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Evaluation on eccentrically braced frame with single and double shear panels

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ABSTRACT

In the present study, Eccentrically Braced Frames' (EBF) seismic behavior with single and double shear panels is introduced. The numerical modeling is done by commercial FE software (Abaqus) under two kinds of cyclic displacement loads (including pushover), to evaluate shear links' behavior. The effects of concrete slab are studied on overall frame behavior, and the ductility coefficient increase for double shear panel models is investigated. The models were exposed to time-history analysis via scaled earthquake records, to study the models' energy parameters. Imparting acceleration on the structure and carrying out time history analysis, the double shear panel system revealed a higher tendency to form shear links, causing greater dissipation capacity in the plastic zone. In pushover and time history analyses, it was demonstrated that concrete slab can have a destructive impact on frame and shear link behavior.

1. Introduction

Extensive researches were conducted in the early 70′s on EBFs. These braced frames are widely applied in seismic force resisting systems. EBF is a braced-frame; one end of the brace is connected to the beam to provide a good performance in case of strong seismic activity, and the other end is connected to a column. Fig. 1 displays an EBF configuration. It acts such as a seismic fuse in which shear forces and flexure are transferred to the frame. Rossi et al. [1] studied the effect of the link over strength on the seismic behavior of EBF designed according to capacity design principles, concluding that EBFs seismic response is generally influenced by the link over strength factors and dynamic features of seismic ground motions. Taking into account the seismic performance of the link, M Bosco et al. [2] performed the seismic performance of EBF link-to-column links, concluding that regardless of the connection length, the fracture most possible for link-to-column connections is near the groove at the link flange. Similarly, the controlled mode of failure was investigated on a global type of eccentrically braced frames. Mastrandrea and Piluso [3] introduced design mythology to study the collapse mechanism. The assumption was based on the fact that link and beam sections are known; hence, the kinematic theorem of plastic collapse was used to describe column and diagonal parts. Applying static and dynamic non-linear analyses, the global type is found to be often the collapse mechanism, corresponding to the proposed design method. Thus, this approach can be used in structures' earthquake-resistant design. Most inelastic dynamic analyses have emphasized the considerable effect of the link over strength parameter on the seismic behavior of traditional eccentrically braced structures [4,5].

Assuming the application of the aforementioned link, Naghipour et al. [6] studied the Reduced Beam Section (RBS) role connection in long links; yielding occurrence could be put off in the link (before the moment at central location of RBS at the column's face is reached) about 10% higher than expected plastic capacity. Also, RBS keeps the plastic hinge away from the link-to-column connection. With respect to the early 1990′s, Engelhardt and Popov [7] evaluated the performance of these links, concluding that long links attached to columns must be avoided in EBFs. Years later, researchers studied the influence of link length on EBFs' performance [8–10], showing that the ductility, strength, stiffness, and energy absorption capacity are less for long links than the short links. Daneshmand and Hashemi [11] studied 68 links by finite element analysis to assess the main behavior of intermediate and long links in EBFs, showing that the required rotation capacity was not obtained for some intermediate links. Zahrai et al. [12] evaluated the hysteretic behavior of eccentrically braced frames with zipper-strut upgrade, revealing that the zipper-strut-equipped

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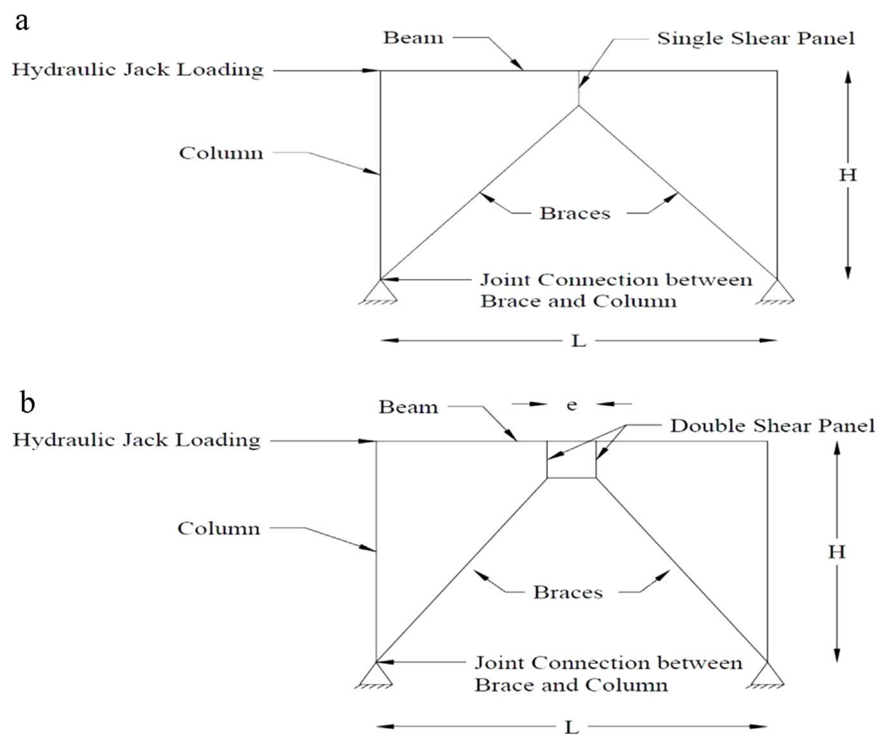

Fig. 1. Eccentrically braced frame a) with single shear panel b) double shear panels.

system showed a stronger tendency to form shear links, causing higher dissipation capacity in the plastic zone. Plastic mechanism control theory was carried out for eccentrically braced frames with inverted y-scheme by Montuori et al. [13]. The plastic mechanism control theory consists of the impact of second order effects using the concept of mechanism equilibrium curve. The design requirements are obtained by kinematic theorem of plastic collapse extended to the mechanism equilibrium curve concept. Column sections are derived by mechanism equilibrium curve matching to the global mechanism below those matching to all undesired mechanisms in a displacement range compatible with the local ductility supply. In addition Montuori et al. [14] provided a complete and exhaustive design procedure for MRF–EBF dual systems, while considering all the brace configurations commonly adopted, with the goal of assuring the development of a collapse mechanism of global type.

Recent studies [15–17] and AISC [18] have assumed stability a crucial factor for all long and intermediate links because these links, in the beam outside the link, produce high shear and bending moments. The current study is to consider the EBF system with single shear panel and EBF system with double shear panels considering energy dissipation, ductility, stiffness, frame shear capacity, and plastic section situation. Moreover, it should be mentioned that those frames assume both, with and without concrete slab. Fig. 1 displays the models with single shear panel and with double shear panels.

According to regulations; such as AISC2002 and IBC2003 [27], shear link should be limited to Eq. (1) to reach shear failure:

$$e \leq \frac{1.6 \quad M_p}{V_p} \tag{1}$$

where e is shear capacity, M_p is beam plastic moment and V_p is vertical shear link length.

However, it has been suggested by other researchers [34] that shear link should be limited to Eq. (2):

$$e \leq \frac{1.4 \quad M_p}{V_p} \tag{2}$$

Boukamp and Vetr stated that regarding the top and the bottom rotational stiffness ratio of vertical shear link, it is recommended to limit shear link length to Eq. (3):

$$e \leq \frac{0.7(k+1) \quad M_p}{V_p} \tag{3}$$

where k calculates as:

$$k = \frac{M_{bottom}}{M_{top}} \tag{4}$$

One of the important parameters for designing vertical shear link, is taking into consideration the importance of shear distortion of the vertical link. According to AISC2002, shear distortion should be limited to 0.08 and 0.02 rad for the vertical links with length longer than $\frac{2.6M_p}{V_p}$ and shorter than $\frac{1.6M_p}{V_p}$ respectively [18]. These magnitudes are limited to 0.09 and 0.03 according to IBC2003. Thus, link web should dissipate earthquake energy using its plastic capacity. Furthermore, other structure members; such as the column, the beam, and also the braces should remain elastic. To properly fulfill its purpose, the braces compressive strength should be more than 1.5 axial forces of shear link. Furthermore, in order to safeguard columns, braces and beams from the undesired effect of real values of the yield strength, the aforementioned ultimate internal forces of links have been increased by means of a factor γ_{ov} (called material over strength factor) equal to 1.20. So, starting from such considerations, the design axial forces of the braces have been evaluated by the vertical equilibrium of the brace-to-link joint. As a result of this equilibrium, the vertical component of the design axial force of the generic brace ultimately balances the final design shear forces of links (i.e. the ultimate shear force times the material over strength factor) and the design shear force in the beam segment outside of the same link. Therefore, the design axial force of braces may be calculated as [1]:

$$N_i^d = \frac{\gamma_{ov} V_{ui}}{\sin\alpha} \left(1 + \frac{e}{2l_b}\right) \tag{5}$$