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Investigating the nonlinear behavior of Eccentrically Braced Frame with vertical shear links (V-EBF)

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ABSTRACT

The excellent hysteretic behavior of Eccentrically Braced Frames (EBFs) using shear links have made these systems an effective alternative for both moment resisting frame as well as concentrically braced structures. In common practical horizontal shear links are located either at the middle or at the ends of the beam section. However, large deformation of the shear links in those beam must be accepted under severe earthquakes. In those instance in which the beam has to remain elastic, vertical shear links should be designed underneath the beam thereby transferring the region of plastic deformation to location where they are tolerable and post-earthquake repair or replacement of damaged parts is easier. In this paper, seismic behavior of Vertical-EBF (V-EBF) have been considered experimentally and analytically. The required relation to design of the system have been proposed. Also, to predict nonlinear behavior of V-EBF systems, an analytical model including Kinematic-Isotropic strain hardening for shear, only kinematic strain hardening for moment, has been presented. Results showed a good convergence of the presented analytical model (with an upper bound for yielding surface to the V-EBF) with experimental results.

1. Introduction

Eccentrically Braced Frames (EBFs) have been used as a seismic load resisting system, primarily in buildings. The system, which relies on the yielding of a horizontal link beam between eccentric braces, has been shown to provide ductility and energy dissipation under seismic loading, and its behavior in various configurations has been investigated [1–12]. In other hand, the EBF systems are metallic passive energy dampers under seismic loading.

Despite high seismic energy dissipation, Horizontal EBF (H-EBF) has substantial disadvantages. It is no simple replace or repair of link beam in the H-EBF after a severe earthquake. This problem is the main disadvantage of this system. In addition, in industrial structures such as power plants, deep beams are sometimes used to carry the loads of heavy and highly sensitive equipment. In these cases, to assure ensure that equipment works accuracy, load-bearing members like beams and columns should remain in the elastic range as much as possible. Therefore, the use of horizontal link beams is not convenient. To overcome the problems in the EBF system, a new system called the V-EBF (Vertical-EBF) system was proposed [13–16]. In this system, reversed-V braces are attached to the story beam with a shear panel. Intensive inelastic deformations are localized in the shear panel and the internal energy is dissipated by this member. Since all inelastic deformation is localized in the shear panel, there is no damage to the main members, also repairing after a strong earthquake are easier than in H-EBF systems. Using the vertical links for seismic rehabilitation of the existing buildings is possible with minor changes in the main structure [17]. In other word, after a severe earthquake vertical links could be easily replaced so that the structure could be repaired in a very efficacy way. Experimental results showed that inelastic deformation is confined in vertical link and its performance governs on the system in all specimens. This occurrence proves that the vertical shear link act as ductile fuse to absorb energy. The shear panel system is one of the simplest and cheapest passive energy dampers. By using the vertical links, in the dynamic analysis, most of the story shear force could be transferred and resisted through the vertical shear link element. Also, uniform energy dissipation was a very desirable feature, as it indicated that the shear link elements were deforming plastically, and no element was excessively strained as evidence by observations and investigations of all the other elements [18–20]. The seismic performance improvement of bridges and towers with shear panel systems was investigated [21–23].

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Because of the shear panel effect on the behavior of V-EBF braces as a lateral load resisting system, improvement of shear panel performance is very important. Several studies have been conducted on this system to prevent buckling the braces [24–27].

In the present study, the nonlinear behavior of the V-EBF system was evaluated experimentally and theoretically. Referring to researcher's reports (desirable performance of link in shear yielding) the short link are evaluated. Therefore, mathematical models are formulated to provide elastic and inelastic behavior predictions. According to the experimental results the require suggestion are detailed and addressed for designing of vertical shear link. Moreover, the nonlinear behavior of V-EBF system under earthquake acceleration is investigated in various configurations.

The link is designed to act as a ductile fuse by yielding and dissipating energy while preventing buckling of the brace members.

2. Design of shear link

2.1. Horizontal shear link

The link is designed to act as a fuse by yielding and dissipating energy while preventing buckling of the brace members. The inelastic response of a link is strongly influenced by the link length and the Mp/Vp ratio of the link cross-section. Using plastic analysis, the plastic shear strength, Vp, and plastic moment strength, Mp, can be written as:

$$Vp = \frac{F_{yw}}{\sqrt{3}}.\ t_w(d - 2t_f) \tag{1}$$

$$Mp = F_{yf}t_f(b - tw).\ (d - t_f) + \frac{F_{yw}t_w d^2}{4} \tag{2}$$

where F_{yw}, F_{yf} are the web and flange yield strength, respectively, t_w is the web thickness, d is the overall beam depth, and t_f is the flange thickness, t_w is the web thickness, and b is the flange width.

Previous investigations [1–12] have shown that horizontal beam shear-link possess excellent ductile, energy-dissipating behavior, provided the link length is designed to allow shear yielding without buckling of the web. In fact high isotropic strain hardening effects and combined bending and shear could be observed. They lead to increasing plastic-moment and shear capacities. In the ultimate state the shear and bending moment capacities reached values of about 1.5 Vp and 1.2 Mp, respectively. In order to impose shear yielding before bending yielding of the horizontal link, the link length e needs to be limited [9,22] to:

$$e \le \frac{2 \times 1.\ 2M_p}{1.\ 5V_p} = 1.\ 6\frac{M_p}{V_p} \tag{3}$$

In other word, classification is based on the normalized link length, ρ, defined as; $\rho = \frac{Vp.e}{Mp}$.

Capacity design approach is followed in an attempt to limit the inelastic activity to the shear links only while all other frame members are designed to behave elastically.

According to AISC Seismic Provision [23] the links with $e \le 1.\ 6\frac{M_p}{V_p}$ or ρ≤1. 6 are shear links that yield predominantly in shear and have a maximum link rotation under the design seismic loading of 0.08 rad. The drift angle of the frame, θ, can then be written in terms of the link rotation angle, ɣ, as (See Fig. 1):

$$\theta_p = \gamma_p\frac{e}{L} \tag{4}$$

There are differences between the horizontal and vertical shear links. So, the equations about the calculation of the link rotation should be modified.

Also, when the link with length e deforms inelastically and resists the applied base shear, the framing outside the link is designed to remain elastic. The elements outside the link are designed to resist the forces generated by the fully yielded and strain hardened link. For short links (ρ≤1.6), the generated forces can be calculated as [28]:

Link shear =1.25 R_y V_p,

Link end moment at beam = R_y V_p,

Link end moment at brace =[1.25 R_y V_p. e -R_y M_p] ≥ 0.75 R_y M_p.

Where Ry is the ratio of the expected yield strength to the minimum specified yield strength Fy prescribed in AISC Seismic Provisions. This ratio is used to account for possible material overstrength. The shear strength is further increased by 1.25 to account for strain hardening. Based on results of 16 links made from A992 steel, the average strain hardening ratio was formed to be 1.28, with a variation ranging from 1.17 to 1.44 [29]. Past researchers [9] have generally recommended a link overstrength factor of 1.5.

In order the elements outside the link are designed to resist the forces generated by the fully yielded and strain hardened link.

In the next parts, the equation to design of the vertical shear link and structural elements outside the shear link are derived from experimental test and numerical results. Therefore, the generated forces factor is modified according to strain hardening results of vertical shear link. In order to design the V-EBF, an analytical model is necessary.

2.2. Vertical shear link

In order, it attempts to design vertical shear links according to horizontal links using some modification factors. It will be presented that in the vertical links, the ultimate state of the shear and bending moment capacities reached values different from 1.5 Vp and 1.2 Mp for horizontal shear link.

Therefore, the link length limitation and generated factor for designing other element outside the vertical link should be modified.

Besides, an analytical model for vertical link is necessary to predict the V-EBF behavior. In order to compare the experimental results with numerically derived values, it is necessary to develop an analytical model to determine the cyclic response of the V-EBF. This analytical model must be able to account the nonlinear cyclic response of the plastificated element.

3. Analytical model of inelastic shear link element

3.1. General

Most of the inelastic behavior of an EBF is confined to the shear link element. Since the active links (horizontal or vertical) are subjected to large shear forces and bending moment, any appropriate formulation should include shear and bending effects at the elastic and inelastic states. An accurate shear link model must be able to consider shear or moment plastification with strain hardening at any time of the loading. In the analysis of links performance, elastic and inelastic deformations of both the shear and flexural behaviors have to be taken into consideration.

Few researchers attempted to develop link models for the dynamic inelastic analysis of EBFs. They modeled the link as two dimensional linear beam-column element with inelastic hinges at each ends with six nonlinear rotational and translational springs at each ends. Three rotational and translational multilinear springs were used to represent the flexural and shear inelastic behavior of the plastic hinge at the link ends represented by the multilinear function [10,30].

3.2. Cyclic inelastic shear link model

The link element is modeled as two dimensional beam-column element with inelastic hinges at each ends. Each hinge has zero length and consists of three subhinges. Each subhinge has a rigid plastic force deformation and moment rotation relationship. Beside, it has yield