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Buckling mechanism of steel core and global stability design method for fixed-end buckling-restrained braces

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ABSTRACT

Based on theoretical analyses, the development of buckling mode of steel core is revealed for buckling restrained braces (BRBs) with fixed ends. The formulae are derived for the bending moments of the stiffening part of the core and restraining member of BRBs. Then a global stability design method of fixed-end BRBs is proposed. The results of the contact force, flexural response and buckling half wavelength with finite-element analysis agree with the theoretical analysis. The design criteria as well as the buckling half wavelength were validated by the quasi-static tests of five BRBs. Compared to existing methods, the proposed design approach can ensure a proper safety margin of the stiffening part.

1. Introduction

Buckling-restrained braces (BRBs) can be designed to sustain yielding under both compression and tension without significant buckling. Therefore, BRBs have been widely used to dissipate seismic energy in seismic regions. The steel core of a BRB is encased in a restraining member to dissipate energy, and the restraining member is designed to prevent the core from failing following an earthquake [1–9]. Global stability of BRBs has been one of the most concerned issues and has attracted wide attention from many researchers in recent years [10–21].

Shimizu et al. [11,12] proposed a widely used global stability design criterion based on the analytical model of BRBs ignoring the effect of boundary condition of the steel core. The design formula has been adopted by Japanese and Chinese standards [13,14], and it was also widely accepted by researchers and engineers [15–17]. However, the flexural demands might be different caused by various connections, i.e., bolted, welded and pinned connections. Tests [11] revealed that the additional flexural demand at the end of BRBs would be caused by the in-plane rotation of joint panel for bolt-connected BRBs. Even more, this rotation of joint panel would also lead to an additional bending moment on the restraining member, which was validated by Tremblay's substructure testing [18]. The boundary constraint at both ends of pin-connected BRBs may be like that of hinged ends because of the isolated bending moment from the gusset plates by pins, and the ones of bolt-connected and weld-connected BRBs should be closer to those of fixed ends due to the strong constraint.

Based on the analytical model of hinged-end BRBs, Zhao et al. [19–21] studied the critical influences of the flexural demand on both of the stiffening part and the restraining member for the pin-connected BRBs, and Wu et al. [22] proposed a global stability design criterion of BRBs considering the stiffening part of the steel core and the effect of friction; Based on the analytical model of fixed-end BRBs, Usami et al. [23] established a continuous fixed-end flexural bar model with a non-uniform cross-section for the weld-connected BRBs. Although the global stability design method based on the former analytical model might be used for the bolt-connected or weld-connected BRBs, it is still not sure that regarding the bolted or welded BRBs as hinged-end BRBs will lead to a more conservative design result than that corresponding to the BRBs with fixed ends.

This paper first investigates the flexural demands of bolt-connected and weld-connected BRBs based on the analytical model of fixed-end BRBs, including the restraining member and the stiffening part. A numerical validation with finite element analysis follows. Finally, five quasi-static test results are used to verify the formulae proposed in this paper.

2. Flexural demands of BRBs with fixed ends

The analytical fixed-end model is shown in Fig. 1, where δ_0 denotes

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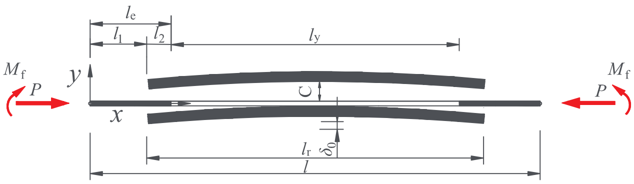

Fig. 1. Analytical model of BRB with fixed ends.

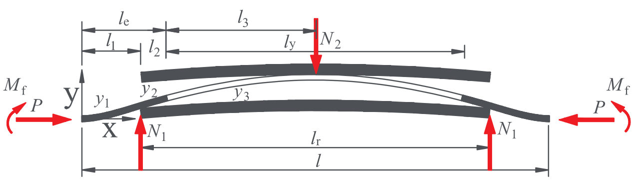

Fig. 2. Analytical model of BRB with contact force.

the initial deflection of the restraining part, C is the total clearance between the steel core and the restraining member, l is the total length of the BRB, l_r and l_y are the lengths of the restraining member and yielding segment, respectively, l_e is the stiffening part of the steel core, l_1 is the stiffening part outside the restraining member, l_2 is the stiffening part inside the restraining member, P is the axial load, and M_f is the fixed-end bending moment of steel core.

2.1. Lower mode

The steel core will buckle and the contact forces N_1 and N_2 will be generated as the axial load increases, as shown in Fig. 2. According to lateral equilibrium, N_1 is half of N_2, and the moment equilibrium equation of one half of the core may be expressed as follows:

$$E_1I_1y_1'' + Py_1 = M_f \quad (0 \leqslant x < l_1) \tag{1}$$

$$E_1I_1y_2'' + Py_2 = N_1(x-l_1) + M_f \quad (l_1 \leqslant x < l_e) \tag{2}$$

$$E_2I_2y_3'' + Py_3 = N_1(x-l_1) + M_f \quad (l_e \leqslant x < l/2) \tag{3}$$

and the moment equilibrium equation for the restraining member is expressed as

$$E_rI_ry_{r1}'' = -N_1 \quad (x-l_1)(l_1 \leqslant x \leqslant l/2) \tag{4}$$

The corresponding boundary conditions include the zero rotation of the steel core located at the left end of the l_1 segment and that located at the right end of the l_3 segment, and the continuities of the displacement and rotation between the segments. Thus, the expressions of deflection of the steel core can be obtained as

$$y_1 = -\frac{M_f}{P}\cos k_1x + \frac{M_f}{P} \tag{5}$$

$$y_2 = B_1\cos k_1x + B_2\sin k_1x + \frac{N}{P}x - \frac{Nl_1}{P} + \frac{M_f}{P} \tag{6}$$

$$y_3 = C_1\cos k_2x + C_2\sin k_2x + \frac{N}{P}x - \frac{Nl_1}{P} + \frac{M_f}{P} \tag{7}$$

where $k_1 = \sqrt{\frac{P}{E_1I_1}}$, $k_2 = \sqrt{\frac{P}{E_2I_2}}$, E_1, E_2 and E_r denote the elastic moduli of the stiffening part, yielding segment and restraining member, respectively. I_1, I_2 and I_r denote the inertia moments of the stiffening part, yielding part and restraining member. The coefficients of y_2 and y_3 can be expressed as

$$B_1 = \frac{N}{Pk_1}\sin k_1l_1 - \frac{M_f}{P} \tag{8}$$

$$B_2 = -\frac{N}{Pk_1}\cos k_1l_1 \tag{9}$$

$$C_1 = \frac{N\sin\frac{k_2l}{2}}{Pk_2} - \frac{N}{P}(l_2+l_3)\cos\frac{k_2l}{2} - \frac{M_f}{P}\cos k_1l_1\cos\frac{k_2l}{2} + C\cos\frac{k_2l}{2} + \frac{N_1l_r^3}{24E_rI_r}\cos\frac{k_2l}{2} + \delta_0\cos\frac{k_2l}{2} \tag{10}$$

$$C_2 = -\frac{N\cos\frac{k_2l}{2}}{Pk_2} - \frac{N}{P}(l_2+l_3)\sin\frac{k_2l}{2} - \frac{M_f}{P}\cos k_1l_1\sin\frac{k_2l}{2} + C\sin\frac{k_2l}{2} + \frac{N_1l_r^3}{24E_rI_r}\sin\frac{k_2l}{2} + \delta_0\sin\frac{k_2l}{2} \tag{11}$$

where l_3 denotes the distance from the contact point to the end of the stiffening part as shown in Fig. 2. Then the contact force N_1 can be obtained as

$$N_1 = \frac{-Pk_2(C+\delta_0)\sin k_2l_3\cos k_1l_e - Pk_1(C+\delta_0)\cos k_2l_3\sin k_1l_e}{\left(\begin{array}{l}\frac{k_2}{k_1}\cos k_1l_1\sin k_2l_3\sin k_1l_2 + \frac{k_1}{k_2}\sin k_2l_3\sin k_1l_e + 2\cos k_1l_1 \\ -\cos k_2l_3\cos k_1l_e \\ -\cos k_1l_2\cos k_1l_1\cos k_2l_3 - k_1(l_2+l_3)\cos k_2l_3\sin k_1l_e \\ -k_2(l_2+l_3)\sin k_2l_3\cos k_1l_e \\ +\frac{Pl_r^3}{24E_rI_r}(k_1\cos k_2l_3\sin k_1l_e + k_2\sin k_2l_3\cos k_1l_e)\end{array}\right)} \tag{12}$$

and the fixed-end bending moment of the steel core can be expressed as

$$M_f = \frac{N_1\left(\frac{\sin k_1l_2}{k_1} + \frac{\sin k_2l_3}{k_2} - (l_2+l_3)\cos k_2l_3 + \frac{Pl_r^3}{24E_rI_r}\cos k_2l_3\right) + P(\delta_0+C)\cos k_2l_3}{(\cos k_1l_1\cos k_2l_3 - \cos k_1l_e)} \tag{13}$$

Based on Eq. (12), the maximum bending moment of the restraining member at the center is obtained as

$$M_r = N_1(l_2+l_3) \tag{14}$$

Based on moment equilibrium at the contact point and Eq. (5), the bending moment of the stiffening part at the end contact point can be calculated as

$$M_s = -M_f\cos k_1l_1 \tag{15}$$

It is seen from the above equation that the fixed-end bending moment of the steel core, M_f, is always greater than that at the contact point, so the critical cross-section of the stiffening part is located at the end of the steel core for fixed-end BRBs, rather than the contact point for BRBs with hinged ends [21,22].

For hinged-end BRBs, the curvature of the steel core at the central contact point reaching zero is the critical condition from one contact point to two contact points [22]. However, for the BRBs with fixed ends, the curvature at the end of the steel core might also be equal to zero, therefore it is yet to know whether or not the development of buckling mode of the steel core herein coincides with that of BRBs with hinged ends. A numerical example of BRB with fixed ends is given below to investigate the development of buckling mode on steel core. Assume a BRB with $l = 2000$ mm, $l_r = 1280$ mm, $l_y = 1035$ mm, $l_1 = 360$ mm, $l_2 = 122.5$ mm, $C = 2$ mm, $\delta_0 = 0$, $I_1 = 435833$ mm^4, $I_2 = 6667$ mm^4, and $E_1 = E_2 = 2.1 \times 10^5$ N/mm^2, and the restraining member is assumed a rigid body. Based on Eqs. (7), (12), and (13), the bending moment and curvature at the center of the core and those at the end of the core are computed and shown in Fig. 3 and Fig. 4, respectively.

Fig. 3 shows that the bending moment and curvature at the center of the core are always greater than zero in contrast with those of BRBs with hinged ends [22]. However, as shown in Fig. 4, the bending moment and curvature at the end of steel core decrease as the axial load increases, and drop to zero when the axial load reaches 112 kN. This implies that the development of buckling mode for BRBs with fixed ends should be different to that with hinged ends.