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Seismic Design Load for Multi-Story Steel Frames Composed of Expanded Sections

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Abstract. The steel frames, which are composed of expanded-section columns and beams, have bigger moment-resisting capacity and better structural stability. So that they are broadly applied in multi-story structures in low to medium seismic zones. However, the section is likely to local buckling due to its expanded sections, which decreases the structure ductility. To support its application, how to determine the horizontal seismic design load is studied here.

1 Introduction

Being more expanded and having wide flange or high web, the expanded sections are of bigger moment-resisting capacity and better structural stability than compact sections. Therefore, the economy of applying the expanded-section members to building structures is significant. With the rapid development of multi-story steel dwelling houses, multi-story expanded-H-section steel frames have been widely applied in areas that is of low or medium risk from earthquake event. However, the problems of expanded H sections are parallel to their economic advantages: low ductility, reduced yield capacity, shrunken energy-absorbing ability. Just for those deteriorated characters, the expanded-section steel frames are excluded by actual seismic code in China. To support their application, a series of studies have been performed from the section performance to structure performance, in both experimental and numerical way [1] to [7]. Based on the preceding studies, here it focuses on how to determine the horizontal seismic design load for this type of steel frame, which is the most important step for the design work. A factor called load reduction factor is introduced to simplify the design process. In this paper, the numerical study is carried out by using push-over analysis method.

2 Load reduction factor

As we all know, three levels of seismic intensity are considered in the current Chinese structure seismic design code[8]: low earthquake, medium earthquake, and high earthquake. For common structures that are of compact sections, the horizontal seismic design load is usually determined according to low earthquake, that means the structure will come to plastic stage when the seismic input is beyond the design load. The structure's good ductility (the deformation in the plastic stage) and energy-absorbing property are relied on to endure medium or high earthquakes without collapsing.

For the steel frame structures of expanded H-section columns and beams, the member sections are expanded and the flange and web are relatively thin so that local buckling of sections easily occurs. The local buckling, sometimes occurring before section yielding, will reduce the structure yield strength as well as deteriorate deformation performance. With these deteriorated properties, the horizontal seismic design load of expand section steel structures needs to be higher than that of common structures in order to achieve the equal energy consuming ability.

For the structures that are designed to be elastic during low to high earthquake, the horizontal seismic design load is taken as F_e; and for the ductile structures that are in elastic stage only during the low earthquake, the horizontal seismic design load is taken as F_y. The load reduction from F_e to F_y is denoted by load reduction factor R_f:

$$R_f = \frac{F_e}{F_y} \tag{1}$$

The relationship of F_e and F_y is plotted in Figure 1. Where, the path OAB or OI is what elastic structure will follow while the path OACEGHA is what the ductile structure will follow.

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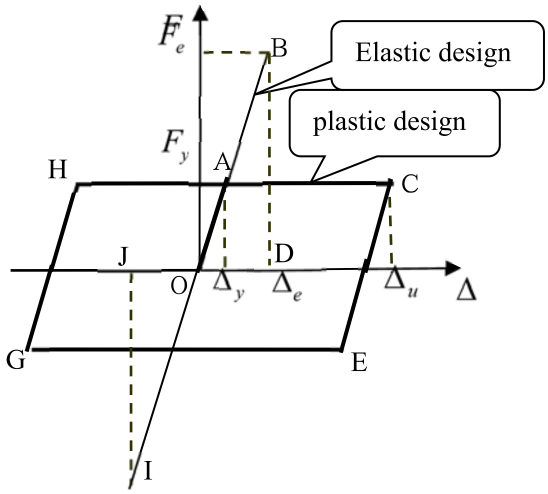

Fig 1. The design load of elastic structure and plastic structure

Introducing the Equal-Energy rule, the consumed energy of elastic structure equals that of ductile structure:

$$\frac{1}{2}\cdot F_e\cdot\Delta_e\cdot 2=F_y\cdot(\Delta_u-\Delta_y)\cdot 4 \quad (2)$$

Where, Δ_e is the displacement of structure in accordance with Fe， Δ_y is the displacement in accordance with Fy. Δ_u is the extreme displacement of ductile structure. The ductile factor is defined as follows:

$$\mu=(\Delta_u-\Delta_y)/\Delta_y \quad (3)$$

Based on equation (2) and (3), and load reduction factor R_f defined in equation (1), the following equation (4) is drawn:

$$R_f=\frac{F_e}{F_y}=\sqrt{4\mu} \quad (4)$$

Where, μ is the ductile factor.

For common structures with compact sections, the adequate ductility ensures the energy consuming ability and therefore makes the reduced design load F_y be acceptable.

However, for steel frame structures with members of expanded sections, the section local buckling will decrease the structure ductility and sometimes make the structure reach the yield point earlier. Therefore, the impact of expanded sections on the structure performance is on two aspects:

(1) decreases the structure ductility

(2) reduce the yield load of structure.

If taking F_{yb} as the factual yield load (or the load where local buckling of critical member occurs, from that on the load will drop) of expand-section steel frames, F_y in Figure 1. will be F_{yb}. Then the factor R_f can be written as:

$$R_{fb}=\frac{F_e}{F_{yb}} \quad (5)$$

The equation (4) is then written as:

$$R_{fb}=\frac{F_e}{F_{yb}}=\sqrt{4\mu} \quad (6)$$

Using symbol ξ to denote the reduction of yield strength when the section local buckling is taken into account:

$$\xi=\frac{F_{yb}}{F_y} \quad (7)$$

Where, F_y is the yield strength obtained by using the conventional push over analysis, in which the sections are treated as compact section with local buckling ignored; F_{yb} is the factual yield strength of expanded-section structures and is obtained by the finite element analysis, in which the local buckling of sections is considered. The latter analysis is called local buckling analysis here; and its difference from the conventional analysis is the expanded section will be local buckled and some portion of section will quit out work. A local buckling hinge whose parameters can accurately demonstrate the section performance is studied in Reference [2], and the local buckling hinge is applied here in push over analysis to take the sectional local buckling into consideration.

Combine equation (7) with equation (5), a new equation can be obtained as follows:

$$R_f=\frac{F_e}{F_y}=\sqrt{4\xi^2\mu} \quad (8)$$

Equation (8) shows that the reduction of ductility (μ) and the reduction of yield strength ($\xi\leq1.0$) can represent the load reduction factor R_f. More expanded the sections are, smaller the factors μ and ξ will be; and as a result, smaller the factor R_f will be. For conventional structures, its horizonal seismic design load F_y is taken according to low earthquake but the structures will sustain the high earthquake. According to the seismic design code, the seismic input of the high earthquake is around 6 times that of low earthquake. Therefore, for conventional structure, R_f is around 6. For structures with expanded sections, R_f will be less than 6, which means the design load F_{yb} will be higher than that of conventional structures.

To implore the load reduction factor R_f, 411 expanded H-section steel frames are analyzed. The analysis is performed by push-over analysis. For each frame, two types of analysis are performed, one is conventional analysis with section local buckling ignored; the other one is local buckling analysis with the section local buckling is considered. The resulted yield strength obtained from two types analysis is applied into the equation (7) to get factor ξ. In parallel, the ductility factor μ can be obtained by P-Δcurve out of the local buckling analysis. With the obtained factors ξ and μ, the load reduction factor R_f can be calculated by equation (8).

Following the process above, each frame will produce a load reduction factor R_f. These values of R_f are then investigated in terms of the structure factors.

3 Structure Factors

3.1 Factors of Steel Frames

Since the steel structures of expanded-sections are commonly applied in low rise structures, the steel frames analyzed in this paper are of 4 stories or 6 stories. The frames properties are summarized as follows.

(1) Steel frame is in plane and is composed of H-

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section column and H-section beam.

(2) The strong axis of column is in the plane of frame.

(3) The connection of column-beam is rigid connection.

(4) Among the steel frames, some are of 4 stories and others are of 6 stories. The story height is 3.0m.

(5) The number of spans and stories is same.

(6) The span length is of two types: 3.6m and 5.4m. For every single frame, the length of spans is same with no variation.

3.2 Section factors of columns and beams

Column section is determined as follows. In the load combinations with no seismic load, factor 0.75 is applied as the design margin while determining the column section area A_{c1}. In the seismic load combination, the pressure to capacity ration of column is set as 0.4, 0.3 and 0.2 while determining the column section area A_{c2}. The bigger area A_c out of A_{c1} and A_{c2} is selected as the column section area in the end.

In similar, factor 0.75 is applied as the design margin while determining the beam section area A_b.

With the Determined A_c and A_b, the H-section with flange and web of different width-to-thickness ratio is selected out to compose the steel frame. In total, 411 steel frames are formed with different sections.

3.3 Structure factors significantly impacting Rf (load reduction factor)

As per the investigation results in fore-issued papers [2] to [5], it is known that the following structure factors will significantly impact R_f (the load reduction factor):

(1) The width-to-thickness ration of section flange of columns and beams (r_f);

(2) The height-to-thickness ration of section web of columns and beams(r_w);

(3) The pressure-to-strength ration of column (n)

(4) The ratio of column linear stiffness to beam linear (r_k)

(5) The ratio of column strength to beam strength (r_s)

r_s is applied here to denote the ratio of column strength to beam strength:

$$r_s = \frac{M_{bc}}{M_{bb}} \tag{9}$$

Where M_{bc}, M_{bb} is the moment capacity of column section and beam section respectively with the sectional local buckling considered. M_{bc}, M_{bb} are obtained from reference [2].

4 Settings of push over analysis

4.1 setting of hinges

As aforementioned, each frame will be push-over analyzed twice. In the conventional analysis, the section yielding is simulated as a plastic hinge; In the local buckling analysis, the sectional local buckling is simulated as a local buckling hinge, whose parameters are from Reference [2].

4.2 The load pattern in the analysis

The vertical load is averagely distributed on beams and is kept unchanged during the push over analysis. In addition, the horizontal load is added to the floor point in pattern of inverted triangle. During the push over analysis process, horizontal displacement is controlled. The horizontal displacement is increased step by step until the analysis is ended without convergence or until the displacement reaches the aimed displacement that is set to be 1/20 of entire structure height.

4.3 The rules to determine yield displacement，yield strength and ultimate displacement

When the stiffness decreases to be 15% of the original stiffness, the corresponding displacement is deemed to be yield displacement Δ_y, and the load at this point is deemed as yield strength F_y. When the load decreases to 90% of the maximum load, the corresponding displacement is deemed to be the ultimate displacement Δ_u.

4.4 Push Over Analysis

For 411 frames, push over analysis are performed twice for each frame. For every time analysis, the displacement of first story Δ_1 and the base shear F are abstracted; then the F-Δ_1 curve is plotted. For each F-Δ_1 curve, the following factors are determined by using the aforementioned method and equations.

(1) Yield strength F_y、F_{yb} from twice push over analysis for same frame.

(2) Yield displacement Δ_y and ultimate displacement Δ_u, both are abstracted from the local buckling analysis.

By applying equation （7)，(3）and（8）, factor ξ (decrease factor of yield strength), factor μ (ductility factor) and R_f (load reduction factor) can all be obtained.

5 F-Δ_1 curve

Typical F-Δ_1 curves are shown in Figure 2. The blue curve is from the conventional analysis; and the orange curve is from the local buckling analysis. The factors of frame that produces curves in Figure 2 are as follows:

- width-to-thickness ratio of column flange: 8; width-to-thickness ratio of column web: 51
- width-to-thickness ratio of beam flange: 18; width-to-thickness ratio of beam web: 108
- pressure-to-strength ratio of column: 0.2
- 4 story; span length of 5.4m.

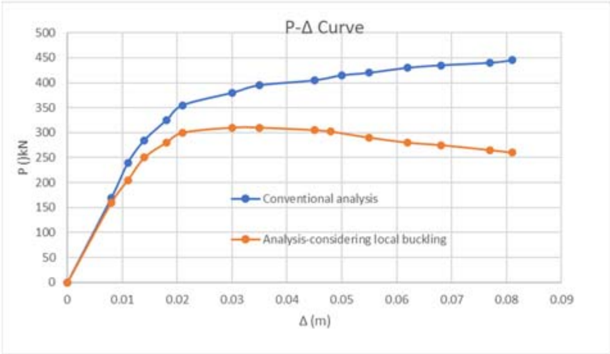

Fig 2. Typical F-Δ_1 curve

6 Formula Derivation of load reduction factor Rf

A new factor is introduced here to simplify the formula derivation:

$$r_{sk} = r_s \cdot r_k^{0.25} \qquad (10)$$

Where, r_s is ratio of column strength to beam strength and rk is ratio of column stiffness to beam stiffness. Therefore, r_{sk} represents the ratio of column to beam in terms of both strength and stiffness. Based on the R_f values out of all 411 frames, the relationship between the R_f factor and the critical structure factors is investigated and formula (11) thru (18) is derived. In the formulas, r_{sk} is the factor representing ratio of column to beam in terms of both strength and stiffness, r_{fb} is the width to thickness ratio of beam flange, n_s is the pressure to strength ratio of column.

6.1 For steel frames with span length 3.6m

$$r_s < 1.0: \quad R_f = 1.49 r_{sk} + 0.07 \qquad (11)$$

$$1.0 \le r_s < 2.2(1.5): R_f = (-0.17 r_{fb} + 3.78) \cdot r_{sk} + (0.11 r_{fb} - 1.17) \qquad (12)$$

$$2.2(1.5) \le r_s < 3.5: R_f = (0.16 n_s - 0.32) \cdot r_{sk} + (0.08 n_s - 0.83) \cdot r_{fb} + (-1.99 n_s + 16.98) \qquad (13)$$

$$r_s \ge 3.5: \quad R_f = 0.13 r_{sk} + (-0.32 r_{fb} + 5.73) \qquad (14)$$

6.2 For steel frames with span length 5.4m

$$r_s < 1.0: \quad R_f = 2.19 r_{sk} + 0.07 \qquad (15)$$

$$1.0 \le r_s < 2.2(1.5): \quad R_f = (-0.06 r_{fb} + 2.07) \cdot r_{sk} + (0.025 r_{fb} + 0.54) \qquad (16)$$

$$2.2(1.5) \le r_s < 3.5: \quad R_f = (0.22 n_s - 0.80) \cdot r_{sk} - 0.35 r_{fb} + (-0.88 n_s + 12.02) \qquad (17)$$

$$r_s \ge 3.5: \quad R_f = 0.08 r_{sk} + (-0.48 r_{fb} + 9.90) \qquad (18)$$

The followings shall be noted:

(1) For span length is between 3.6m and 5.4m, the interpolation is applied between the R_f values calculated using equations of 3.6m and equations of 5.4m separately.

(2) In Equation (11) thru (18), r_{fb} equals 13 when it is less than 13. R_f equals 6 when it is larger than 6.

(3) For r_s value in equations (12), (13), (16), (17), the value outside brackets (2.2) is for 4-story structure, value inside brackets (1.5) is for 6-story structure. For 5-story structure, use interpolation between 1.5 and 2.2; for 3-story or lower structures, take the same value as 4-story structure.

7 Verification of formulas by the test structure

In reference [4], two full scale frames are tested. the loads are added in same way as push over analysis. Based on the *P-Δ* curves out of the specimen, the reduction factor of strength (R_f^a) is calculated. In parallel, the reduction factor of strength (Rf) is calculated using formula (11) through (18). R_f^a and Rf are compared as shown in Table 1. From comparison, it can be seen that formula (11) through (18) are reliable for calculating the load reduction factor R_f.

Table1. Comparison of Load Reduction Factor Rf

	ξ	μ	$\xi^2\mu$	Rf^a	L_{sp} （m）	r_{fb}	r_s	r_k	r_{sk}	Rf
Test Frame1	0.85	0.94	0.67	1.63	3.9	8.9	0.82	1.68	0.96	1.61
Test Frame2	0.78	2.09	1.26	2.25	3.6	19.3	1.71	2.68	2.18	1.91

8 Determination of horizontal design seismic load

For steel frames with expanded sections, the load reduction factor R_f is calculated using equation (11) thru (18), its horizontal seismic design load (F_{yb}) is determined as below: $F_{yb} = F_e / R_f$

Where, F_e is the seismic design load of elastic structure, which is same as the seismic input according to high earthquake.

For conventional structure, As aforementioned, R_f is 6, so its seismic design load is $F_y= F_e /6$.

Using F_y to denote F_{yb}: $F_{yb} = 6 F_y / R_f$.

Take an example, for common structures in Shanghai, the horizontal seismic design seismic load is:

F_y = 0.08×G (G is the represented gravity load of structure).

So $F_e = 6F_y = 6×0.08G=0.48G$. Where, F_e is the seismic design load of elastic structure, the same as the seismic input according to high earthquake.

When designing steel structures of expanded-section beams or columns, the first step is to calculate the load reduction factor R_f using equation (11) thru (18). Assuming the calculated R_f is 2. Then, its horizontal seismic design load F_{yb} can be calculated:

$F_{yb} = F_e / R_f$ = 0.48×G/2=0. 24G (G is R_f the represented gravity load of structure)

From comparison, it is known that the horizontal seismic design load of expanded-section structures (F_{yb}) is 3 times the design load of common structures of compact sections (F_y).

9 Conclusion

The load reduction factor R_f of steel frame is investigated by analyzing 411 frames, which are composed of expanded-section columns and beams. The resulted load reduction factors R_f are investigated in terms of the section factors and the member factors. Formulas are derived to calculate R_f based on the structure factors. To verify the formula, the R_f values out of the full-scale test structures are compared to the results out of formulas, and the comparison proves that the formulas are reliable. In the end, how to determine the horizontal seismic design load using the load reduction factor R_f is described. It can be seen that the design load can be easily determined with the calculated load reduction factor R_f for the expanded-section steel structures.

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