Investigation on Eccentrically Braced Frames with Tubular Links Using Non-Linear Time History Analysis

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SUMMARY:
Eccentrically Braced Frames (EBFs) have been shown to exhibit excellent seismic performance. Recently, interest in the use of EBFs and also energy dissipation systems with Wide Flange (WF) or I-shaped links has been increased in bridge piers or towers. Typically the links have had a wide-flange or I-shaped cross-section that requires lateral bracings to prevent lateral torsional buckling. The link is self-stabilizing and does not require lateral bracing, making it suitable for using in steel bridge piers where lateral bracing can be difficult to provide. This subject has limited the use of EBFs in bridge piers, since lateral bracings are difficult to provide in those situations. This paper describes an analytical investigation into the use of members with hollow rectangular cross-sections as EBF links which do not require lateral bracings. Finally, a finite element model of the frame is developed using shell elements. Time history analysis has been done to show the good behaviour under three specified earthquakes. To show the good seismic behaviour of the frames non-linear time history analysis has been performed and the results show the expected good behaviour of this new bracing under earthquake.

Keywords: Eccentrically Braced Frame, energy dissipation, buckling, ductility.

1. GENERAL INSTRUCTIONS

EBFs have been primarily used as seismic load resisting systems in buildings for more than a decade. Typically the links have had a wide-flange or I-shaped cross-section that requires lateral bracings to prevent lateral torsional buckling. The link is self-stabilizing and does not require lateral bracing, making it suitable for using in steel bridge piers where lateral bracing can be difficult to provide (building applications are possible as well). This subject has limited the use of EBFs in bridge piers, since lateral bracings are difficult to provide in those situations. Links of this type would also be useful in situations in buildings or other structures where lateral bracing may not be feasible or easily provided. The link beam which has a hybrid tubular cross-section composed of webs and flanges of different thicknesses. Experimental results indicate that the link beam reach a rotation of 0.15 radian, almost twice the current limit of 0.08 radian for wide-flange links and prior to suffering flange fracture (Almost twice the maximum allowed in building codes for I-shaped links). Providing a laterally stable link for using in bridge piers which prevent lateral torsional buckling, has been developed hollow rectangular cross sections such a tubular link which are able to achieve the maximum rotation level presented in the AISC Seismic Provisions for links with WF cross sections. The achieved large rotations are more than the required rotation in a seismic event, indicating that hybrid rectangular links without lateral bracing can indeed be a viable alternative for using in steel footbridges placed in seismic regions. This paper describes an analytical investigation into the use of members with hollow rectangular cross-sections as EBF links which do not require lateral bracings (Berman J.W, et al. 2007; Berman J.W, et al. 2008; Berman J. W., et al. 2006), . Finally, a finite element model of the frame is developed using shell elements, and reasonable agreement with the experimental results is observed. Time history analysis has been done to show the good behaviour under specified earthquakes.

In the late seventies (Roeder CW et al. 1978) Eccentrically Braced Frames (EBFs) were proposed as basic elements of structural typologies which are able to satisfy the objectives of the modern seismic design philosophy at a moderate expense (M. Bosco, et al., 2009). Eccentrically braced frames (EBFs) have been used for more than a decade as a seismic load resisting system, primarily in buildings. This system, which relies on the yielding of a link beam between eccentric braces, has been shown to provide suitable ductility and energy dissipation under seismic loading. Its behaviour in various configurations has been investigated (Berman J.W, et al. 2007).

Guidelines for the EBFs design with links having wide-flange (WF) cross-sections are established in the AISC Seismic Provisions for Structural Steel Buildings (2005). However, the use of WF shapes as link beams necessitates that they be braced out-of-plane to prevent lateral torsional buckling. This requirement has limited their use in bridge piers where lateral bracing is difficult to be provided (Berman J.W, et al. 2008). Recently, interest in the use of EBFs or energy dissipation systems with WF or I-shaped links has been increased in bridge piers or towers; such systems have been designed, tested, and implemented for the San Francisco-Oakland Bay Bridge and the Richmond-San Rafael Bridge. In these cases, special considerations for link stability were made that increased the cost of the project. Therefore, it seems that the development of a link type that does not require lateral bracing is desirable for application of EBFs in bridge piers (Berman J.W, et al. 2007). Eccentrically braced frames with tubular links were suggested by Berman and Bruneau in the first decade of the present century (Berman J.W, et al. 2008; Berman J.W, et al. 2007). Furthermore, the design of EBFs to protect existing pier bracing members may be employed using the approach in Berman and Bruneau. Additionally the excellent seismic behaviour reported in the experimental works, high energy dissipation and preventing lateral torsional buckling by this new type of EBFs (Berman J.W, et al. 2008; Berman J.W, et al. 2007) produced much enthusiasm within the scientific community.

Such self-stabilizing links would also be useful in situations in buildings or other structures where lateral bracing may not be feasible or easily provided (Berman J.W, et al. 2008; Berman J.W, et al. 2007). Tube shapes have substantial torsional stability, making them less susceptible to lateral torsional buckling, and may thus not require lateral bracing (Berman J.W, et al. 2007).
Although Eccentrically braced frames have been shown to exhibit excellent seismic performance, eccentrically braced frames have had limited use in the steel piers of bridges due to the difficulty of providing the lateral bracing required to prevent possibility of lateral torsional buckling of the link. An eccentrically braced frame system in which lateral bracing of the link is not necessary would make it desirable in the context of seismic design and retrofit of bridges, especially since eccentrically braced frames have been shown to exhibit excellent seismic performance (Roeder, C.W., et al. 1977; Berman J. W., et al. 2006). From this point of view , the concept of an EBF utilizing a rectangular hybrid cross-section for the link is explored (hybrid in this case means that yield stresses of the webs and flanges are different). Rectangular cross-sections inherently have more torsional stability than I-shaped cross-sections and may not require lateral bracing.
First, link design is discussed in terms of plastic shear force and moment, normalized link length and compactness ratios and stiffener requirements are provided. Then, the specimen and test setup are described. Experimental work on this type of EBFs is initially done by Berman and Bruneau in 2007. They discussed and compared the experimental results with the results of finite element modeling of the link. They focused on the behaviour the EBFs with tubular links under cyclic loading (Berman J. W., et al.2004; Berman J. W., et al. 2006). This paper highlights the behaviour of link beams with hollow rectangular cross-sections for EBFs using time history analysis with three records of earthquakes. For this purpose, EBFs with rectangular links with various cross-sectional properties and lengths are analysed using the finite element software.

2. NON-LINEAR TIME HISTORY ANALYSIS PROCEDURES

In dynamic analysis procedures, the horizontal seismic load is determined from the dynamic response of building subjected to an appropriate ground motion. The acceleration time histories representing the ground motion effects shall reflect the expected earthquake at the site. According to the Iranian Code of Practice for Seismic Resistant Design of Buildings (Standard No. 2800, 3rd edition) in this paper three pairs of appropriate horizontal components are used. The below factors are regarded for choosing the records:

a) All magnitudes, fault distances and source mechanisms are consistent with those of the design-basis earthquake.

b) All records belong to the sites with same geology and tectonic and particularly soil characteristics. Based on soil site classifications, the soil type is Type B.

c) The time duration of the strong ground motion accelerograms is at least 10 seconds which satisfies the requirements of the Standard No. 2800.

2.1. Scaling the records

As mentioned before to satisfy the requirements of Iranian Code of Practice for Seismic Resistant Design of Buildings, three pair of accelerograms are used; Chi-Chi earthquake (Taiwan; 1999/09/20) with a magnitude of 7.6 on the Richter scale; Northridge earthquake (California, United States, 1994/01/17) with a magnitude of 6.7 on the Richter scale and finally Victoria (Mexico; 1980/06/09) with a magnitude of 6.4 on the Richter scale. PGA values of these records are 0.712g, 0.563g and 0.623g respectively.
All records are scaled based on Standard No. 2800 using the following procedure:

a) Each record is normalized to its maximum acceleration (PGA), so its peak value is equal to the gravity acceleration, 1g.

b) For each pair, the 5 percent-damped response spectra are calculated and combined, using the square root of the sum of the squares (SRSS) , so that a unique spectrum is constructed for each pairs of records.

c) The motions are scaled such that the average value of their SRSS spectra does not fall below 1.4 times the Standard Design-Spectra for periods of 0.2T seconds to 1.5T seconds, where T is the fundamental period of vibration in this paper, as expressed in Eqn. 2.1:

$$T=0.08H^{3/4} \tag{2.1}$$

d) The resulting scale factor is applied to the normalized records. They are used for non-linear time history analyses.
The finalized records are shown in Figs. 2.1, 2.3 and 2.3.

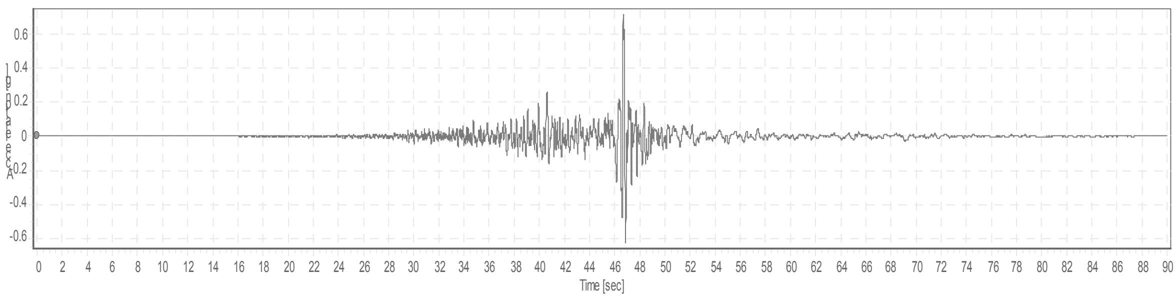

Figure 2.1. Time history of Chi Chi record

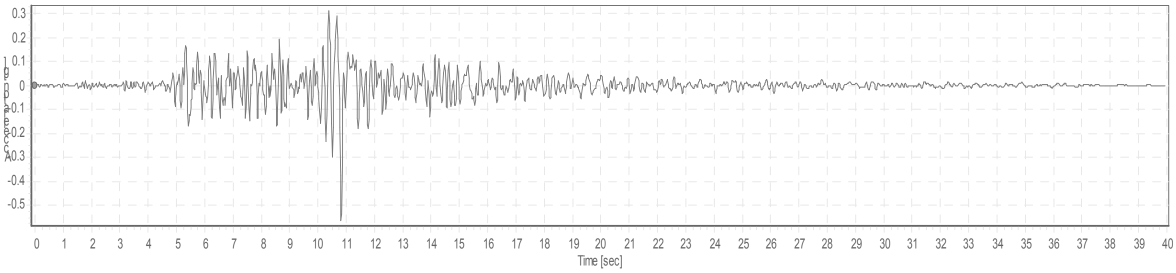

Figure 2.2. Time history Northridge record

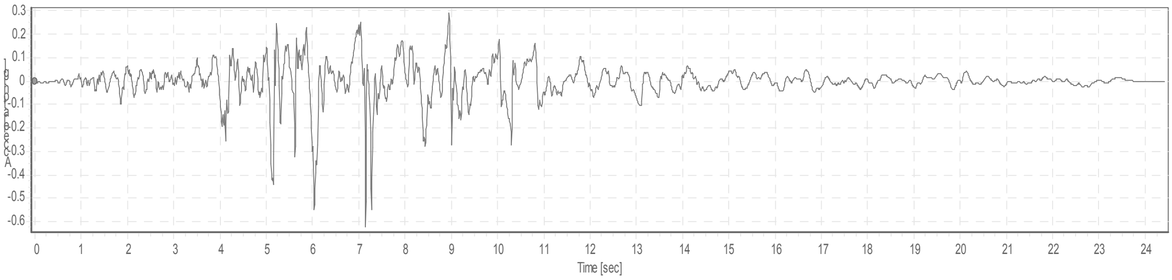

Figure 2.3. Time history of Victoria record

3. FINITE ELEMENT MODELLING

To investigate the use of tubular cross-sections for links in EBFs where no lateral bracing of the link is provided, a proof of-concept single storey the overall model dimensions were set to a height of 3150 mm and width, L, of 3660 mm (Berman J. W., et al.;2007). The test setup is shown in Fig. 3.1.

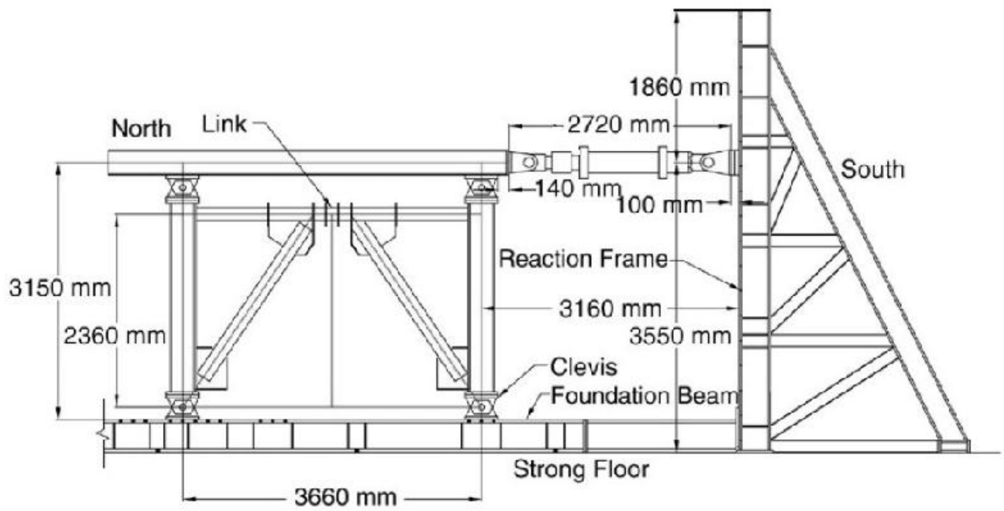

Figure 3.1. Proof-of-concept test setup in elevation **(Berman J. W., et al.;2007)**

According to the test (Berman J. W., et al.; 2007) for the proof-of-concept test specimen, a full penetration groove weld was chosen to join the 4 plates (2 webs and 2 flanges) that were used to build the link's hybrid cross section. To simulate the situations the part instances of the models have been merged to each other (Fig. 3.2).

A finite element model of the frame from the proof-of-concept test was developed. To model the flanges, webs and stiffeners of the proof-of-concept link 4-node shell elements used. Finally non-linear dynamic time history analyses have been done under three scaled records which mentioned before. The model shown in Fig 3.3.

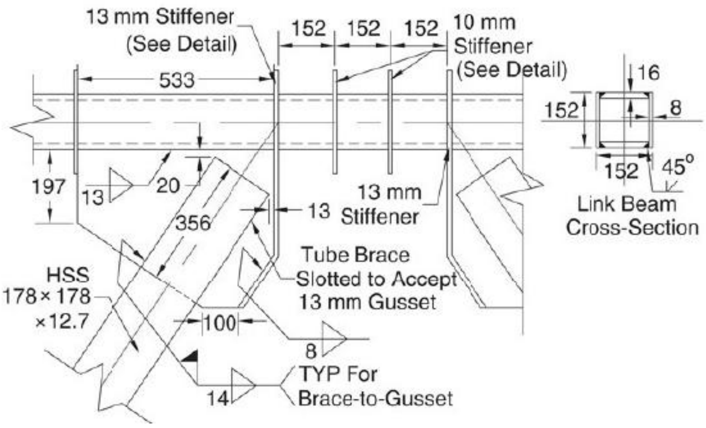

Figure 3.2. Proof-of-concept link details **(Berman J. W., et al.; 2007)**

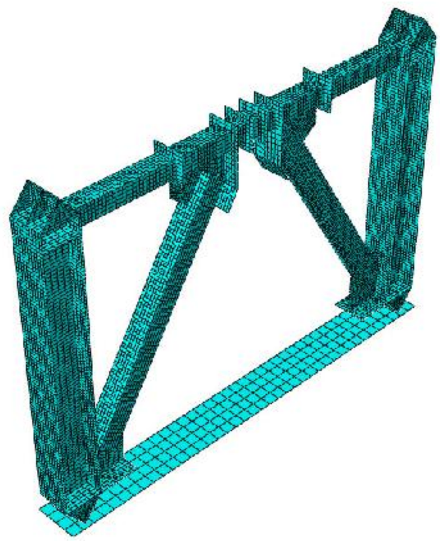

Figure 3.3. 3D Finite element model

In accordance with the previous research (Berman J. W., et al.; 2007) the nonlinear kinematic hardening plasticity material model is used in the finite element model of the proof-of-concept link. The characteristics of flange material are also used for the stiffeners.

The results of the time history analyses under three mentioned earthquakes are shown in Figs. 3.4 to 3.6.

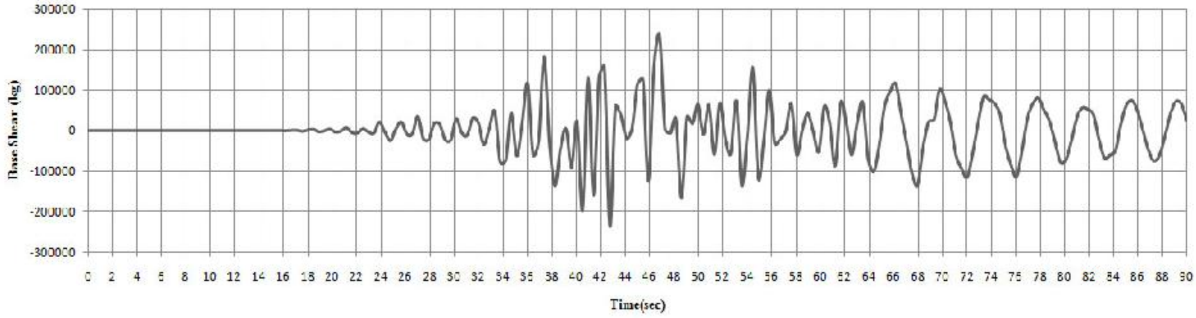

Figure 3.4. Base Shear due to Chi Chi record

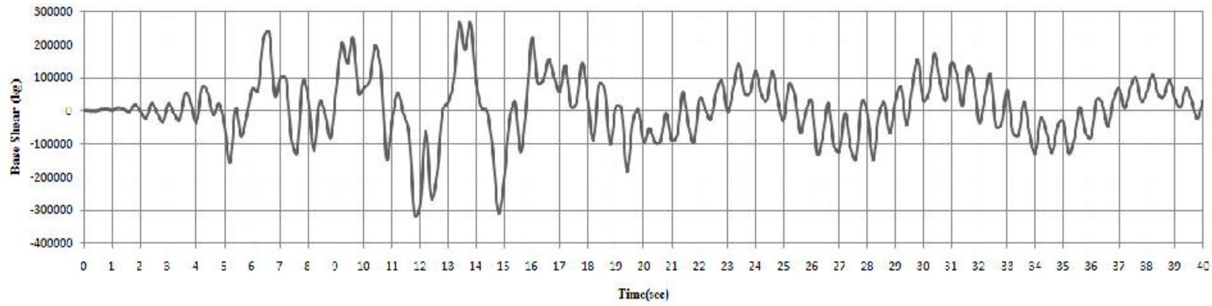

Figure 3.5. Base Shear due to Northridge record

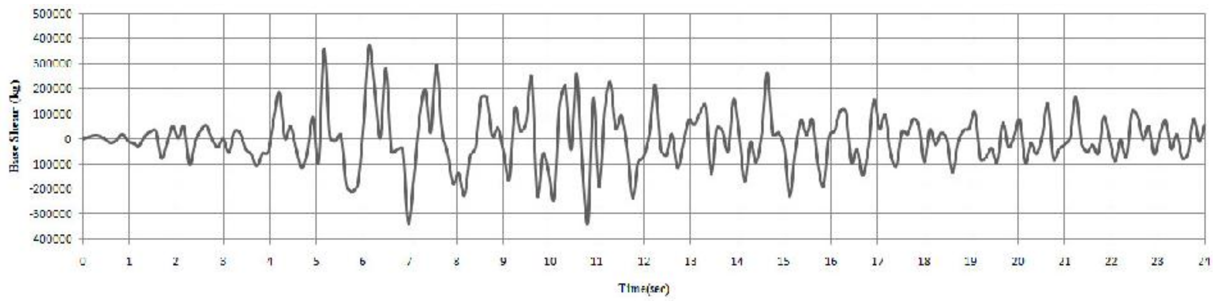

Figure 3.6. Base Shear due to Victoria record

Top and bottom displacement of structure under Victoria earthquake are shown in Figs. 3.7 and 3.8 respectively.

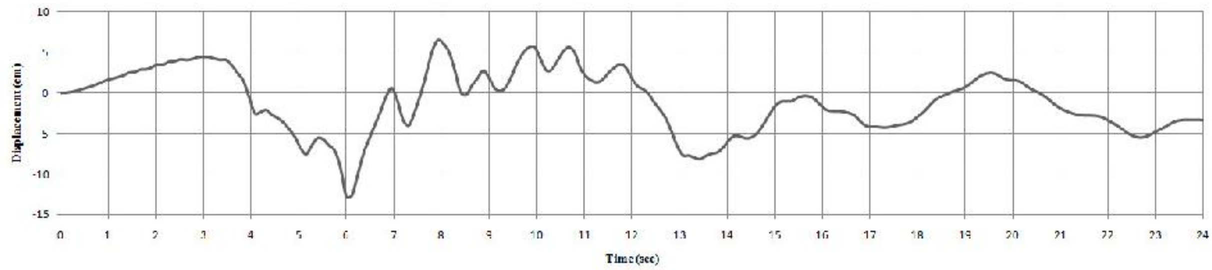

Figure 3.7. Top displacement of structure under Victoria record

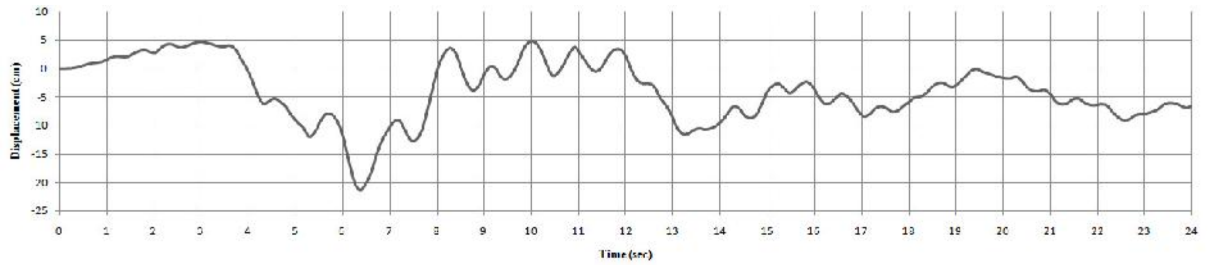

Figure 3.8. Base displacement of structure under Victoria record

4. CONCLUSIONS

According to the Figs. 3.4 to 3.6, it can be shown that the base shear has not been zero at the end of the records. It means that the structure is experienced inelastic deformation due to earthquake loading and there are some residual stresses as in the structure. While all records have an identical PGA, the maximum base shear is different in the structure due to these records. One of the main reasons is that frequency contents are different for the records. In Chi Chi earthquake the maximum of base shear is less than that in others earthquakes significantly. Earthquake magnitude and epicentral distance are other important factors affecting frequency content. The larger earthquake magnitude, the higher frequency content nevertheless high frequencies are absorbed by the earth when earthquake waves are travelling through the earth, In fact the earth act as a low pass filter. Comparison of base shear diagrams with top and base displacement diagrams shown in Figs. 3.7 and 3.8 results displacement diagrams have less sensitivity to shocks of the earth and their changes are smooth. The reason is yielding of some parts of structure which decreases sensitivity of structure.

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