SEISMIC ANALYSIS AND DESIGN OF MULTI STORIED BUILDING IN STEEL STRUCTURE 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 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 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 diagram. 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 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
LOAD PARAMETERS:

dead load is taken as = 5KN/m² and live load is taken as 3 KN/m². 

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 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 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 $V_B = A_b \times W$

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.

**Table 4.1: Analysis by lateral force method**

<table>
<thead>
<tr>
<th>Storey no.</th>
<th>$D_i$ (m)</th>
<th>$D_f$ (m)</th>
<th>$V_{tot}$ (KN)</th>
<th>$Y_{P_{tot}}$ (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.003869</td>
<td>0.003869</td>
<td>1.969</td>
<td>179.201</td>
</tr>
<tr>
<td>2</td>
<td>0.012595</td>
<td>0.008726</td>
<td>7.951</td>
<td>177.232</td>
</tr>
<tr>
<td>3</td>
<td>0.023837</td>
<td>0.011242</td>
<td>17.83</td>
<td>169.281</td>
</tr>
<tr>
<td>4</td>
<td>0.035892</td>
<td>0.012055</td>
<td>31.657</td>
<td>151.451</td>
</tr>
<tr>
<td>5</td>
<td>0.047566</td>
<td>0.011674</td>
<td>49.212</td>
<td>119.794</td>
</tr>
<tr>
<td>6</td>
<td>0.058123</td>
<td>0.010557</td>
<td>70.582</td>
<td>70.582</td>
</tr>
</tbody>
</table>
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 story 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 diaphragms 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>
<thead>
<tr>
<th>Storey no.</th>
<th>Absolute displacement of storey Δi (m)</th>
<th>Design inter storey drift Δi (m)</th>
<th>Storey lateral force Vtot (KN)</th>
<th>Shear at storey Ptot (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.00491</td>
<td>0.00491</td>
<td>1.877</td>
<td>120.981</td>
</tr>
<tr>
<td>2</td>
<td>0.0115</td>
<td>0.0066</td>
<td>6.112</td>
<td>119.104</td>
</tr>
<tr>
<td>3</td>
<td>0.0161</td>
<td>0.0046</td>
<td>10.651</td>
<td>112.992</td>
</tr>
<tr>
<td>4</td>
<td>0.0196</td>
<td>0.0035</td>
<td>17.331</td>
<td>102.341</td>
</tr>
<tr>
<td>5</td>
<td>0.0219</td>
<td>0.0023</td>
<td>29.98</td>
<td>85.01</td>
</tr>
<tr>
<td>6</td>
<td>0.0234</td>
<td>0.0015</td>
<td>55.03</td>
<td>55.03</td>
</tr>
</tbody>
</table>
**Table 4.3: Base shear and mass participation factor**

<table>
<thead>
<tr>
<th>MODE</th>
<th>BASE SHEAR (KN)</th>
<th>Mass participation factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>252.75</td>
<td>85.33</td>
</tr>
<tr>
<td>2</td>
<td>27.8</td>
<td>8.13</td>
</tr>
<tr>
<td>3</td>
<td>12.1</td>
<td>3.54</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>5</td>
<td>0.02</td>
<td>0.01</td>
</tr>
<tr>
<td>6</td>
<td>5.85</td>
<td>2.04</td>
</tr>
</tbody>
</table>
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 $\frac{PΔ}{L}$ 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 $\Delta x$, 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 $\Delta x$ i.e.
  $$\Delta x = \Delta x + \Delta \theta_x + \Delta \theta_x^2 + \ldots$$
  which is equal to $\Delta x(1) \Rightarrow \Delta x = M' = P_x \Delta x$
- $M_x''$ is the secondary moment
- $P_x =$ total weight (DL+LL) at level X & above
- $\Delta_x =$ drift of storey X

Fig (4.2) graph of modes Vs base shear

Fig(4.3) Graph of mass participation factor
• \( V_x = \) shear force of storey X
• \( H_x = \) height of storey X
• The code (uniform building code UBC) stipulates that the P-\( \Delta \) 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)

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

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

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

*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)
θ (1.11 at storey 2 and 1.12 at storey 3)

The maximum bending moment is at storey 2: 230.172KN
Θ) increase: 1.1164×230.172=255.868KNm
Beams are ISMB350: M=Z×f_y=877×250=219.25KNm And 219.25<230.172

![Image](image-url)

**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 P_d of a member is given by:

\[ P < P_d \quad \text{where} \quad P_d = A_e \times f_{c_d} \]

where

\[ A_e = \text{effective sectional area as defined in 7.3.2, and} \]
\[ f_{c_d} = \text{design compressive stress, obtained as per 7.1.2.1.} \]

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

\[ f_{c_d} = \left( \frac{f_y}{y_{mo}} \right) \left( \sqrt{\lambda^2} - 0.5^2 \right) \left( \frac{K_{L/r}}{E} \right) \]

\[ \lambda = \text{non-dimensional effective slenderness ratio} \]
\[ f_{c_d} = \sqrt{f_y} \times \left( \frac{K_{L/r}}{E} \right) \]

where:

- \( f_y \) = Euler buckling stress = \[ \frac{E}{K_{L/r}} \]
- \( KL/r \) = effective slenderness ratio or ratio of effective length, KL to appropriate radius of gyration
\[ \chi = \text{stress reduction factor (see Table 8) for different buckling class, slenderness ratio and yield stress} \\
= 1/(\phi + [\phi^2 - \lambda^2]^{0.5}) \]

\[ \gamma_{m0} = \text{partial safety factor for material strength, } \alpha = \text{Imperfection factor from Table 7} \]

**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

\[ T_{dg} = \frac{A_f}{\gamma_{m0}} \]

where

\( f_y = \text{yield stress of the material,} \)

\( A_f = \text{gross area of cross-section, and} \)

\( \gamma_{m0} = \text{partial safety factor for failure in tension by yielding (see Table 5).} \)

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

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

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

<table>
<thead>
<tr>
<th>Sl no.</th>
<th>Failed member no:</th>
<th>Failed section</th>
<th>Critical condition</th>
<th>Staad design section(passed)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1,13</td>
<td>I80012B50012</td>
<td>IS 7.1.2</td>
<td>I80012B50016</td>
</tr>
<tr>
<td>2</td>
<td>2,14</td>
<td>I80012B50012</td>
<td>IS 7.1.2</td>
<td>I0012B55012</td>
</tr>
<tr>
<td>3</td>
<td>3,15</td>
<td>I80012B50012</td>
<td>IS 7.1.2</td>
<td>ISWB550</td>
</tr>
<tr>
<td>4</td>
<td>7,8,9,40,42</td>
<td>ISMB350</td>
<td>IS 6.2</td>
<td>I100012B50012</td>
</tr>
<tr>
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<td>IS 7.1.2</td>
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</tr>
<tr>
<td>6</td>
<td>27</td>
<td>I80012B50012</td>
<td>IS 7.1.2</td>
<td>ISWB600A</td>
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<tr>
<td>7</td>
<td>41</td>
<td>ISMB350</td>
<td>IS 6.2</td>
<td>ISMB600</td>
</tr>
</tbody>
</table>

**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 \approx 8 \text{ bolts} \\

R = \text{bolt value (area} \times \sigma_{tf}) \\
m = 4 \text{ lines, } M = 278.682 \text{KN, } W = 120 \text{KN} \\
\text{After calculation } n = 7.997 \approx 8 \text{ no.s bolts per line} 

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

100×100×8 mm angle
UNSTIFFENED SEAT CONNECTION:

Assume 2 angle sections ISA 150×115×8

Strength of bolts in single shear $\Rightarrow 300 \times (21.5)2 = 108.9\, KN$

Strength of bolts in bearing 12mm plate $= 30 \times 21.5 \times 12 = 77.4\, KN$

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

Bearing length $a = \frac{R}{tw\times \sigma_p}$

$h_2 = \frac{120 \times 10}{\sqrt{3}} = \frac{11.8 \times 18}{7.5} = 46.05 \times \sqrt{3} = 25.52$ (negative)

But bearing length $a \geq \frac{R}{2 \times tw \times \sigma_p} = \frac{120 \times 10}{\frac{11.8 \times 18}{7.5}} = 27.11\, mm$
**Check for thickness**

Max bending moment at critical section:

\[ R = \frac{120 \times 10^3 (12 + \frac{27.11}{2} - 8 - 11)}{2} = 7.86 \times 10^8 \text{ N mm} \]

At critical section, moment of resistance:

\[ \sigma \times t^2 = \frac{165 \times 250 \times t^2}{6} = 6875 t^2 \]

Equating bending moment and moment of resistance:

\[ 6875 t^2 = 7.86 \times 10^8, \ t = \frac{\sqrt[5]{78.6 \times 10^5}}{6875} = 10.69 \text{ mm} \]

\[ t = 10.69 \text{ mm} \approx 12 \text{ mm} \]

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

\[ \sigma_{bs} = \frac{P}{b \times l} \pm \frac{6M}{b \times l^2} \]

\[ \sigma_{bs} = 185 \text{Mpa} \]

In single shear = \[ \tau d \times d^2 = \frac{100}{4} \times 25.52 = 51.05 \text{KN} \]

Therefore bolt value = 51.05KN

Force due to moment = \[ \frac{\text{moment}}{\text{lever arm between flanges}} \]

\[ = \frac{48093}{824} = 58.23 \text{KN} \]
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 = \( \frac{1}{2} \times \frac{1963.83}{2} + 582.63 = 1073.59KN \)

No. of bolts = \( \frac{1073.59}{244.46} \approx 22 \) 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 > 1150 = \( \frac{l}{6} \) -

Therefore the stress diagram will be

\[ \sigma = \frac{P}{b} = \frac{P}{2} \]

\[ X = 502.62 \text{mm} \]

The extreme fibre stress \( \sigma \) 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 \( R = \frac{P \times X}{b} = \frac{P}{2} \)

\[ b = \frac{P \times 2}{2 \times 1963.83 \times 1000} = 976.796 \approx 980 \text{mm} \]

Assume = 1150 x 980 mm base plate

**Thickness of base plate:**

Bending moment at section Z-Z

\[ Mz = 5.33 \times (151 \times 980) \times \frac{151}{2} + \frac{5.33}{2} \times (151 \times 980) \times 2 \times 151 = 79436403.9 \]

Equating bending moment to the moment of resistance

\( \frac{1}{2} \times 980 \times r^2 \times 195 = 79436403.9 \)

\( r = 51.27 \text{mm} \)

Thickness of base plate = t - thickness of angle = 51.27 - 12 = 39.27 \( \approx 40 \text{mm} \)

**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 0.6
2803 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 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 (e.g.: ISMB 350) have failed and Staad Pro has redesigned and adopted higher section (e.g.: 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

REFERENCE

- Design Example by Arcelor-Mittal (www.arcelor-mittal.org)