 36; 24 36 30; 25 30 24;
26 24 18; 27 18 12; 28 12 6; 29 5 11; 30 17 11; 31 17 23; 32 23 29; 33 29 35;
34 35 41; 35 40 34; 36 34 28; 37 28 22; 38 22 16; 39 16 10; 40 10 4; 41 39 33;
42 33 27; 43 27 21; 44 21 15; 45 15 9; 46 9 3; 47 32 33; 48 33 34; 49 34 35;
50 35 36; 51 30 29; 52 29 28; 53 28 27; 54 26 27; 55 20 21; 56 21 22; 57 22 23;
58 23 24; 59 18 17; 60 17 16; 61 16 15; 62 15 14; 63 8 9; 64 9 10; 65 10 11;
66 11 12;
DEFINE MATERIAL START
ISOTROPIC STEEL
E 2.05e+008
POISSON 0.3
DENSITY 76.8195
ALPHA 1.2e-005
DAMP 0.03
END DEFINE MATERIAL
MEMBER PROPERTY INDIAN
7 9 11 13 15 17 19 TO 22 47 TO 66 TABLE ST ISMB350
1 TO 6 8 10 12 14 16 18 23 TO 46 TABLE ST I80012B50012
CONSTANTS
MATERIAL STEEL ALL
```

SUPPORTS

1 TO 6 FIXED

DEFINE 1893 LOAD

ZONE 0.075 RF 1 I 1 SS 1 DM 3

SELFWEIGHT 1

JOINT WEIGHT

8 TO 11 14 TO 17 20 TO 23 26 TO 29 32 TO 35 38 TO 41 WEIGHT 240

7 12 13 18 19 24 25 30 31 36 37 42 WEIGHT 120

MEMBER WEIGHT

7 9 11 13 15 17 19 TO 22 47 TO 66 UNI 24

LOAD 1 LOADTYPE Seismic TITLE EQX 1

1893 LOAD X 1

LOAD 2 LOADTYPE Dead TITLE DL

SELFWEIGHT Y -1 LIST 1 TO 66

MEMBER LOAD

7 9 11 13 15 17 19 TO 22 47 TO 66 UNI GY -15

LOAD 3 LOADTYPE Live TITLE LL

MEMBER LOAD

7 9 11 13 15 17 19 TO 22 47 TO 66 UNI GY -9

LOAD COMB 4 GENERATED INDIAN CODE STEEL_PLASTIC 1

2 1.7 3 1.7

LOAD COMB 5 GENERATED INDIAN CODE STEEL_PLASTIC 2

2 1.7 1 1.7

LOAD COMB 6 GENERATED INDIAN CODE STEEL_PLASTIC 3

2 1.7 1 -1.7

LOAD COMB 7 GENERATED INDIAN CODE STEEL_PLASTIC 4

2 1.3 3 1.3 1 1.3

LOAD COMB 8 GENERATED INDIAN CODE STEEL_PLASTIC 5

2 1.3 3 1.3 1 -1.3

LOAD COMB 9 GENERATED INDIAN CODE STEEL_PLASTIC 6

2 1.7

LOAD COMB 10 GENERATED INDIAN CODE STEEL_PLASTIC 7

2 1.3 3 1.3

PERFORM ANALYSIS PRINT ALL

PRINT STORY DRIFT

PARAMETER 1

CODE IS800 LSD

CHECK CODE ALL

PARAMETER 2

CODE IS800 LSD

SELECT ALL

PARAMETER 3

CODE IS800 LSD

STEEL TAKE OFF LIST 1 TO 66

FINISH

STEEL TAKEOFF:

PROFILE	LENGTH(METE)	WEIGHT(KN)
ST I80012B50012	108.00	321.075
ST ISHB400	60.00	45.493
ST ISHB350	36.00	23.756
ST ISMB350	54.00	27.669
ST ISWB350	30.00	16.708

	TOTAL =	434.700

Therefore total takeoff of the steel building is:

$= 6\times\ (steel\ takeoff\ of\ x-frame)$
$+4\times\ (steel\ takeoff\ of\ y-frame) - 4\times(wt\ of\ column\ sections\ in\ y-frame)$

$= 6\times\ (380.330) + 4\times(437.700) - 4\times(321.075)$

=2748.48 KN

Response spectrum analysis:

X-frame (CODE)

STAAD PLANE

START JOB INFORMATION

ENGINEER DATE 04-May-13

END JOB INFORMATION

INPUT WIDTH 79

UNIT METER KN

JOINT COORDINATES

1 0 0 0; 2 8 0 0; 3 16 0 0; 4 24 0 0; 5 0 3 0; 6 8 3 0; 7 16 3 0; 8 24 3 0;

9 0 6 0; 10 8 6 0; 11 16 6 0; 12 24 6 0; 13 0 9 0; 14 8 9 0; 15 16 9 0;

16 24 9 0; 17 0 12 0; 18 8 12 0; 19 16 12 0; 20 24 12 0; 21 0 15 0; 22 8 15 0;

23 16 15 0; 24 24 15 0; 25 0 18 0; 26 8 18 0; 27 16 18 0; 28 24 18 0;

MEMBER INCIDENCES

1 1 5; 2 5 9; 3 9 13; 4 13 17; 5 17 21; 6 21 25; 7 25 26; 8 26 27; 9 27 28;

10 28 24; 11 24 20; 12 20 16; 13 16 12; 14 12 8; 15 8 4; 16 26 22; 17 22 18;

18 18 14; 19 14 10; 20 10 6; 21 6 2; 22 27 23; 23 23 19; 24 19 15; 25 15 11;

26 11 7; 27 7 3; 28 21 22; 29 22 23; 30 23 24; 31 17 18; 32 18 19; 33 19 20;

34 13 14; 35 14 15; 36 15 16; 37 9 10; 38 10 11; 39 11 12; 40 5 6; 41 6 7;

42 7 8;

START USER TABLE

TABLE 1

UNIT METER KN

WIDE FLANGE

COLUMNUPTO2NDFLOOR

0.0216 0.824 0.012 0.5 0.012 0.00249018 0.000250115 1.0368e-006 0.009888 0.012

TABLE 2

UNIT METER KN

WIDE FLANGE

SEC1AND2

0.03568 0.824 0.02 0.5 0.02 0.0040359 0.000417189 4.75733e-006 0.01648 0.02

END

DEFINE MATERIAL START

ISOTROPIC STEEL

E 2.05e+008

POISSON 0.3

DENSITY 76.8195

ALPHA 1.2e-005

DAMP 0.03

END DEFINE MATERIAL

MEMBER PROPERTY INDIAN

1 TO 6 10 TO 27 TABLE ST I80012B50012

7 TO 9 29 40 TO 42 TABLE ST ISMB350

28 29 30 TO 39 TABLE TB ISWB600A WP 0.25 TH 0.012

CONSTANTS

BETA 90 MEMB 1 TO 6 10 TO 15

MATERIAL STEEL ALL

SUPPORTS

1 TO 4 FIXED

DEFINE 1893 LOAD

```
ZONE 0.16 RF 3 I 1 SS 2 ST 2 DM 3
SELFWEIGHT 1
JOINT WEIGHT
5 TO 24 WEIGHT 144
25 TO 28 WEIGHT 72
MEMBER WEIGHT
28 TO 42 UNI 33
LOAD 1 LOADTYPE Seismic  TITLE SEISMIC
SELFWEIGHT X 1 LIST 1 TO 42
SELFWEIGHT Y 1 LIST 1 TO 42
MEMBER LOAD
28 TO 42 UNI GX -33
28 TO 42 UNI GY -33
7 TO 9 28 TO 42 UNI GX -9
7 TO 9 28 TO 42 UNI GY -9
SPECTRUM SRSS 1893 X 1 ACC SCALE 0.024 DAMP 0.03
SOIL TYPE 2
LOAD 2 LOADTYPE Dead  TITLE DL
SELFWEIGHT Y -1 LIST 1 TO 42
MEMBER LOAD
28 TO 42 UNI GY -33
LOAD 3 LOADTYPE Live  TITLE LL
MEMBER LOAD
7 TO 9 28 TO 42 UNI GY -18
LOAD COMB 4 GENERATED INDIAN CODE STEEL_PLASTIC 1
2 1.7 3 1.7
LOAD COMB 5 GENERATED INDIAN CODE STEEL_PLASTIC 2
```

```
2 1.7 1 1.7
LOAD COMB 6 GENERATED INDIAN CODE STEEL_PLASTIC 3
2 1.7 1 -1.7
LOAD COMB 7 GENERATED INDIAN CODE STEEL_PLASTIC 4
2 1.3 3 1.3 1 1.3
LOAD COMB 8 GENERATED INDIAN CODE STEEL_PLASTIC 5
2 1.3 3 1.3 1 -1.3
LOAD COMB 9 GENERATED INDIAN CODE STEEL_PLASTIC 6
2 1.7
LOAD COMB 10 GENERATED INDIAN CODE STEEL_PLASTIC 7
2 1.3 3 1.3
PERFORM ANALYSIS PRINT ALL
PERFORM ANALYSIS PRINT ALL
PARAMETER 1
CODE IS800 LSD
CHECK CODE ALL
PARAMETER 2
CODE IS800 LSD
STEEL MEMBER TAKE OFF LIST 1 TO 42
PARAMETER 3
CODE IS800 LSD
SELECT ALL
PARAMETER 4
CODE IS800 LSD
STEEL TAKE OFF LIST 1 TO 42
FINISH
```

CHAPTER-9

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