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Applying Performance Based Plastic Design Method to Steel Moment Resisting Frame in Accordance with the Indian Standard Code

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ABSTRACT

The current Indian Standard code uses the limit state procedure (which is a force based design) for design of steel structures to ensure a good earthquake resistant design which at times may fail in case of a severe earthquake as it is based on elastic analysis. The recently developed Performance based Plastic Design method is a displacement based method in which a predetermined failure pattern is used at certain points of a structure based on “strong column –weak beam concept”. An attempt has been made here to design a steel moment resisting frame using the Performance Based Plastic Design Method in accordance to the provisions of Indian Standard code IS 800: 2007. Plastic design is performed to detail the frame members as per IS 800: 2007 in order to achieve the intended yield mechanism and behavior. It is expected that a frame would perform better under seismic attack if it could be assured that plastic hinges would occur in beams rather than in columns, and if the shear strength of members exceeded the shear corresponding to flexural strength. This method also ensures possible inexpensive repairs even after damage.

Keywords: *Performance Based Plastic Design Method, Plastic hinges, Reduced Beam Sections, Yield Mechanism.*

1. INTRODUCTION

It is well known that structures designed by current codes undergo large inelastic deformations during major earthquakes. However, current seismic design approach is generally based on elastic analysis and accounts for inelastic behavior in a somewhat indirect manner. In the current Indian Standard seismic design practice, the design base shear is obtained from code-specified spectral acceleration, assuming the structures to behave elastically, and reducing it by force reduction factor, R, depending upon available ductility of the structural system. The design forces are also adjusted for the importance of specific structures by using an occupancy importance factor; I. Appropriate detailing provisions are then followed in order to meet the expected ductility demands.

However, when struck by severe ground motions, the structures designed by such procedures have been found to undergo inelastic deformations in a somewhat ‘uncontrolled’ manner. The inelastic activity, which may include severe yielding and buckling of structural members and connections, can be unevenly and widely distributed in the structure. This may result in a rather undesirable and unpredictable response, sometimes total collapse, or difficult and costly repair work.

Performance Based Plastic Design (PBPD) method which uses pre-selected target drift and yield mechanisms as key performance objectives, was developed by Lee and Goel (2001) and Leelataviwat et al., (1999). Results of extensive inelastic static and dynamic analyses have proven the validity of the method. The method has been successfully applied to steel Moment Frame (MF) (Lee and Goel, 2001), buckling restrained braced frame (BRBF) (Dasgupta et al., 2004), Eccentrically Braced Frame (EBF) (Chao and Goel, 2006a), Special Truss Moment Frame (STMF) (Goel and Chao, 2008), and concentric braced frames (Chao and Goel 2006b).In all cases, the frames developed the desired strong column–weak beam yield mechanisms as intended, and the storey drifts/ductility demands were well within the selected design values, thus meeting the selected performance objectives. Comparisons of responses with corresponding baseline frames designed by current practice have consistently shown superiority of the proposed methodology in terms of achieving the desired behavior.

In this paper, a steel moment resisting frame has been designed using the PBPD method in accordance with the IS 800: 2007 code. The IS 800: 2007 code is based on the Limit State Design Philosophy and in this code, provisions have been made for plastic design also. The design base shear is calculated by equating the work needed to push the structure monotonically up to the target drift to the energy required by an equivalent Elasto Plastic Single Degree Of Freedom system to achieve the same state (Housner, 1960). Also, a new distribution of lateral design forces is used that is based on relative distribution of maximum storey shears consistent with inelastic dynamic response results (Chao et al., 2007).

2. DESIGN OF A STEEL MOMENT RESISTING FRAME USING THE PBPD METHOD

A steel moment resisting frame as shown in Figure 1 for a given values of floor seismic weight (which is calculated in accordance with the IS 875 Parts I to V and IS 1893:2000) is designed for data given in Table 1. The value of inelastic spectral acceleration coefficient "$S_{a\ inelastic}$" is taken from the Idealized inelastic spectra by Newmark and Hall (1982). It is intended to design this frame for a target drift of 2% i.e. for ductility factor 2. It is interesting to note that if the target drift is chosen 1% (same as yield drift) then, the design becomes an elastic design.The step by step procedure of the design of this steel moment resisting frame using the PBPD method is as follows

Step 1: Calculate gravity loading and Seismic loading for the structure "W". (Using IS-875 Parts I to V)

Step 2: Select an appropriate vertical distribution of forces based on the mode shapes obtained from Modal Analysis.(usually mode shape 1 is chosen)

Step 3: Select a desired Target Yield Mechanism for design earthquake hazard.

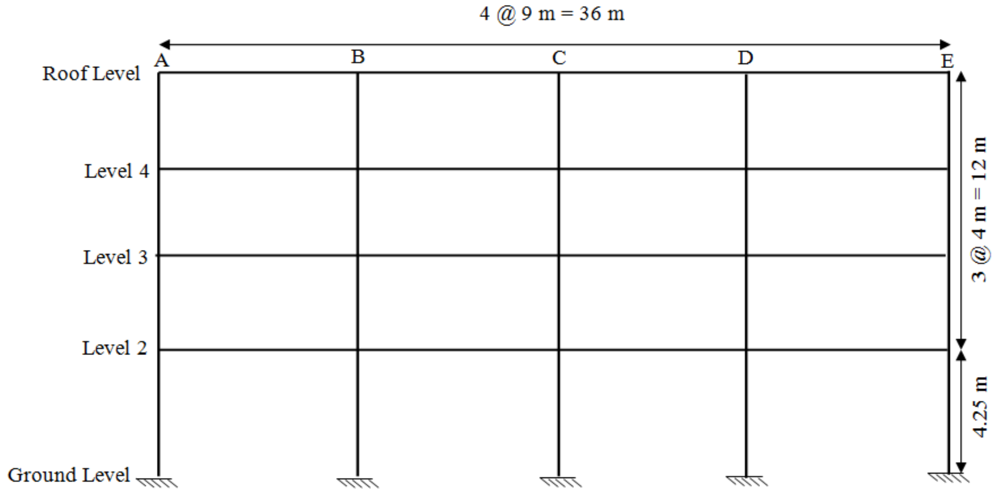

Figure 1. The Elevation of the Steel frame.

Table 1. Design parameters of the steel frame

Type of structure	Steel Moment Frame.
Number of stories	4
Floor height	4 m for all floors and 4.25 m for first floor
Frame Height	16.25 m
Materials	Structural steel with f_y = 250 N/mm^2
Floor Seismic Weight for Roof	300 KN
Floor Seismic Weight for Level 4	280 KN
Floor Seismic Weight for Level 3	270 KN
Floor Seismic Weight for Level 3	270 KN
Seismic zone factor, Z	0.4
Soil Profile Type	Type 2 Medium
Importance factor, I	1
Inelastic Spectral Acceleration $S_{a\ inelastic}$	0.64 g
Natural Time Period T	0.4 sec
Yield drift ratio θ_y	1 %
Target drift ratio θ_u	2 %
Inelastic drift ratio $\theta_p = \theta_u - \theta_y$	1%
Ductility factor $\mu = \frac{\theta_u}{\theta_y}$	2.0
Reduction Factor due to Ductility R_μ	2.0

Energy Modification Factor γ,where $\gamma = \frac{2\mu-1}{R_\mu^2}$	0.75

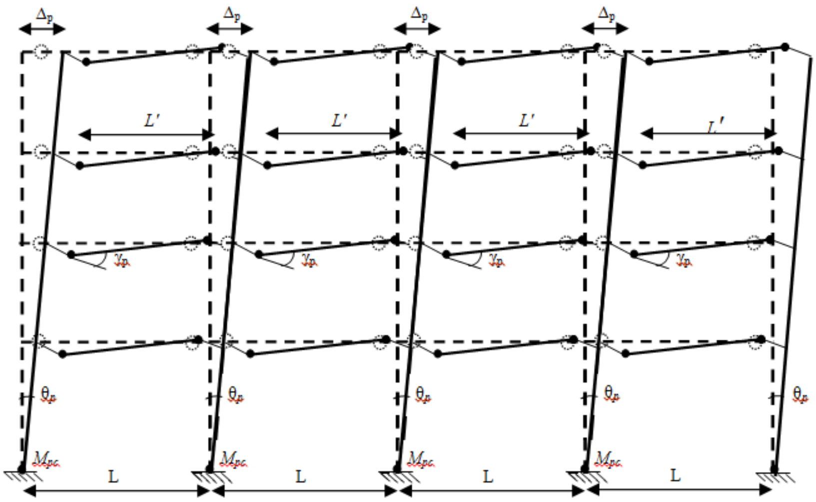

Figure 2. The Target and Yield mechanism selected for the Steel frame.

The PBPD method is a direct design method where drift and yield mechanism, e.g. strong column–weak beam condition, are built in the design process from the very start. Figure 2 shows the design yield mechanism of the frame with Reduced Beam Section (RBS) subjected to design lateral forces and pushed through the design target plastic drift, "Δp". All inelastic deformations are intended to be confined within plastic hinges in beams. Because plastic hinges at column bases may also form during a major earthquake, the global yield mechanism also includes plastic hinges there. The distance between centers of RBS cuts or the distance between two hinges is taken as 0.8375 times the span of the beam (L'=0.8375L).Since the distance between centers of RBS cuts is less than the beam centerline span, the plastic rotation of the beams (γ_p) is greater than the plastic (or inelastic) drift ratio (θ_p) of the global frame and can be calculated as

$$\gamma_p = \left(\frac{L}{L'}\right)\theta_p \qquad (1)$$

Where

L = span of the bay
L'= The distance between the two plastic hinges of the beam = 0.8375L

The hinges are formed in beams only and hence they are named as Designated Yielding Members (DYM) and columns are Non Designated Yielding Members (NDYM).

Step 4: Calculate the shear distribution factor "β_i" of each floor.

The current design codes obtain these lateral forces on the assumption that the structure behaves elastically and primarily in the first mode of Vibration. However, building structures designed according to these procedures undergo large deformation in the inelastic range when subjected to major earthquakes. In order to achieve the main goal of performance based design i.e. a desirable and predictable structural response, it is necessary to account for inelastic behavior of structures directly in the design process. Unlike the force distribution in the current codes, the design lateral force distribution used in the PBPD method is based on maximum story shears as observed in nonlinear Time history analysis results. (Chao et al ,2007) as follows

$$\beta_i = \left(\frac{\sum_{j=1}^{n} w_j h_j}{w_n h_n}\right)^{0.75T^{-0.2}} \qquad (2)$$

Where

β_i = Shear distribution factor at level i
w_j= seismic weight at level j
h_j= height of level j from base
w_n= seismic weight at the top level
h_n= height of roof level from base
T= Natural Time Period

Step 5: Calculate α.

The calculation of story shear is based on the work energy equation The design base shear is calculated by equating the work needed to push the structure monotonically up to the target drift to the energy required by an equivalent Elasto Plastic Single Degree Of Freedom system to achieve the same state (Housner, 1960)

$$\alpha = \left(\sum_{i=1}^{n}(\beta_i - \beta_{i+1})h_i\right)\left(\frac{w_n h_n}{\sum_{j=1}^{n} w_j h_j}\right)^{0.75T^{-0.2}}\left(\frac{\theta_p 8\pi^2}{T^2 g}\right) \qquad (3)$$

Where

g = gravitational force
h_i=height of level i from base

Step 6: Calculate Story shear "V"

$$V = \left(\frac{-\alpha + \sqrt{\alpha^2 + 4(\gamma/\eta) S_{a\,inelastic}{}^2}}{2} \right) W \tag{4}$$

Where

V = total story shear at base (i.e. level 2)
η = 1.0 for ductile designated yield members
W = Total design Seismic Load
$S_{a\ inelastic}$ = Design Inelastic Spectral Acceleration
γ = Energy modification factor

Step 7: Calculate the Lateral force "F_n" of Roof Floor.

$$F_n = \frac{V}{\sum(\beta_i - \beta_{i+1})} \tag{5}$$

Where

F_n =Lateral Force at roof level (n^{th} level)

Step 8: Calculate the Lateral force "F_i" of each level.

$$F_i = F_n(\beta_i - \beta_{i+1}) \tag{6}$$

Where

F_i =Lateral Force at i^{th} level

Step 9: Calculate the required beam moment capacity "M_u" at each level

$$M_u = \beta_i M_{pb} = \beta_i \frac{\sum_{i=1}^{n} F_i h_i - 2M_{pc}}{2\sum_{i=1}^{n}\left(\beta_i \frac{L}{L'}\right)} \tag{7}$$

Where

$$M_{pc} = \frac{1.1 V' h_1}{4} \tag{8}$$

where

V' = base shear for a 1-bay frame, which is equal to V divided by the number of bays
M_{pc}= required plastic moment of columns in the first story of the 1-bay model
M_{pb} = required moment strengths at the top floor level
$\beta_i M_{pb}$ = required moment strengths at level i.
M_u = required moment strength = M_{beam} i.e The required maximum moment capacity of the beam

Step 10: Calculate the design beam moment by applying proper safety factors

$$M_{design} = 1.46\ M_u \tag{9}$$

Where

M_{design}= the required design moment capacity of the beam with a factor of safety 1.46.

Step 11: Calculate the section modulus required.

$$Z_p = M_{design} / f_y \tag{10}$$

Where

Z_p = plastic section modulus for full beam cross-section,

Table 2 shows the Calculation of Loads, Moments and Section Modulus Required for design of beams.

Step 12: Design of Beams using the RBS sections.

Select RBS dimensions a, b, and c (Figure 3) subject to the following limits:

$$0.5b_f \le a \le 0.75b_f$$

$$0.65d_b \le b \le 0.85d_b$$

$$0.1b_f \le c \le 0.25b_f$$

Where

b_f = width of beam flange
d_b= depth of beam section
a = distance from face of column to start of RBS cut
b = length of RBS cut
c = depth of cut at the center RBS section at the center of reduced beam section.

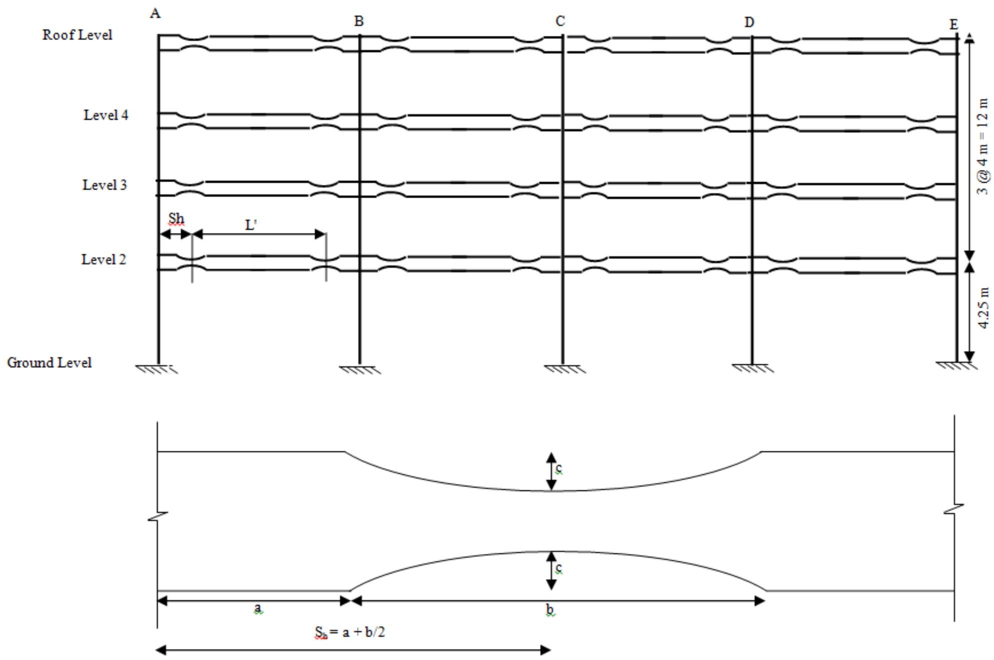

Figure 3. The Reduced Beam Sections designed for the Steel frame.

Table 2. Calculation of Loads , Moments and Section Modulus Required for design of beams

	Roof	Level 4	Level 3	Level 2	Σ Sum
Seismic Weight of the floor (KN)	300	280	270	270	1120
Shear Distribution Factor "β i"	**1.000**	**1.616**	**2.002**	**2.197**	**6.815**
ψ i	0.455	0.280	0.176	0.089	1.000
α				6.4	
Lateral force "Fn" of Roof Floor (KN)				147.2	
Lateral force "F_i" of each level (KN)	**147.2**	**90.7**	**56.8**	**28.8**	323.4
Story Shear Vi (KN)	**147.2**	**237.9**	**294.6**	**323.4**	
Actual Design Lateral force "F_i" of each level for one bay (KN)	36.8	22.7	14.2	7.2	80.9
Required Plastic moment of columns in the first story of the 1-bay model " M pc "					94.5
Required moment strengths at the top floor level " M pb"					**51.3**
Required Section Modulus "Z preq" (cm ^{3})	**300.8**	**486.0**	**602.1**	**660.8**	

Step 13: Check the compactness of the RBS. The ratio of Z_{RBS}/Z_p should be near to the assumption 0.75

$$Z_{RBS} = Z_p - 2ct_f(d_b - t_f) \qquad (11)$$

Z_{RBS} = plastic section modulus at center of the reduced beam section,
t_f = thickness of beam flange.

The Moment Capacity M_{RBS} of this RBS beam is calculated as

$$M_{RBS} = \frac{Z_{RBS} f_y \beta_b}{\gamma_{mo}} \qquad (12)$$

β_b= 1.0 for plastic and compact sections(Clause 8.2.1.2)
γ_{mo} = Partial safety factor = 1.1 (Clause 5.4.1)

The calculated Z_{RBS} values for all section are shown in Table 3, where the ratios of Z_{RBS}/Z_p are also given. The assumption of Z_{RBS}/Z_p = 0.75 is generally satisfied.

Step 14: Checking the Moments and Shears of the beams Checking Shear

Compute the probable maximum moment at the center of the reduced beam section:

$$M_{prRBS} = C_{pr}\, \gamma_{mo}\, f_y\, Z_{RBS} \qquad (13)$$

Where

M_{prRBS} = probable maximum moment at the center of the reduced beam section

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C_{pr} = factor to account for the peak connection strength, including strain hardening, local restraint, additional reinforcement, and other connection condition (=1.15)
f_y = tensile strength of steel= 250 N/mm^2

However, it can be over – conservative, especially for the design of columns, by using C_{pr} =1.15 for beams at all levels because it is unlikely that beams at all levels reach their ultimate strengths at the same time. In addition, column hinging is allowed at the top floor. Therefore, a value of C_{pr}= 1.0 for roof beams and 1.075 (average of 1.0 and 1.15) for beams at the other levels was used for the design of columns.

The shear at the center of the reduced beam section, is determined by a free – body diagram of the portion of the beam between the centers of the reduced beam section. This calculation assumes that moment at the center of reduced beam section is M_{prRBS} and gravity loads are included. Where L' is the distance between the centers of RBS cuts

$$V_{RBS} = \frac{2M_{prRBS}}{L'} + \frac{(udl)L'}{2}$$

$$V'_{RBS} = \frac{2M_{prRBS}}{L'} - \frac{(udl)L'}{2} \tag{14}$$

This shear V_{RBS}<V_n as per the Clause 8.4.1 IS 800:2007

$$V_n = \frac{A_v f_y}{\sqrt{3}} = \frac{ht_w f_y}{\sqrt{3}} = \frac{d_b t_w f_y}{\sqrt{3}} \tag{15}$$

Where

V_n = nominal plastic shear resistance of section
V_{RBS} and V'_{RBS} = probable maximum shear force at the center of the reduced beam section
udl = uniformly distributed load

Checking Moments

Compute the probable maximum moment, "M_{fcRBS}" at the face of the column. If it is larger than the expected moment capacity of the beam section at that location, the beam section will need to be further reduced. The expected moment at the face of the column is computed as follows:

$$M_{fcRBS} = M_{prRBS} + V_{RBS}S_{\boldsymbol{h}} \quad \text{and}$$

$$M'_{fcRBS} = M_{prRBS} + V'_{RBS}S_{\boldsymbol{h}} \tag{16}$$

The moment carrying capacity of the section is given by Clause 8.2.1.2 IS 800: 2007

$$M_p = \frac{Z_p f_y \beta_b}{\gamma_{mo}} \tag{17}$$

Where

S_h =The distance from a column face to the center of RBS cut = a + b/2
M_{fcRBS} and M'_{fcRBS} = probable maximum moment at the face of the column
M_p=he moment carrying capacity of the section
This "M_{fcRBS}" should not exceed "M_p".

Table 3 shows the design of beams and Checks

Step 15: Calculation of Design Axial Force and Moments for the Exterior and Interior Columns

The columns are designed as Non Designated Yielding Members as they are intended to remain elastic and are based on capacity design approach. These members should have design strength to resist combination of factored gravity loads and maximum expected strength of the Designated Yielding Members accounting for reasonable strain hardening and material over strength.

When a structure is subjected to seismic loading, it undergoes large inelastic deformations due to large moments in columns. The column moments are underestimated in the elastic design approach, as only the moments coming from beams and from other members connected to columns are considered in to the calculation. But apart from these, the columns also have moments due to their own deformations.

But, these shortcomings are overcome in the PBPD method by considering the equilibrium of the entire column tree in the extreme limit state.

Exterior Column Tree

$$F_L = \frac{\left[\sum_{i=1}^{n}(M_{prRBS})_i\right] + \left[\sum_{i=1}^{n}\left\{(V_{RBSi})\left(S_h + \frac{d_c}{2}\right)_i\right\}\right] + [M_{pc}]}{\sum_{i=1}^{n}\psi_i h_i} \tag{18}$$

Where

F_L = sum of lateral forces in columns
V_{RBSi} = probable maximum shear force at the ith level at the center of the reduced beam section

Where

$$\psi_i = \frac{(\beta_i - \beta_{i+1})}{\sum_{i=1}^{n}(\beta_i - \beta_{i+1})} \tag{19}$$

Interior Column Tree

$$F_L = \frac{\left[2\sum_{i=1}^{n}(M_{prRBS})_i\right] + \left[\sum_{i=1}^{n}\left\{(V_{RBSi} + V'_{RBSi})\left(S_h + \frac{d_c}{2}\right)_i\right\}\right] + [2M_{pc}]}{\sum_{i=1}^{n}\psi_i h_i} \tag{20}$$

Table 3. Design of beams and Checks

	Roof	Level 4	Level 3	Level 2	Σ Sum

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udl (KN /m)	9.0	10.0	10.0	10.0	
Plastic Section modulus required "Z preq" (cm^3)	300.8	486.0	602.1	660.8	
Selection of IS Section	ISLC250	ISMC300	ISMB300	ISMB300	
Plastic Section modulus of IS Section (cm^3)	338.0	496.0	651.7	651.7	
Sectional Area (cm^2)	35.7	45.6	56.3	56.3	
Depth of section (cm)	25.0	30.0	30.0	30.0	
Width of flange (cm)	10.0	9.0	14.0	14.0	
Thickness of flange (cm)	1.07	1.36	1.24	1.24	
Thickness of web (cm)	0.61	0.76	0.75	0.75	
Choose a (cm)	6.0	6.0	6.0	6.0	
choose b (cm)	20.0	20.0	20.0	20.0	
choose c (cm)	2.0	2.0	2.0	2.0	
S_h cm (cm)	16.0	16.0	16.0	16.0	
Plastic section modulus of RBS "Z rbs" (cm^3)	235.6	340.2	509.1	509.1	
Compactness Zrbs/Zp	0.7	0.7	0.8	0.8	
Moment capacity of RBS "M rbs" (KN m)	58.9	85.0	127.3	127.3	
Moment capacity of built up beam "M p" (KN m)	84.5	124.0	162.9	162.9	
Probable Moment capacity at centre of RBS	64.8	100.6	150.5	150.5	466
Shear force at the centre of RBS (downwards)	51.4	64.6	77.6	77.6	
Shear force at the centre of RBS (upwards)	-17.6	-12.0	1.0	1.0	
Nominal Shear Force Capacity of the section	220.1	329.1	324.8	324.8	
Probable Moment at face of column due to RBS	73.0	110.9	162.9	162.9	
Probable Moment at face of column due to RBS	62.0	98.6	150.7	150.7	

Table 4 shows the calculation of Loads, Moments and Section Modulus Required for design of Columns

Step 16: Design of Exterior and Interior Columns

The columns are to be designed as beam columns as per Clause 9.3 IS 800:2007 for combined axial force and bending moment. It should be noted that the Moment acts about the direction of lateral force. The following should be satisfied (Clause 9.3.1.1 IS 800:2007)

$$\frac{N}{N_d} + \frac{M_y}{M_{ndy}} \leq 1.0 \qquad (21)$$

Where

N = Factored applied axial force
N_d = Design Strength in Tension obtained as $A_g f_y/\gamma_{mo}$
A_g = Gross Cross Sectional Area
M_y = Factored applied moments
M_{ndy} =Design reduced flexural strength obtained as $\beta_b Z_p f_y/\gamma_{mo}$

Table 5 shows design and Checks for columns

Table 4. Calculation of Loads, Moments and Section Modulus Required for design of Columns

Exterior Column Tree

	Roof	Level 4	Level 3	Level 2	Σ Sum
d_c (cm)	30.0	30.0	30.0	30.0	
$S_h + d_c / 2$ (cm)	31.0	31.0	31.0	31.0	

F_L (KN)					51.0
Column Shear (KN)	23.2	37.5	46.4	51.0	
Point Load on Columns (downwards) (KN)	20.0	20.0	20.0	20.0	
Total Design Axial Force	71.4	155.9	253.5	351.1	
Total Moment due to vertical load + moment due to RBS	80.7	201.3	375.9	550.4	
Total Moment due to lateral load	0.0	92.8	242.7	428.4	644.9
Negative Bending Moment	-12.1	-41.4	-52.5	-94.5	
Total Design Moment (KN m)	80.7	108.5	133.2	122.1	

Interior Column Tree

	Roof	Level 4	Level 3	Level 2	Σ Sum
"V rbs + V' rbs" (KN)	33.8	52.5	78.6	78.6	
(V rbs +V' rbs)($S_h + d_c / 2$)	10.5	16.3	24.4	24.4	75.5
Probable Moment capacity at centre of RBS for Interior Columns	129	201	301	301	932
F_L (KN)					94.6
Column Shear (KN)	43.1	69.6	86.2	94.6	
Point Load on Columns (downwards) (KN)	10.0	10.0	10.0	10.0	
Total Design Axial Force	78.9	165.5	252.1	338.7	
Total Moment due to vertical load + moment due to RBS	140	357	682	1008	
Total Moment due to lateral load	0	172	450	795	1197
Negative Bending Moment	-32.2	-93.0	-112.4	-189.0	
Total Design Moment (KN m)	140	185	232	213	

Table 5. Design and Checks for columns

Exterior Columns

	Roof	Level 4	Level 3	Level 2
Plastic Section modulus required (cm^3)	355.1	477.5	586.0	537.0
Selection of IS Section	ISLB275	ISLB300	ISMB350	ISMB350
Plastic Section modulus of IS Section (cm^3)	443.0	554.0	889.0	889.0
Sectional Area (cm^2)	42.0	48.1	66.7	66.7
Depth of section (cm)	27.5	30.0	35.0	35.0
Width of flange (cm)	14.0	15.0	14.0	14.0
Thickness of flange (cm)	0.88	0.94	1.42	1.42
Thickness of web (cm)	0.64	0.67	0.81	0.81
Section modulus of section (cm^3)	392.0	488.0	779.0	779.0
Shape Factor " Z p / Zxx"	1.13	1.1	1.1	1.1
Moment of Resistance of the Section	100.7	125.9	202.0	202.0
Axial Strength of the Section	955.00	1092.73	1515.91	1515.91

Interior Columns

	Roof	Level 4	Level 3	Level 2
Plastic Section modulus required (cm^3)	616.2	815.2	1022.3	937.3
Selection of IS Section	ISLB325	ISMB350	ISMB400	ISMB400
Plastic Section modulus of IS Section (cm^3)	687.0	889.0	1176.0	1176.0
Sectional Area (cm^2)	54.9	66.7	78.4	78.4
Depth of section (cm)	32.5	35.0	40.0	40.0
Width of flange (cm)	16.5	14.0	14.0	14.0
Thickness of flange (cm)	0.98	1.42	1.60	1.60
Thickness of web (cm)	0.70	0.81	0.89	0.89
Section modulus of section (cm^3)	607.00	779.00	1020.00	1020.00
Shape Factor " Z p / Zxx"	1.1	1.1	1.2	1.15
Moment of Resistance of the Section	156.1	202.0	267.3	267.3
Axial Strength of the Section " N d"	1247.727	1515.909	1781.818	1781.818

3. DISCUSSION

A Steel Moment Resisting Frame is designed using the Performance Based Plastic Design Method with pre selected failure mechanism. The PBPD method is based on the "strong column weak beam" concept in which the beams are designed as Reduced Beam Sections at certain predetermined points. Due to this, failure always occurs in beams and not in columns which prevents the Total Collapse of the Structure increasing the life safety. Following Points were observed during the whole design process

- The Structure is designed taking into consideration its inelastic properties. This leads to the optimum utilization of the sections.
- The lateral forces are calculated based on the Spectral Acceleration of the inelastic Design Spectra, depending on the performance objective.
- These lateral forces are distributed according to new distribution factor defined on the basis of real ground motions.
- As compared to explicitly designed Earthquake resistant Structures (like base isolation, dampers, energy dissipation devices), this method proves to be more practical as the designer includes the effect of earthquake forces and designs accordingly.
- The fabrication of the RBS will have to be done exclusively for a particular structure.

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