IJCRT.ORG ISSN : 2320-2882

INTERNATIONAL JOURNAL OF CREATIVE RESEARCH THOUGHTS (IJCRT)

An International Open Access, Peer-reviewed, Refereed Journal

SEISMIC ANALYSIS AND DESIGN OF MULTI STORIED BUILDING IN STEEL STRCUTURE USING IS 800 DRAFT CODE

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1. INTRODUCTION

Seismic Analysis is a subset of structural analysis and is the calculation of the response o f a building structure to earthquakes. It is part of the process of structural design, earthquake engineeri ng 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 plate s which cover the surface of the globe. These plates tend to move relative to one another but are prev ented by doing so by friction until the stresses between plates under the epicenter point become so hi gh that a move suddenly takes place. This is an earthquake. The local shock generates waves in the g round which propagate over the earth‟s surface, creating movement at the bases of structures. The im portance 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 mai n tectonic plates

Besides the major earthquakes which take place at tectonic plate boundaries, others have thei r 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 an d vertical components. The horizontal movement is the most specific feature of earthquake action bec ause of its strength and because structures are generally better designed to resist gravity than horizont al 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. Experienc eshows that steel structures subjected to earthquakes behave well. Global failures and hugenumbers o f casualties are mostly associated with structures made from other materials. This may be explained b y 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 stresse
- 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 stru cture‟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 structur e are intentionally designed to undergo cyclic plastic deformations without failure, and the structure a s 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.

2.1. 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 a nalysis 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%

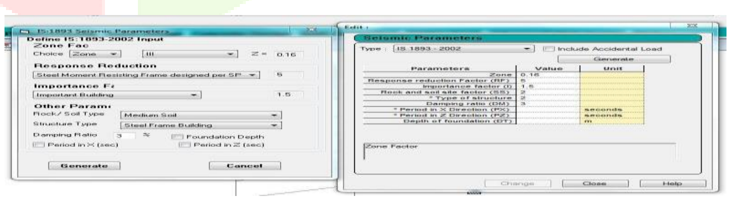


FIG 2.1 : STAAD input of seismic parameters

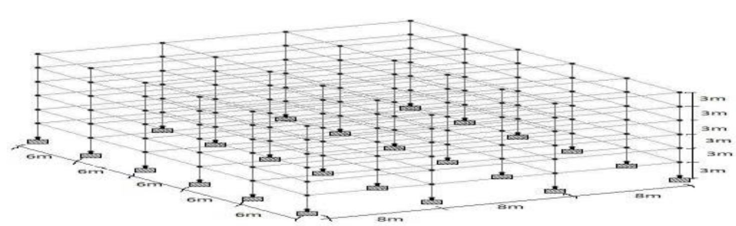
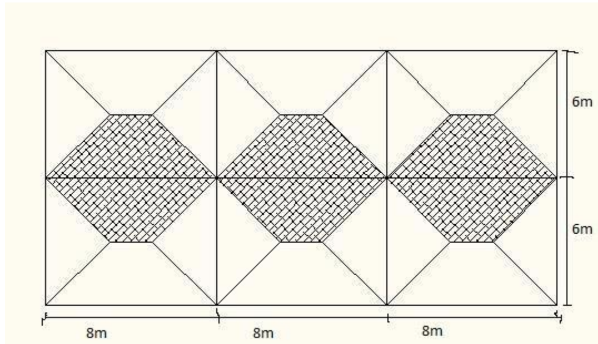


FIG 2.2 : 3-dimensional view of the steel building frame

LOAD PARAMETERS:

dead load is taken as = 5KN/m^2 and live load is taken as 3 KN/m^{2}Loa
d Calculatio



4.1 LATERAL FORCE METHOD:

The seismic load of each floor is calculated at its full dead load and imposed load. The weight of col umns and walls in any story should be appropriately divided to the floors above and below the story. Buildings designed for the storage purposes are likely to have large percentages of service load prese nt 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 man ner, 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 dyna mic 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

Table 4.1 : Analysis by lateral force method

Storey no. i	Absolute displacement of storey D_i (m)	Design inter storey drift D_r(m)	Storey lateral force V_{tot} (KN)	Shear at store yP_{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 sp ectrum diagram is used to perform it.The response spectrum method uses the idealization of a multi st orey shear building by a basic assumption. The assumption used is that the mass is lumped at the roo f diaphragm levels and at the floor levels. The diaphragms are assumed as infinitely rigid and the col umn axially inextensible but laterally flexible. The dynamic response of the spectrum is represented i n the form of lateral displacements of the lumped mass with the degrees of dynamic freedom(or mod es of vibration n) being equal to the number of masses. The undamped analysis of the building can b e 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 ob tained. The distribution of mass and the stiffness of the building determine the mode shapes.

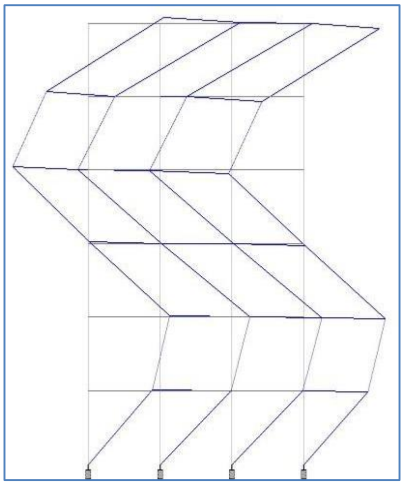
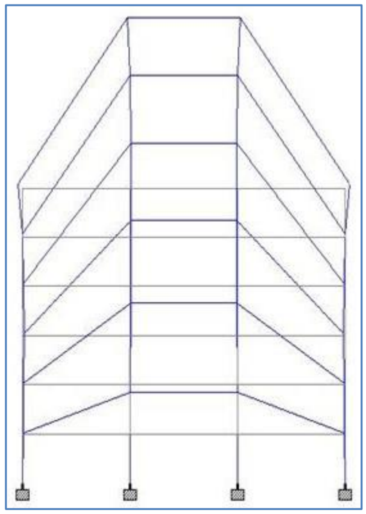
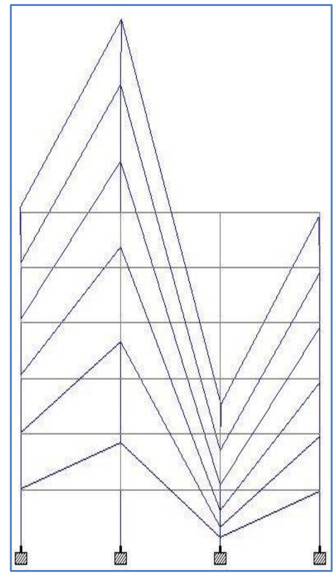
As the ground motion is applied at the base of the multi mass system, the deflected shape is but a combination of all mode shapes, which otherwise can be obtained by superposition of t he vibrations of each individual lumped mass. A modal analysis procedure is utilized in determining th e dynamic response 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 ow n 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 da mping ratio does not exceed 5%.The CQC is a method which can account for modal coupling method s 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 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

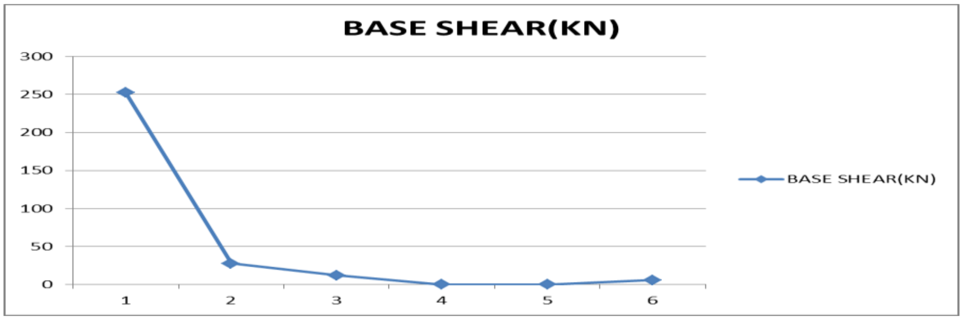
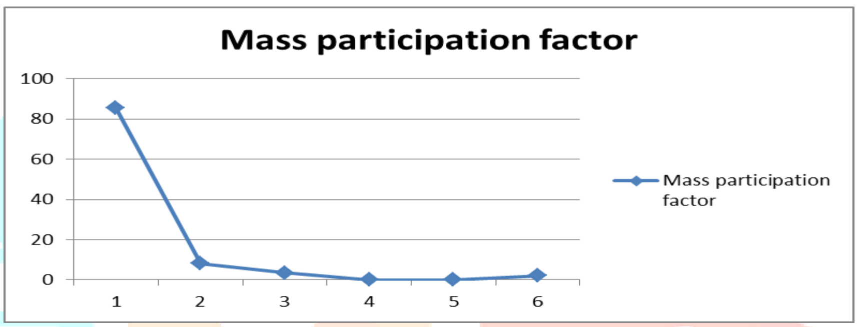


Fig (4.2) graph of modes Vs base shear



Fig(4.3) Graph of mass participation factor

<u>P-Δ ANALYSIS:</u>

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 $M_s = P\Delta$. Which is resisted by an additional shear force $\frac{P\Delta}{L}$ in the column. This secondary moment $M_s = P\Delta$ 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.
- $\Delta_{tx} = \Delta_x + \Delta_x\theta_x + \Delta\ \theta^2 + \ldots\ldots$
- Which is equal to $\simeq \Delta_x\left(\frac{1}{1-\theta x}\right)$ 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 store y D_i (m)	Design inter storeydri ft D_r(m)	Storey lateral forces	Shear at storey V_{tot}(KN)	Total cumulative gravity lo ad 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 store y D_i (m)	Design inter storeydr ift D_r(m)	Storey lateral forces	Shear at storey V_{tot}(KN)	Total cumulative gravity l oad at st orey 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

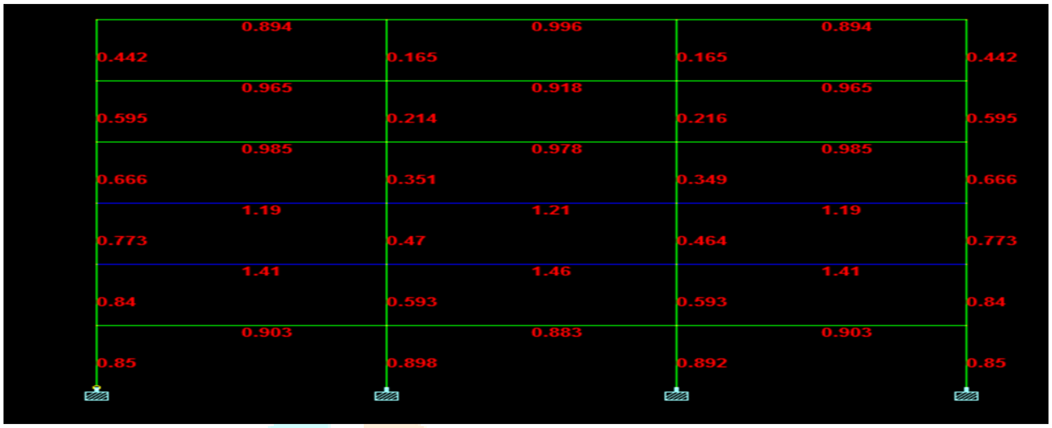
*Beams in this storey failed to satisfy P-Δ effect
From the above table checks are made on the limitation of P-Δ effects with the results from the **lateral force method**. The value of resultant base shear is: 179.201KN

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 : 230.172KN mWith the 1/(1-

Θ) increase: $1.11164 \times 230.172 = 255.868$ KNm

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



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

5.1 CONNECTION DESIGN

DESIGN STRENGTH

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. 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.

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 gyrat ion, r

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

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

Υ_{m0} =partial safety factor for material strength. α=Imperfec tion factor from Table 7

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

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

Tdg= Agfy /Υ 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 meth od)

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 an alysis)

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

CONNECTION OF BEAM TO THE STEEL-PLATE:

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

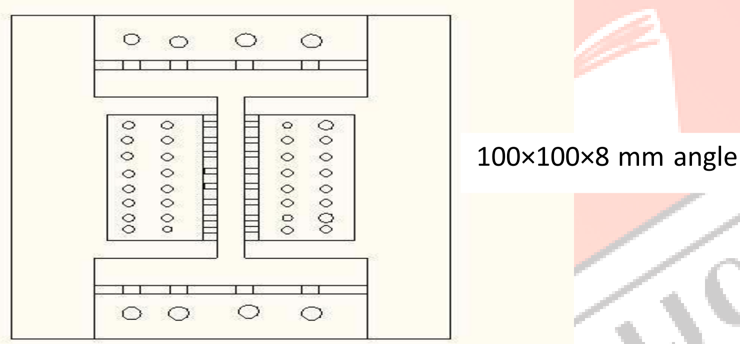
$$n = \sqrt{\frac{6\times M}{m\times p\times R}} \text{ (no. of bolts required)} = \sqrt{\frac{6\times 278.682}{4\times 0.06\times 108.915}} = 7.997 \approx 8 \text{ bolts}$$

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



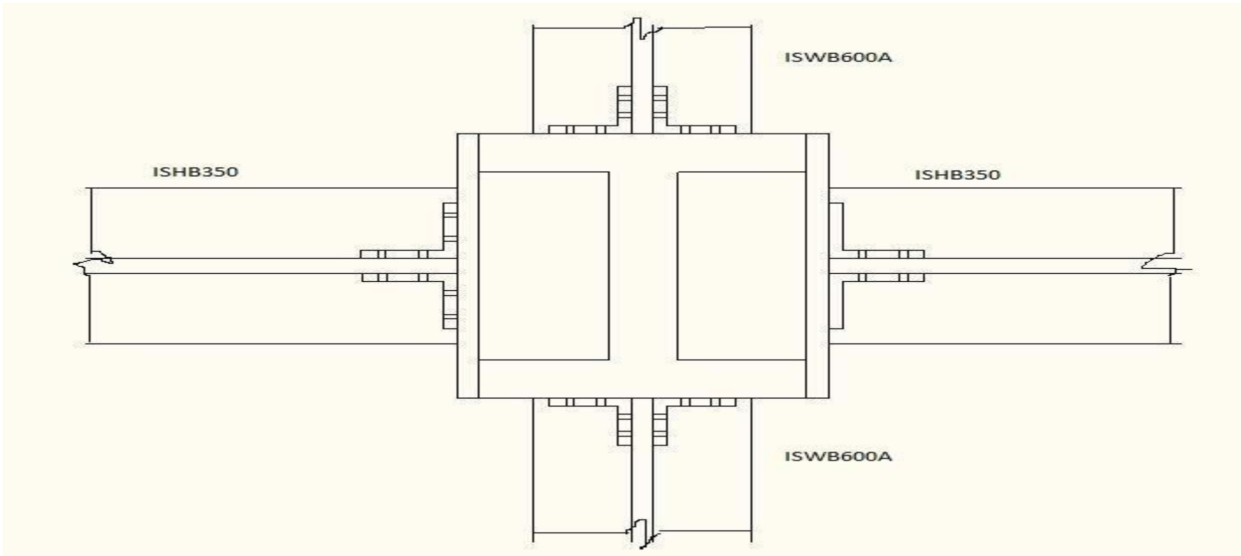


Fig 5.4: Top view of interior joint

<u>UNSTIFFENED SEAT CONNECTION:</u>

Assume 2 angle sections ISA 150×115×8

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

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

But bearing length $a \geq \frac{R}{2\times tw\times\sigma p} = \frac{120\times10^3}{2\times11.8\times187.5} = 27.11mm$

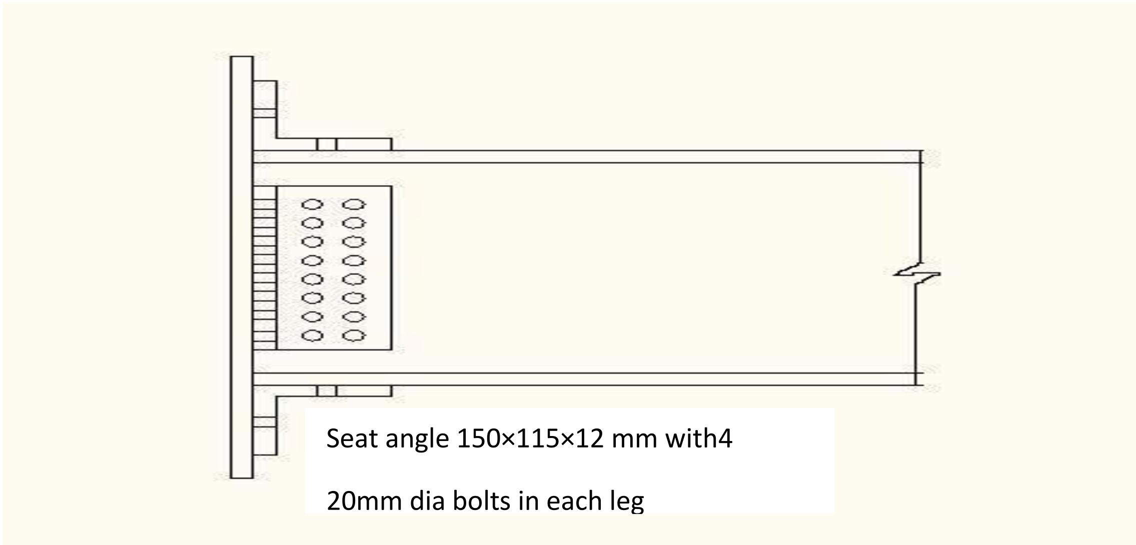


Fig 5.5: Unstiffened seat connection

Check for thickness

Max bending moment at critical section ;;

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

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

Equating bending moment and moment of resistance

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

$t=10.69\text{mm} \approx 12\text{mm}$

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 $\frac{P}{b \times l} \pm \frac{6M}{b \times l2}$

$\sigma_{bs} = 185$Mpa

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

Therefore bolt value= 51.05KN

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

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= $\frac{1}{2} \times \frac{1963.83}{2} + 582.63 = 1073.59$KN

No. of bolts= $\frac{1073.59}{51.07} = 21.02 \approx 22$ 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, $\frac{M}{P} = \frac{480093}{1963.83} = 244.46$mm > $\frac{1150}{6} = \frac{l}{6}$

Therefore the stress diagram will be

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

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

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

Assume = **1150×980mm base plate**

Thickness of base plate:

Bending moment at section Z-Z

$Mz = 5.33 \times (151 \times 980) \times \frac{151}{2} + \frac{8 - 5.33}{2} \times (151 \times 980) \times \frac{2}{3} \times 151 = 79436403.9$

Equating bending moment to the moment of resistance

$(\frac{1}{6}) \times 980 \times t^2 \times 185 = 79436403.9$

T= 51.27mm

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

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 tha

n 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

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