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# SEISMIC DESIGN VERIFICATION OF STEEL PRE-ENGINEERED BUILDINGS

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Steel built-up I sections, composed of plates with high width-to-thickness ratios (slender sections), are commonly used in pre-engineered buildings under the premise that the design is governed by wind. However, in the event of a severe earthquake, the sections are susceptible to local buckling and may exhibit a non-ductile behavior. Therefore it is imperative to check the performance of such structures under the maximum credible earthquake (MCE). As a first step towards this objective, it is necessary to evaluate the post-buckling strength and ductility of such sections. In this study, a Finite element model is developed to analyze the inelastic post-buckling response of semi-compact and slender plates. The information can be used to predict the moment-rotation curves for I-sections with slender webs. A parametric study was carried out on a total of 54 pinned-base PEB frames of varying spans and heights. The elastic seismic demand under severe earthquake was estimated and compared with the design lateral capacity of PEB frames. From the results, it is concluded that even for higher seismic zones, low ductility sections (1.5 to 2) are adequate to survive MCE. Alternatively, if the design is verified for a response reduction factor of 2, then non-ductile sections can also be used.

*Keywords*: Single-storey industrial buildings, Slender sections, Ductility, Pushover analysis, Response reduction factor.

## 1 INTRODUCTION

Single-storey Pre-engineered Building (PEB) structures are preferred because of faster construction, superior quality and economy compared to the conventional steel structures. These buildings have relatively light roofs and are characterized by low axial load ratios. Hence they are designed with built-up I-sections having high plate slenderness (width-to-thickness ratios) which makes them stiff enough for controlling deformations. Being light-weight structures, the lateral load due to design basis earthquake is often less as compared to the design wind force. The ratio of wind to earthquake load is also influenced by factors such as the span to height ration of the structure and its geographical location and has been found to vary from 1 to 17 as shown later in this study. Hence, designers often assume that there is no need to adhere to ductile detailing as stipulated in section 12 of the code for structural steel design IS800:2007. However, in the event of a severe earthquake, the sections are susceptible to local buckling and may exhibit a non-ductile behavior. Therefore it is imperative to check the performance of such structures under the maximum credible earthquake (MCE).

Structures are designed for seismic force which is reduced by a factor 'R' known as response reduction factor. The R value accounts for the level of structural ductility as well as the energy dissipation capacity of the structure. Ductility of the cross-section is ensured in design by section classification criteria based on the flange width to thickness ($b/t_f$) ratio and web depth to thickness ($d/t_w$) ratio. Since Rafters and columns in PEBs are subjected to predominant bending and very little axial loads, they are designed as plate girders with slender webs. Standard codes permit the use of slender sections in design, provided the extra depth of web and width of flange, in excess of the semi-compact limits, is considered as ineffective in calculating the section properties. Hence effect of plate slenderness ratios on the seismic performance of PEBs needs to be assessed to justify the R values used in design.

## 2 MOMENT-ROTATION CURVES FOR SLENDER WEB SECTIONS

Finite Element Analysis was carried out to obtain the moment-rotation curve for beams with semi-compact and slender webs well beyond the point of local bucking. A cantilever beam was modeled in the finite element software ABAQUS (2011). The four-noded, S4R5 shell element was used in this model since the objective was to capture the local buckling behavior of the beam cross-section. Cyclic loads tend to reduce or remove the yield plateau and work-harden the steel. Hence, a bi-linear elastic-plastic-hardening (EPH) stress-strain relationship was used with Young's modulus E of 200000 MPa and a strain hardening stiffness Et of E/100 as prescribed by Smitha and Satish Kumar (2013). Initial imperfections were applied by first carrying out a buckling analysis and then including the local buckling shape with of amplitude of 1/250th of the depth of the web. To avoid the lateral-torsional buckling of the member, the movement at the intersection of web with top and bottom flange was restricted in the lateral direction.

The validation of the FEM model was obtained by analyzing two wide-flange cantilever beams of span 3700 mm tested by D'Aniello *et al*. (2012) the properties of which are as given below:

(a) HEA 160: Flange 160x9; Web 152x6; yield stress = 337 MPa; Ultimate stress = 465 MPA.

(b) HEB 240: Flange 240x17.5; Web 240x10; yield stress fy=319MPa; Ultimate stress=431 MPA.

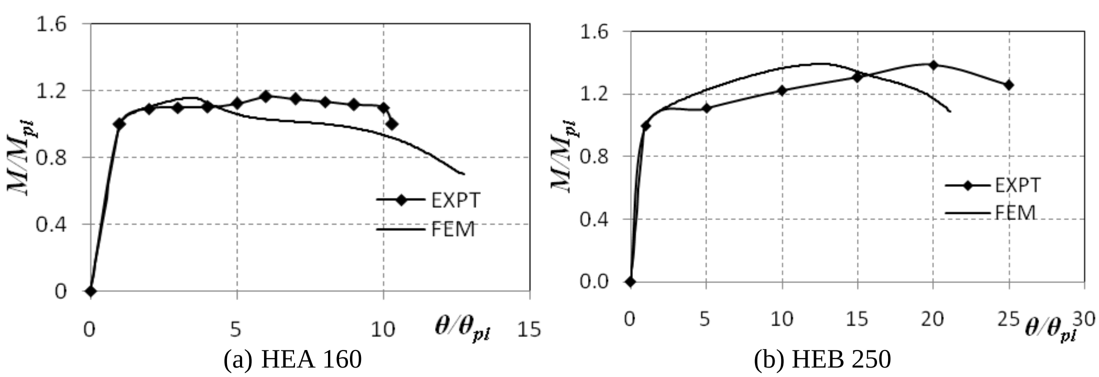


Figure 1. Validation of FEM results with tests by D'Aniello *et al.* (2012).

The analysis results are compared with test results in Fig. 1 for the two beams where the downward drooping of the curves is due to flange local buckling. The FEM Model was then used to generate the moment-rotation curve for various required sections and was used in the pushover analysis as explained in the next section.

## 3 SEISMIC PERFORMANCE OF PEB FRAMES

The Strength of PEB frames is governed by several factors such as the magnitude of the gravity, wind and seismic loads acting as well as the values of span, height and girder depths. In order to understand the effect of these on the seimic performance, 54 frames with varying parameters were designed. Pinned-base PEB frames with spans of 15, 20, 25 and 30 m length and heights of 6, 10 and 15 m were considered giving nine different L/h values. Dead load (DL), live load (LL) and collateral load (CoL) are taken as 0.1 kN/m$^2$, 0.6 kN/m$^2$ and 0.5 kN/m$^2$, respectively giving a total gravity load of 1.2 kN/m$^2$. The lateral forces such as wind and seismic loads are calculated depending on the span, height and site location of PEB frames as per the required codal provisions. Elastic analysis was used to calculate the member forces. Serviceability criteria were checked by limiting the lateral roof displacement to h/150 and vertical deflection to span/240 (IS 800:2007). Design of members was done by excel spreadsheet and complies with code provisions. In general, for spans larger than 30 m the gravity load dominates the design while for tall frames lateral drift becomes the design criteria.

Based on the Indian codes for wind (IS 875-part 3: 1985) and earthquake forces (IS 1893: 2002), it was found that depending on the frame size and location, the ratio of wind load to earthquake loads (WL/EL) varied from 1 to 17.3 thereby testifying to the fact that these frames are often governed by wind rather than earthquake loads. Another reason for this is that the earthquake loads considered is the Design Basis Earthquake (DBE) which is further reduced by a reduction factor R which is prescribed as 4 for ordinary moment frames. The reduction factor is for general building frames which have higher degree of redundancies and are assumed to have sections with moderate ductility. Thus, it is important to verify the applicability of these values to PEB frames with low redundancies and slender (non-ductile) sections.

Considering the fact that the Maximum Credible Earthquake (MCE) can be twice as large as the Design Basis Earthquake (IS 1893:2002) it can be realized that frames with ratios of Wind Load to Earthquake Load (WL/EL) greater than eight will respond elastically under MCE. Only six frames satisfied this condition. Of the remaining 48 frames, 7 were governed by drift under wind, 13 were governed by strength under wind load and 28 were governed by deflection under gravity load. For all 54 frames, the required reduction factor $R_d$ was calculated as the ratio of design lateral capacity to the seismic strength demand under MCE and plotted in Fig. 2 against the ratio of WL/EL. It can be observed that for 17 of these frames, the value of $R_d$ was greater than unity which means that these frames will experience significant inelastic deformations under a severe earthquake. It can also be noted that the frame designs were governed by all the three criteria, namely drift under wind, strength under wind and serviceability under gravity loads. The ductility demands on these frames under MCE were obtained by pushover analysis as below.

Pushover analysis was carried out on the 17 frames using SAP 2000 (2011) software programme. Second-order effects were considered by using the P-Δ feature available in SAP2000. For both rafter and columns,'M3 hinges' were defined based on the results obtained from the finite element model of beam. Fig. 3 shows the actual and idealized moment-rotation curve as defined in SAP2000. Point 'B' refers to the yield moment of the effective section based on the effective width of the plate elements and its corresponding rotation was obtained from the finite element analysis. Point 'C' defines the maximum moment capacity $M_{max}$ taken as $1.1M_{eff}$ and its corresponding $\theta_{max}$ is taken as $1.4\theta_{eff}$. The moment capacity of section at point 'D' is considered to be the yield moment $M_{eff}$ and its corresponding rotation $\theta_u$ is obtained as $\mu_s\ \theta_{eff}$, where the section ductility $\mu_s$ is given as a function of the depth-to-thickness ratio of the web by the following equation (valid only for plastic and compact flanges) as obtained from FEM.

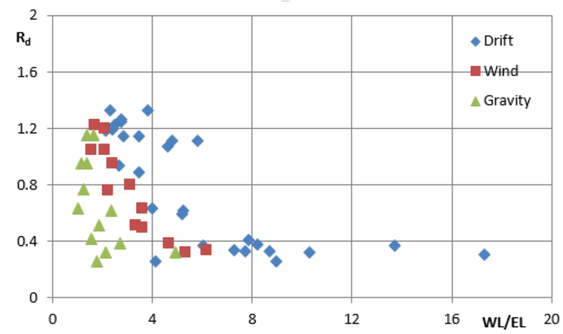


Figure 2. Variation of $R_d$ with *WL/EL* for different governing design criteria.

$$\mu_s = 0.0004(d/t_w)^2 - 0.117(d/t_w) + 10.2 \quad (1)$$

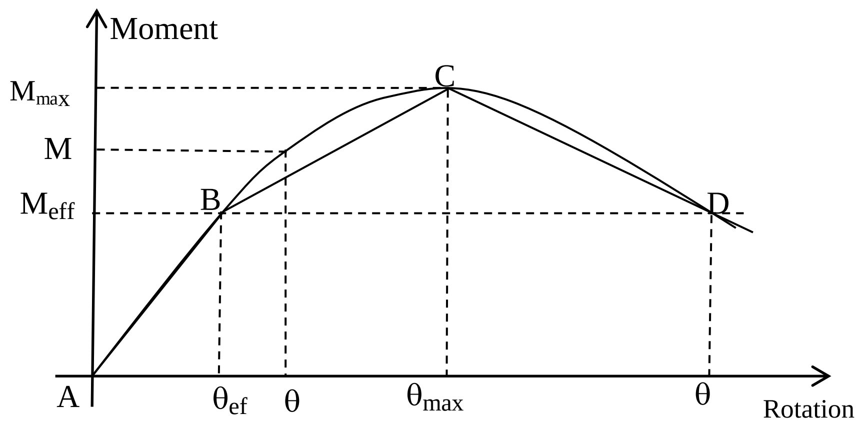


Figure 3. Actual and Idealized Moment-Rotation curves for members.

The elastic response of PEB Portal frame under the application of increasing monotonic lateral force (H) was studied to calculate the design lateral capacity of frames. To have more realistic and conservative results, the frames were subjected to constant seismic weight (2V). Fig. 4(a) shows a typical frame. Increase in lateral force leads to increase of reactions and bending moments in members. Once any section of the frame reaches its yield strength, the structure deviates from its elastic path and goes into the strain hardening region. The curve generated from pushover analysis is converted to an energy-equivalent elastic-perfectly-plastic curve (EEPP) by the bi-linear idealization method by equating the area under the actual response of frame is with the area under the bi-linear curve and yield capacity ($H_y$) and corresponding yield displacement ($\Delta_y$) (see Fig. 4(b)). The ultimate displacement ($\Delta_u$) of the frame is considered as the displacement at which the strength drops back to $H_y$. Structural ductility $\mu$ is defined as the ratio of ultimate displacement to the yield displacement of structure.

Permissible capacity reduction factor for a frame is denoted as $R_c$ because it depends on the strength and ductility of the sections provided. A structure can sustain an elastic seismic demand equivalent to $R_c$ times its yield strength by virtue of its ductility. The $R_c$ values obtained for the frames are plotted against the section ductility in Fig. 5 and it can seen that Rc increases with ductility $\mu$ and its average value for sections with d/tw ratios from 100 to 150 is about 1.9 and corresponds to an average ductility of 1.6. Also shown are the $R_N$ factors prescribed by Newmark and Hall as $\sqrt{(2\mu-1)}$ which are much lower than those obtained in this study.

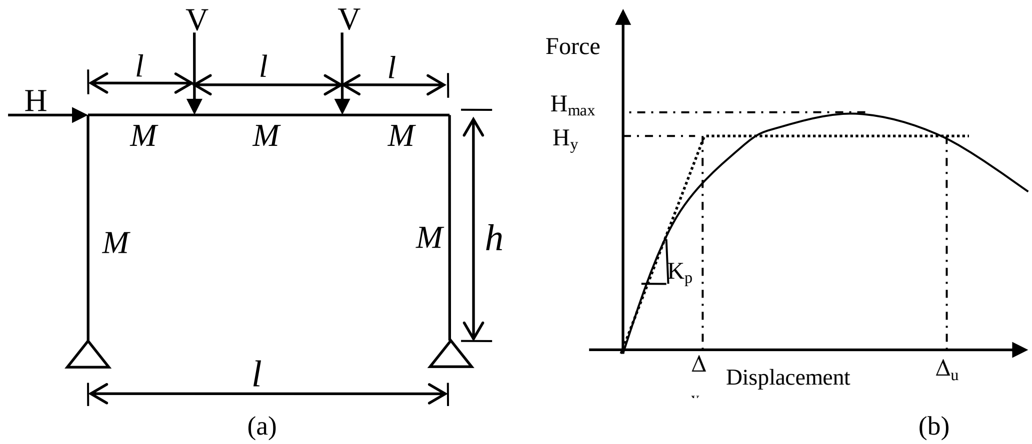


Figure 4. Typical Portal frame analyzed and its bilinear force-displacement curve.

The demand reduction factor, $R_d$ is the ratio of elastic strength demand on the structure under a severe earthquake to the design lateral strength of the structure. $R_d$ value depends on the seismic zone, gravity load considered as well as the importance factor of structure. In seismic design, $R_c$ of a structure should always be greater than $R_d$ value for collapse prevention. The values of $R_c$ are plotted against the $R_d$ values obtained from pushover analysis in Fig. 6. It can be observed that $R_c$ is always higher than $R_d$ thus testifying that the frames are safe against MCE. However, for frames with importance factor I=1.5, several frames are proving to be unsafe as shown in the same figure.

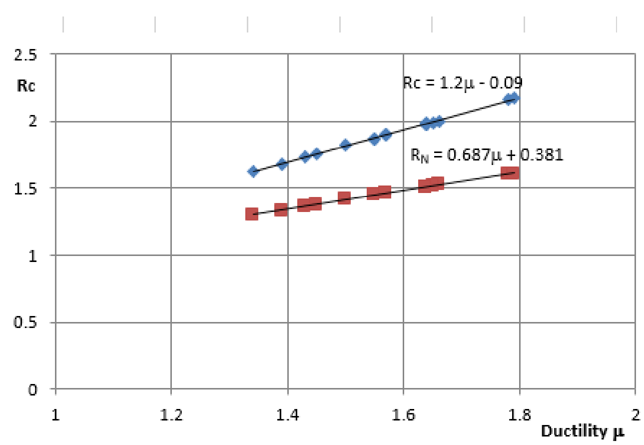


Figure 5. Variation of Rc with section ductility μ.

## 4 CONCLUSIONS

Based on the study, the following conclusions can be drawn:

- When the ratio of WL/EL exceeds 4 for frames with I=1, the frames will remain within their elastic limit under MCE. Similarly, frames with I=1.5 will also remain within their elastic limit for WL/EL ratios exceeding 6. So no limitations on width-to-thickness ratios need to be imposed on these frames to resist severe seismic loads.
- For WL/EL ratios less than 4 and frames with I=1, the frames will undergo inelastic deformations but will not collapse due to the limited ductility. However, for frames with I=1.5, the flanges can be plastic or compact and webs can be semi-compact so as to develop enough ductility to prevent collapse under MCE. Alternatively, the frames can be designed with R factor of 2 to ensure adequate strength to resist MCE.

The above provisions may be incorporated in the code to enable safe and economical design of PEB frames.

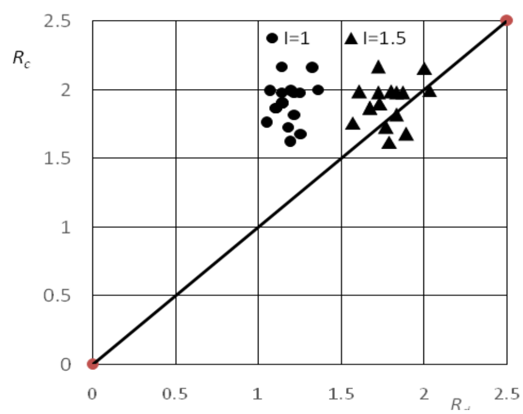


Figure 6. $R_c$ versus $R_d$ for *I=1* and *I=1.5*.

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