Seismic Analysis and Design of Steel Framed Multi-Storied Building

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Abstract— Earthquakes are the natural hazard under which the disasters are mainly caused by damage to or collapse of building. So for new construction it is necessary to establish earthquake resistant regulations and their implementation is the critical safeguard against earthquake induced damage. Depending on the conditions different design base shear calculation format given in IS 1893:2002, Eurocode 8. These formats are applied to a multi-storied steel frame building and using Equivalent Static Analysis approach base shear are calculated and compared. According to IS 800:2007 Elastic Analysis and Design of steel structure are suggested. Eurocode 3 also has given limit state design approach for the design of steel structure. Design of multi-storied steel frame building has been carried out for elastic analysis as per IS 800 2007 and for plastic analysis as per IS 800 1984. Similarly the multi-storied steel frame building has been designed by Eurocode 3 and Eurocode 4 mentioned procedures. The design consists of design of beam, column, connection, gusseted base plate and seismic design of the beam-to-column joint panel zone. All the checks are shown according to requirement of seismic codes respectively.

Key words: steel frame building,

I. INTRODUCTION

Earthquakes are the natural hazard under which disaster are mainly caused by damage to or collapse of building and other man made structure. So its prevention for collapse of building or reduction in loss of life is also in our hand. The primary objective of earthquake resistant design is to prevent building collapse during earthquakes thus minimizing the risk of death or injury to people in or around those buildings. Because damaging earthquakes are rare, economics dictate that damage to buildings is expected and acceptable provided collapse is avoided. Experience has shown that for new construction; establish earthquake resistant regulation and their implementation is the critical safeguard against earthquake induced damage. As regards existing structure, it is necessary to evaluate and strengthen them based on evaluation criteria before an earthquake.

Urbanization has led to a dramatic increase in the number and variety of high-rise building structures. The seismic vulnerability of these high-rise infrastructures is poorly understood and probabilistic assessment tools of their performance is lacking.

A. Earthquake Design Philosophy

The engineers do not attempt to make earthquake proof buildings that will not get damaged even during the rare but strong earthquake, instead, the engineering intention is to make buildings earthquake resistant such buildings resist the effects of ground shaking, although they may get damaged severely but would not collapse during the strong earthquake. Thus, safety of people and contents is assured in earthquake-resistant buildings, and thereby a disaster is avoided. So, below the earthquake resistant design philosophy may be summarized in Table 1.

Ground Shaking	Structural Damage	Collapse	Non Structural Damage
Strong	A	NA	A
Moderate	L	NA	S
Minor	NA	NA	NA

Table 1: Earthquake Design Philosophies

A – Acceptable
L – Limited
S – Sustainable
NA – Not Allowed

a) Under minor but frequent shaking, the main member of the building that carry vertical and horizontal forces should not be damaged; however building parts that do not carry load may sustain repairable damage.
b) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake.
c) Under strong but rare shaking, the main member may sustain severe (even irreparable) damage, but the building should not collapse.

Thus, after minor shaking, the building will be fully operational within a short time and the repair costs will be small. And, after moderate shaking, the building will be operational once the repair and strengthening of the damaged main members is completed. But, after a strong earthquake, the building may become dysfunctional for further use, but will stand so that people can be evacuated and property recovered.

B. Design Methodology

Depending on the characteristic value of the material strength and usefulness the design are classified as follows.

C. Limit state design

For ensuring the design objectives, the design should be based on characteristic value of material strengths and applied loads (action), which take in to account the probability of variations in the material strengths and in the loads to be supported. The limit state method is based on any choose limit of structural usefulness. The limit can be attainment of yield point stress, brittle fracture, fatigue, instability, deflection or maximum plastic Strength. Thus the strength of member depends on the criteria of design adopted. The design values are derived from the characteristic values through the use of partial safety factors, both for material strengths and for loads. Reliability of design is ensuring by requiring that

Design action ≤ Design strength

Limit state are the states beyond which the structure has no longer satisfied the performance requirements specified. The limit states are classified as,

1) Limit state of strength

Limit state of strength are those associated with failure under the action of probable and most unfavorable combinations of load on the structure using the appropriate partial safety factor, which may endanger the safety of life and property.

2) Limit strength of serviceability

It includes deformation and deflection, which may adversely affect the appearance or effective use of structure. Vibration of the structure of any of its components causing discomfort to people, damage to the structure, its contents, of which may limits its structural effectiveness.

D. Plastic Design

Steel has unique physical property, ductility, because of which it is able to absorb large deformation beyond the elastic limit without fracture. Due to this property steel possesses a reserve of strength beyond its yield which tries to utilize in the plastic method of design. Plastic design is an aspect of limit design that extends the structural usefulness up to the plastic strength or Ultimate load carrying. In plastic ultimate strength is the criteria rather than yield stress hence behavior of member beyond the yield stress in the inelastic (or plastic) range is considered.

II. MODELLING AND ANAYSIS OF BUILDING

A. Overview

Building seismic codes have given different seismic format for the calculation of design seismic forces and different seismic force distribution along the height of the structure. These formats are discussed in this chapter. According to that these formats are applied to the multi-storey 3D frame and the seismic acceleration coefficients and design seismic forces are compared along the height of the frame.

B. Details of the Building

An example from Proposed Draft for IS 1893 (Part 1), explanatory examples of Provisions on Seismic Design of Buildings is considered[3].

The details of the building are shown below.

Four storey steel framed office building is considered. Building is located in Shilong (seismic zone V).The soil conditions are hard. The steel frames are infilled with brick masonry. The floors are catering for a live load of 3kN/m^2 on floors and 1.5 kN/m^2 on the roof.

C. Data for Load Calculation

Floor Finish (roof) = 2.5 kN/m
Floor Finish (floor) = 1 kN/m
Live Load (roof) = 1.5 kN/m
Live Load (floor) = 3 kN/m
Density of Concrete = 25 kN/m^3
Slab Thickness = 125 mm
Density of Brickwork = 20 kN/m^3

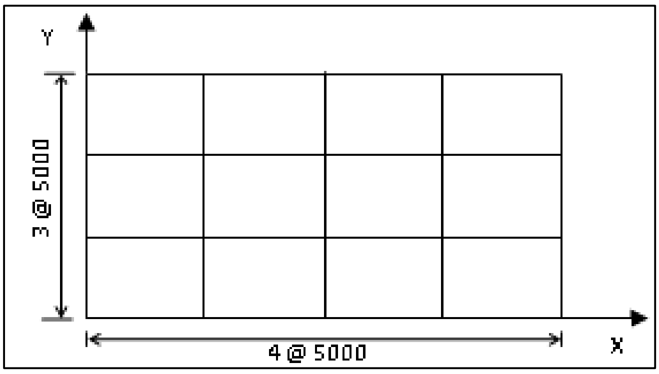

Fig. 1: Plan of multi storey building

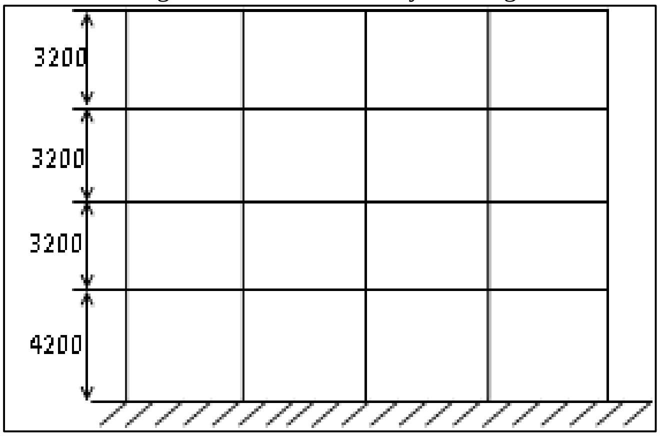

Fig. 2: Elevation of multi-storey building

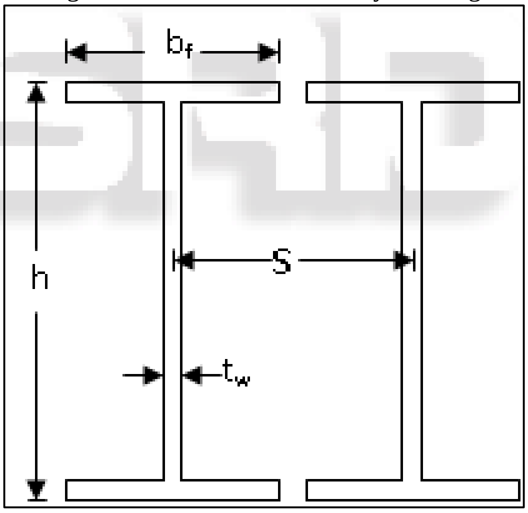

Fig. 3: Sections used for modeling the building

1) Columns

For limit state design ISHB 350
(Spacing) S = 300 mm
For plastic design ISWB 600
(Spacing) S = 400 mm

2) Beam

For limit state design ISMB 400
For plastic design ISMB 500

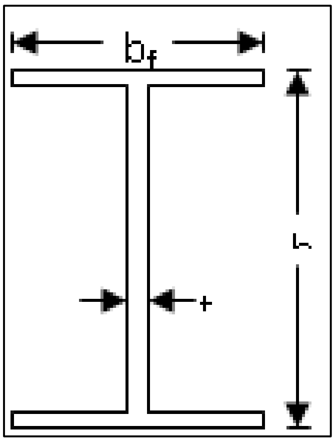

Fig. 4: Sections used for modeling the building

D. *Other Necessary Details*

1.Slab 120 mm thick	2.Self wt. of ISMB 500 = 0.8525 kN/m
3.Floor Finish 1 kN/m^2	4.Self wt. of ISMB 550 = 1.0173 kN/m
5.Live Load (Roof) 1.5 kN/m^2	6.Density of concrete 25 kN/m^3
7.Live Load (Floors) 3 kN/m^2	8.Density of concrete 20 kN/m^3

Table 2:

E. *Model Used For Analysis in STAAD Pro*

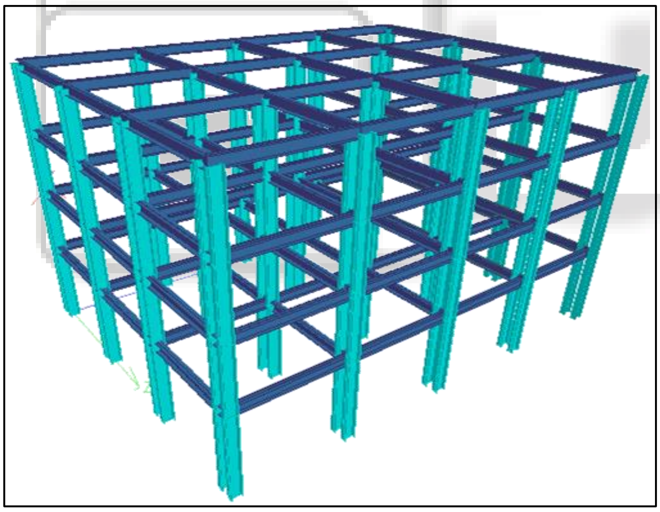

Fig. 5:

F. *Modeling Steps Used For Analysis in STAAD Pro*

Model is prepared in STAAD Pro software. The building is three dimensional multi-storied structure with four bays in X direction and three bays in Y direction. The building is with different column height i.e. 4.2 m for ground level and 3.2 m for rest above storeys. Steel sections are used to model the building. Internal column is provided the higher section as compared to the other columns. Beams are provided same section for the whole building. Dead load, live load and earthquake loads are applied. Earthquake loads are calculated using Equivalent Static Analysis and applied directly as the nodal forces on each node of each storey by dividing the storey shear with the number of nodes of each storey. Then analysis is performed and the maximum forces are obtained from summary and designed for various International codes.

G. *Seismic Weight Calculations*

Seismic weight of the frame of Figure is calculated

Level	Slab +_ FF	Parapet Wall	Brick Wall	LL	Beam	Column	Total Load (kN)
Roof	(0.12×15×20×25) = 900	(1×70×0.115×20) = 161	(155×20×0.23×1.6) = 1141	(1.5×0.25×15×20) = 112.5	(31×5×0.85×25) = 132	(20×1.6×2×1.0173) = 33	2896.92
Third	900	-	(155×20×0.23×3.2) = 2282	(3×0.25×15×20) = 225	132	65	3452.84
Second	900	-	2282	225	132	65	3452.84
First	900	-	(155×20×0.23×3.7) = 2638	225	132	75	3723.06
Total seismic weight of building							13526

Table 3: Seismic Weight Calculation

Equivalent static analysis (As per IS1893:2002)

Seismic details for IS1893:2002

Importance Factor : 1.0
Response reduction Factor : 5.0
Zone : V
Type of Soil : Hard
Damping in Steel Structure : 2%

Table 4: Seismic details

H. *Design base shear*

The horizontal acceleration coefficient is given by,

$$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}$$

Consider a panel with brick infilled walls the time period is calculated as,

$$T = \frac{0.09h}{\sqrt{d}} = \frac{0.09 \times 13.8}{\sqrt{20}} = 0.2777 \quad \text{sec}$$

For the calculated time period the spectral acceleration coefficient is, for 2% damping multiplying factor=1.4

$$\frac{S_a}{g} = 2.5 \times 1.4 = 3.5$$

Therefore the horizontal acceleration coefficient is,

$$A_h = \frac{0.36}{2} \times \frac{1}{5} \times 3.5 = 0.126$$

Design base shear

$$V_B = A_h \times W$$

$$V_B = 0.126 \times 13526$$

$$V_B = 1704 \quad \text{kN}$$

I. *Distribution along the Height*

Distribution of base shear along the storey height for IS1893:2002(Part1) is given as follows

Distribution of forces along the height of building

Level	Storey Wt. (kN)	Height (m)	$W_ih_i^2$	$\frac{w_ih_i^2}{\sum w_ih_i^2}$	Storey Shear (Q_i) (kN)
Roof	2896.92	13.8	551689.44	0.46	787
Third	3452.84	10.6	387961.1	0.32	554
Second	3452.84	7.4	189077.52	0.16	270
First	3723.06	4.2	65674.778	0.05	94
		$\sum$	1194402.8	$\sum$	1704

Table 5: Design Guidelines given by Eurocode-8

J. *Importance Classes for Buildings*

In Eurocode-8 all categories of buildings are divided into four groups according to its purpose and function. For each seismic use group a seismic factor or importance factor is given. In IS1893-2002 (Part 1) also the importance factors are given for buildings according to function and importance. The importance classes and importance factor as given by Eurocode-8 is presented.

K. *Importance Classes of Building*

Importance Class	Building	Seismic factor(γI)
I	Building of minor importance for public safety ,e.g. agricultural building	0.8
II	Ordinary building not belonging in the other categories.	1.0
III	Building whose seismic resistance is of importance in view of the consequences associated with the collapse ,e.g. school .assembly hall, Cultural institutions etc.	1.2
IV	Building whose integrity during earthquake is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.	1.4

Table 6:

L. *Lateral Force Method of Analysis*

1) *Limitations*

The requirement is deemed to be satisfied in a building which fulfils both of the two following conditions.

a) They have fundamental periods of vibration *T*1 in the two main directions which are smaller than the following values

$$T_1 = \leq \begin{cases} 4T_C \\ 2.0s \end{cases}$$

Where,

TC is given in Table 3 or Table5 of Eurocode-8;

b) They meet the criteria for regularity in elevation given in 4.2.3.3 of Eurocode-8.

Base shear force

The seismic base shear force ‘F_b’, for each horizontal direction in which the building is analyzed, shall be determined using the following expression

$$F_b = S_d(T_1)m\lambda$$

Where,

Sd (T_1) = the ordinate of the design spectrum (see 3.2.2.5EC-8) at period T_1;

m = the total mass of the building, above the foundation or above the top of a rigid basement.

λ = the correction factor, the value of which is λ = 0.85 if $T_1 < 2\ T_C$ and the building has more than two storey’s, or λ = 1.0 otherwise

T_1 = the fundamental period of vibration of the building for lateral motion in the direction considered.

For buildings with heights of up to 40 m the value of T_1 (in Sec) may be approximated by the following expression

$$T_1 = C_t H^{3/4}$$

Where, C*t* = 0.085 for moment resistant space steel frames, 0.075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0.050 for all other structures.

H = the height of the building, in m, from the foundation or from the top of a rigid basement.

The value of the damping correction factor η may be determined by the expression

$$\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55$$

Where,

ξ = the viscous damping ratio of the structure, expressed as a percentage which is 2% for steel structure.

2) *Distribution of the horizontal seismic forces*

The seismic action effects shall be determined by applying, to the two planar models, horizontal forces *Fi* to all storey.

$$F_i = F_b \frac{Z_i m_i}{\sum Z_i m_i}$$

Where,

Z_i, Z_j = the heights of the masses m_i, m_j above the level of application of the seismic action (foundation or top of a rigid basement)

Equivalent static analysis (EC-8)

Seismic details for EC-8

Seismic details

Importance Factor :	1.0 (Class II)
Behavior factor :	6.0
PGA :	0.36g
Type of Soil :	Type A (Rock)
Damping in Steel Structure :	2%

Table 7:

M. *Design base shear calculation*

The seismic base shear force *Fb*, is given as follows:

$$F_b = S_d(T_1) m \lambda$$

From equation above,

$$T_1 = 0.05 \times 13.8^{3/4}$$

$$T_1 = 0.358 \text{ sec}$$

Sd (T1) can be determined from the design response spectrum (3.2.2.5) of EC-8. From the Table 7 (EC-8) the parameter governing the recommended elastic response spectra (Refer Table 8)

Ground Type	S	T_B (sec)	T_C (sec)	T_C (sec)
A	1.00	0.15	0.40	2.00
B	1.20	0.15	0.50	2.00
C	1.15	0.20	0.60	2.00
D	1.35	0.20	0.80	2.00
E	1.40	0.15	0.50	2.00

Table 8: Value of parameter describing elastic response spectra

From design response spectrum for ground type A

$$T_B \le T \le T_C : S_d(T) = a_g \times S \times \frac{2.5}{q}$$

According to clause 3.2.2.5(Eurocode-8)The values of the behavior factor *q*, which also account for the influence of the viscous damping being different from 5%, are given for various materials and structural systems according to the relevant ductility classes.

$$\frac{S_d(T)}{g} = 0.36 \times 1 \times \frac{2.5}{5}$$

$$\frac{S_d(T)}{g} = 0.18$$

Now, the seismic base shear force F_b, is given by

$$F_b = S_d(T) \times m \times \lambda$$

$$F_b = \frac{S_d(T)}{g} \times m \times g \times \lambda$$

$$F_b = S_d(T) \times W \times \lambda$$

$$F_b = 0.18 \times 13526 \times 0.85$$

$$\lambda = 0.85$$

$$F_b = 2069 \; kN$$

Also, Horizontal Seismic Coefficient = $\frac{m}{F_b}$

$$\frac{m}{F_b} = \frac{2069}{13526}$$

$$\text{Ah} = \frac{m}{F_b} = 0.153$$

N. Distribution along The Height

Distribution of base shear along the storey height for Eurocode-8 is given as follows
(Refer Table 9)

FLOOR	*W* (kN)	*h* (m)	$z_i m_i$	$\frac{z_i \times m_i}{\sum z_i \times m}$	Storey Shear (kN)
4th	2896.92	13.8	39977.5	0.34	703
3rd	3452.84	10.6	36600.1	0.31	643
2nd	3452.84	7.4	25551.02	0.22	443
1st	3723.06	4.2	15636.85	0.13	275
		$\sum$	117765.5	$\sum$	2069

Table 9: Distribution of forces along the height of building.

O. Comparison of Horizontal Seismic Acceleration and Seismic Base Shear

The calculated seismic acceleration coefficient and seismic base shear in sections 3.3.4, 3.3.5 and 3.3.6 are as shown in Table 10.

P. Comparison of Acceleration and Base Shear

		IS 1893:2002	Euro Code 8 2003
Natural Period		0.2777 sec	0.358 sec
Horizontal Seismic Coefficient		0.126	0.153
Design Base Shear		1704 kN	2069 KN
Storey Shear	4th	787 kN	703 kN
	3rd	554 kN	643 kN
	2nd	270 kN	443 kN
	1st	94 kN	275 kN

Table 10:

Q. Design of Panel Zone

1) Loads on panel zone

From column

Top:-

Bottom:- Axial force N_{c1} = 326 kN

Axial force N_{c2} = 764 kN

Moment M_{c1} = 89 kN-m

Moment M_{c2} = 190 kN-m

Shear force V_{c1} = 95 kN

Shear force V_{c2} = 143 kN

From beam

Left:-

Right:-

Moment M_{bl} = 215 kN-m

Moment M_{br} = 215 kN-m

Shear force V_{bl} = 133 kN

Shear force V_{br} = 133 kN

2) Properties of Section

Properties of column 2 ISHB450 and beam ISMB 450 are,

Overall depth of column section d_c = 450 mm

Overall depth of beam section d_b = 450 mm

Thickness of column flange t_{cf} = 13.7 mm

Thickness of beam flange t_{bf} = 13.7 mm

Moment of inertia of column section I_{xx} = 740437000 mm^4

Area of column section A = 22228 mm^2

Average of storey height H = 3200 mm

3) Calculation of Axial Force and Resisting Moments

Axial force in column

$$N = \frac{764 - 326}{2} = 438 \text{ kN}$$

Squash load

$$N_y = \frac{250 \times 22228}{1000} = 5557 \text{ kN}$$

Summation of resisting moments in beam

$$\sum M_{R,b} = 215 + 215 = 430 \text{ kN-m}$$

4) Shear Force in the Columns

The shear force in the column evaluated by assuming that the zero moment point is located in the middle section of columns is,

$$V_c = \frac{\sum M_c}{H - d_b} = \frac{89 + 190}{3200 - 450} = 101455 \text{ N}$$

5) Shear Force in Panel Zone

$$V_p = \frac{\sum M_b}{(d_b - t_{bf})} - V_c$$

$$V_p = \frac{(215 + 215) \times 10^6}{(450 - 13.7)} - 101455 = 884106 \text{ N}$$

6) Thickness of Panel Zone

Under the assumption that the shear stress is uniformly distributed in the panel zone, the shear stress develop with in the panel zone is given by,

$$\tau_p = \frac{V_p}{(d_c - 2t_{cf})t_p}$$

The average shear stress in the panel zone is given by

$$\tau_p = \frac{\sum M_b}{t_p(d_c - 2t_{cf})(d_b - t_{bf})}\left(1 - \frac{d_b - t_{bf}}{H - d_b}\right)$$

In order to assume that yielding occurs in the beam rather than the panel zone according to the von Mises yield criterion, the following condition has to be satisfied.

$$\frac{\sum M_{R,b}}{t_p(d_c - 2t_{cf})(d_b - t_{bf})}\left(1 - \frac{d_b - t_{bf}}{H - d_b}\right) \le \tau_y \left(1 - \left(\frac{N}{N_y}\right)^2\right)^{\frac{1}{2}}$$

This provides the design value t_p of the panel zone thickness, including doublers plates,

$$t_p \ge \frac{1}{\tau_y \left(1 - \left(\frac{N}{N_y}\right)^2\right)^{\frac{1}{2}}} \frac{\sum M_{R,b}}{(d_c - 2t_{cf})(d_b - t_{bf})}\left(1 - \frac{d_b - t_{bf}}{H - d_b}\right)$$

$$t_p \ge \frac{1}{144 \times \left(1 - \left(\frac{438}{5557}\right)^2\right)^{\frac{1}{2}}} \frac{884106}{[450 - (2 \times 13.7)]} = 14.54 \text{ mm}$$

Required thickness of panel zone is 14.54 mm; therefore the actual thickness is more and hence safe.

III. DESIGN USING INTERNATIONAL CODE

A. Introduction to IS 800:2007

In this chapter the frame is analyzed for the seismic forces calculated from the equivalent static analysis using IS 1893 2002 (Part 1).Bending moment and shear forces are calculated using STAAD Pro-2006 as per the IS 800 2007 load combinations. Then the design of steel frame is carried out according to IS 800 2007.

B. General Design Procedure

a) According to section steps for the design of beam and column are as follows
b) Determine possible loading conditions
c) Compute the factored design load combinations
d) Member subjected to bending shall be checked for
   - Class of the section
   - Adequacy of section
   - Shear strength of section
   - Design capacity of section
   - Check for deflection
e) Member subjected to combines axial force and bending moment shall be checked for
   - Check for bi-axial bending
   - Check for buckling resistance

C. Load Combinations

Load combinations according to the Eurocode recommendations are as follows

1) 1.5(DL+LL)
2) 1.2(DL+LL+EQX)
3) 1.2(DL+LL-EQX)
4) 1.2(DL+LL+EQY)
5) 1.2(DL+LL-EQY)
6) 1.5(DL+EQX)
7) 1.5(DL-EQX)
8) 1.5(DL+EQY)
9) 1.5(DL-EQY)
10) 0.9DL+1.5EQX
11) 0.9DL-1.5EQX
12) 0.9DL+1.5EQY
13) 0.9DL-1.5EQY

D. Analysis Results

The building is analyzed in STAAD Pro2006 for the dead, live and seismic loads and the results of maximum axial forces, bending moment and shear forces are used for the design of members.

IV. COMPARATIVE STUDY

The comparative study is carried out from the seismic design of steel frame by using IS1893:2002 and Eurocode-8. The governing parameters and results for seismic design are shown in Table 11.

Sr. No.	Description	IS1893:2002	EUROCODE-8
1	Horizontal acceleration coefficient	$A_h = \frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}$	$F_b = \frac{S_d(T_1)}{g} \times \lambda$
2	Design base shear(kN)	$V_B = A_h \times W$	$F_b = S_d(T_1) m \lambda$
		1704 kN	2069 kN
3	Base shear distribution along the building height	$\frac{w_i h_i^2}{\sum w_i h_i^2}$	$F_i = F_b \frac{Z_i m_i}{\sum Z_i m_i}$

4	Storey Shear	4th	787 kN	703 kN
		3rd	554 kN	643 kN
		2nd	270 kN	443 kN
		1st	94 kN	275 kN
5	Time period of building (Sec)		$T = \frac{0.09h}{\sqrt{d}}$	$T_1 = C_t H^{3/4}$
			0.277 sec	0.358 sec
6	Damping Factor(η)		η = 1 for 5% viscous damping η = 1.4 for 2%	$\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55$
7	Zone factor		Zone V (Z=0.36)	PGA=0.36g
8	Response reduction factor		R=5(Special moment resisting frame)	q=f(material, structural system) q=5αu/α1=6(SMR Steel frame)
9	Importance factor(I)		Important building(I)=1.5 Other building(I)=1.0	Class I=0.8 Class II=1.0 Class III=1.2 Class IV=1.4
10	Beam-column Strength ratio		$\frac{\sum M_C}{\sum M_B} > 1.2$	$\frac{\sum M_{rc}}{\sum M_{rb}} \geq 1.3$
11	Load Combination		1.5(DL+LL) 1.2(DL+LL±EQ) 1.5(DL±EQ) 0.9DL±1.5EQ	1.35Gk+1.5Qk 1.35Gk+1.05Qk ± 1.5EL Gk=dead load Qk=live load
12	Storey drift limitation		0.004*h* *h*=height of building	0.005*h* *h*=height of building
			0.0552 m	0.069 m
13	Design Methodology		Limit State Design Design Action ≤ Design Strength	Limit State Design Design Action ≤ Design Strength

Table 11: Comparisons

V. CONCLUSION AND FUTURE SCOPE

A. *General*

The following conclusions are drawn from the analysis and design of steel framed multi-storied building for IS 1893-2002 (part 1), IS 800-1984, IS 800-2007, EURO CODE 8, EURO CODE 3, EURO CODE 4.

1) Design criterion for IS 800-2007, EC 3 and EC 4 is same.
2) Composite beam and column requires smaller sections as compared to normal steel beam column i.e. very economical and also reduced dead loads.
3) The behavior of structure depends on the time period of the structure. Hence the distribution of the seismic forces along the storey height can be done according to the time period of the structure.
4) Time period calculated from EC 8 is more than IS 1893 2002 (Part 1)
5) Seismic forces along the storey height are distributed in parabolic curvature in IS 1893 2002 (Part 1) instead in linear way in the EC 8. So IS 1893 2002 (Part 1) gives well distributed forces along the height as compared to EC 8.
6) Plastic design method is a part of limit state design which uses the full strength of the material used.

B. *Scope for Future Work*

In the present study the analysis and design of a four storey 3D frame has been carried out for different codes like IS 1893-2002 (part 1), IS 800-1984 and IS 800-2007, EURO CODE 8, EURO CODE 3, EURO CODE 4. The same study can be extended by using different code for concentric braced frames, eccentric braced frames.

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