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Seismic Performance of Concentrically Braced Frame with Zipper Column in the Near Fault Region

Mahdi Azizi , PhD Candidate in Structural Engineering, Department of Civil Engineering, Islamic Azad University, Tabriz, Iran. mehdi.azizi875@gmail.com

Abstract
Steel braced frame is one of the structural systems used to resist earthquake loads in buildings. The unexpected failure of steel structures during past strong seismic excitation led to full fill adequate strength for modern structures in seismic areas. Many existing steel structures with this system need retrofitting to overcome deficiencies and to resist seismic loads. In the present study, the seismic behavior of concentrically braced frame (CBF) structures was investigated and the "zipper column" idea for retrofitting was proposed. Two types of structures, including CBF and CBF with zipper columns for 5, 12 and 20 stories were designed per the Iranian Seismic Code, the Iranian Steel Structures Design Code and the Seismic Provisions. Dynamic analysis using 4 earthquake records were carried out to obtain dynamic responses. According to the analysis results, the maximum displacement, base Shear and the inter-story drift ratio in CBF systems are decreased as zipper columns are utilized. Also the frames showed good dissipating-energy capacity and large deformation ductility without significant strength losses. Moreover the seismic behavior of CBF system is improved.
Keywords: concentrically braced frames, chevron-braced frames, zipper columns, near-fault region and seismic design.

Introduction

Concentrically braced frame (CBF) in chevron configuration is a cost-effective system for resisting lateral loads. This structural system is usually employed for low- and mid-rise steel framed buildings. Braces in chevron configuration provide support for the CBF beams at the brace to beam intersection point. However, under strong seismic excitations, this configuration shows a concentration of damage within a single floor and the tendency of story mechanism formation. For instance, extensive damage was found in CBF buildings during Tohoku earthquake on March 2011 [1], Christchurch earthquake on 2010 [2] and former major earthquakes such as Loma Prieta (1989), Northridge (1994) and Kobe (1995) [3].

Figure 1: CBF failure from top down and left to right: (a) Buckled brace (Christchurch Earthquake); (b) Out-of-plane deformation and fracture of gusset plates (Tohoku Earthquake); (c) Local buckling in square-HSS brace (Tohoku Earthquake). [1, 2]

In light of this, frequent damage was observed in braced frames where braces were proportioned to resist tension only, where connections were weaker than the braces attached to them, where braces framed directly into columns, and where braces were inclined principally in one direction. Under strong ground motions, braces in compression have buckled, and in consequence lose their buckling resistance strength. [6].

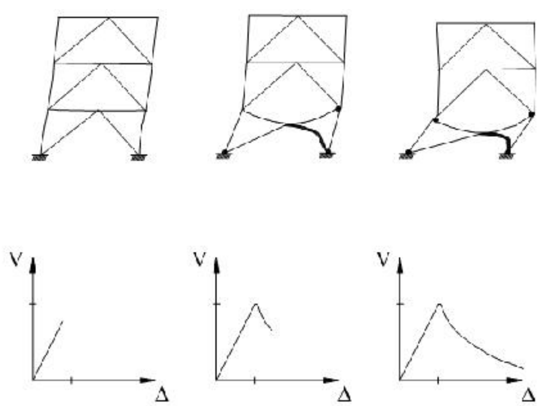

Figure 2: Collapse mechanism and load-displacement relationships for conventional braced frames [7]

After buckling of braces occurred, beams were deflected downward as a result of the combined action of the gravity loading and the unbalanced force developed at the braces to beam intersection point due to the difference between the tensile and post-buckling capacity of brace members. In this case, strong floor beams are required to stabilize the system when the unbalance vertical load transferred from braces to beams has increased due to the attaining of the post-buckling strength in the compressive brace. Thus, due to this behavioral characteristic, the chevron bracing system shows a limited efficiency in terms of distributing the lateral loads over the building height [6].

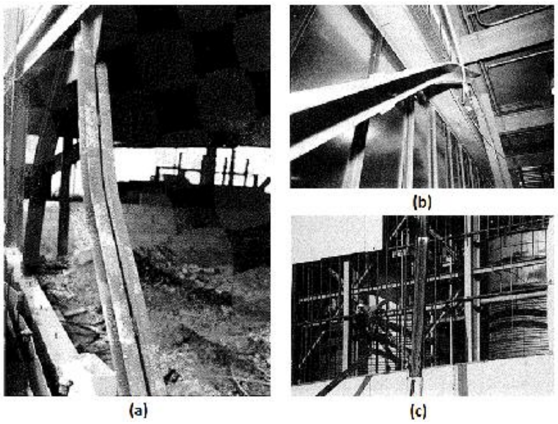

Figure 3: CBF failure from top down and left to right: (a) out-of-plane buckling of the first-story bracing members;
(b) Buckling of double angle chevron bracing in the tower portion;
(c) Close-up view of cladding failure on warehouse revealing steel braced frame. (Northridge Earthquake) [3]

In order to mitigate the formation of story mechanism and to achieve a stable inelastic seismic response, Khatib et al in 1988 [7] proposed to add a zipper column to link together all brace-to-beam intersecting points, with the aim being to force all compression braces to buckle and tensile braces to yield, such that a large amount of energy will be dissipated. If the compression brace in the first story buckles while all other braces remain elastic, a vertical unbalanced force is then applied at the middle span of the first story beam. The zipper elements mobilize the stiffness of all beams and remaining braces to resist this unbalance. The unbalanced force transmitted through the zipper elements increases the compression of the second story compression brace, eventually causing it to buckle [8]. The proposed method by Khatib et al [7] is called "Tension zipper column approach".

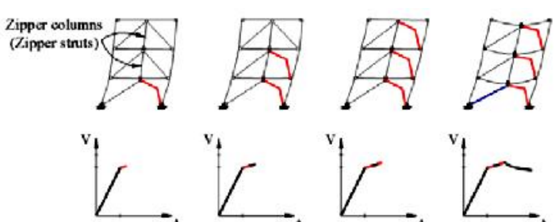

Figure 4: Expected behavior and performance of zipper frame [17]

Although in the last decade several researchers in North America have conducted analytical and experimental studies in the field of behavior and design of zipper braced frame systems, the concept is different and can be defined as follows:

- CBF with weak zipper strut (inelastic behavior) [9];
- CBF with strong zipper strut (elastic behavior) [10, 11] and
- CBF with suspended zipper strut [12].

In advance, experimental studies have been conducted only for the CBF system with suspended zipper struts. These researches are presented in the following sections [13, 14].

This paper presents the seismic performance of steel concentrically braced frames with zipper columns that acted as energy dissipation parts through the loading in the near-fault region. Numerical response analysis for the models with and without zipper columns has been carried out for the purpose of comparison.

Analytical models

To evaluate the seismic behavior of CBF system and compare it with CBF that retrofitted by zipper columns, three models with 5, 12 and 20 stories and a bay length of 5 meter were considered. The height of stories is assumed to be 3.6 m in all models. Fig.5 shows the plan of the structure, in which the zipper braces are located in the mid-bay of the perimeter frames (Fig.6).

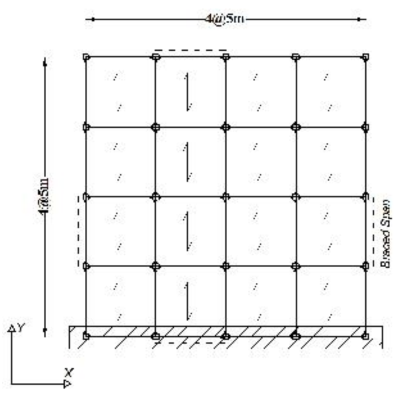

Figure 5: Plan of the model structures

Figure 6: Concentrically braced frame with and without zipper column for 5 story model

According to the Iranian loading code, the rate of live and dead force have been considered 200 kg/m^2 and 600 kg/m^2 respectively and then, structures are analyzed and designed by ETABS V.9. Assuming the conditions of area with much relative danger, the type of usage for residential buildings and lands will be of type III and the loading of frames will be done according to Iranian Seismic Code. The type of the steel utilized in frames is of St37. Yield stress of steel is 2400 kg/cm^2 and ultimate stress of steel is 3700 kg/cm^2, Poison factor 0.3

and modulus of elasticity of steel is 2.1×10^6 kg/cm^2. After statically analyzing of the structure, it has been designed and specified sections to members in the design have been determined according to Iranian steel structures design code.
Time history analysis of models is considered by Abaqus V.6.14 software. Situation of plastic hinges in frames are considered by Iranian Instruction for Seismic Rehabilitation of Existing Buildings.

Near-fault ground motion

The 1971 San Fernando earthquake was probably the first one to drive the attention of scientists on the peculiarity of earthquakes in the near-fault area and on their potential to damage structures. Since then many other destructive earthquakes occurred near densely built areas (Imperial Valley 1979, Whittier 1987, Northridge 1994, Kobe 1995, Chichi 1999 etc.) and the study of damage on structures coupled with the analysis of recorded seismograms helped to understand some of the most important aspects of earthquakes in the near-fault [19].
Near-fault ground motions, which have caused much of the damage in recent major earthquakes (Northridge 1994, Kobe 1995), are characterized by a short-duration impulsive motion that exposes the structure to high input energy at the beginning of the record. This pulse-type motion is particularly prevalent in the “forward” direction, where the fault rupture propagates towards a site at a velocity close to the shear wave velocity. The radiation pattern of the shear dislocation of the fault causes the pulse to be mostly oriented perpendicular to the fault, causing the fault-normal component of the motion to be more severe than the fault parallel component. The need exists to incorporate this special effect in the design process for structures located in the near-fault region. The near-source factors incorporated in recent codes cannot solve the problem consistently, because design procedures should pay attention to the special frequency characteristics of near-fault ground motions. Moreover, the emerging concepts of performance-based design require a quantitative understanding of response to different types of ground motion at different performance levels, ranging from nearly elastic to highly inelastic behavior [20].
It is recognized that the characteristics of near-fault earthquake ground motions are different from those records in the far-field. The fault normal component is of higher peak ground acceleration than the fault parallel component at the same recording station. In the forward directivity zone, the velocity record is characterized by pulse type motion of long duration. The effect of this pulse type motion on the response is important in the design of structures for near-fault events. In the near-fault region, the short travel distance of the seismic waves does not allow enough time for the high frequency content to be damped out of the record as is normally observed in far field records. Near fault effects were observed in failures during the 1994 Northridge and 1995 Kobe earthquake events [21]. Figure 7 presents the pulse type motion in velocity time history for Kobe earthquake.

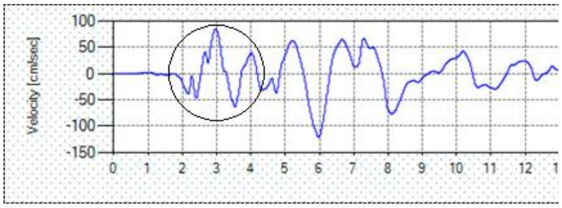

Figure 7: pulse type motion in velocity time history for Kobe earthquake

The designed moment resisting frames are subjected to a set of selected near-fault earthquakes. The selected earthquakes are shown in Table 1. All the earthquakes are larger than magnitude 6 with short epicentral distances of 1 to 12 km. The peak ground acceleration PGA, peak ground velocity PGV and peak ground displacement PGD of each record are listed in Table 1. For near-fault records in forward directivity zone, the ratio of the PGV (in cm/s) to the PGA (in g) is calculated for comparing reasons.

Table 1: Near-fault records [22]

Earthquake	Station	Distance (KM)	PGA (g)	PGV (cm/s2)	PGD (cm)	PGV/PGA
KOBE 1995	TAK-090	1.47	0.616	120.72	37.73	195.97
LOMA PRIETA 1989	CLS-000	3.5	0.64	55.14	10.82	86.15
DUZCE 1999	DZC-270	6.58	0.535	83.5	51.6	156.07
MANGIL 1990	ABAR-L	12.56	0.635	42.46	14.91	66.86

Structural behavior in the near-fault region

Bertero et al [23] studied for the first time the effects of near-fault impulse-type ground motions on structures. They observed that structural damage was imputable mainly to few large displacement excursions, which could be caused by a single large pulse with a short rise time and a long duration relative to system’s period, provided the mean value of the acceleration pulse exceeds system’s seismic resistance coefficient. Near fault registrations that match the “forward directivity” conditions (rupture front propagates towards the site and the direction of slip on the fault is aligned with the site), in fact, are characterized by a single large impulse of motion at the beginning of the seismogram (Somerville) [24], and may give rise to large velocity pulses, i.e. to a great amount of energy demand that have to be dissipated nearly instantaneously. Structures are forced to absorb such energy with one or few large plastic displacement excursions and therefore the ductility demand is probably the main responsible of building collapse during earthquakes in the near source. Anderson and Bertero [25] observed that impulse-type ground motion may be characterized by an increase in elastic spectral response in the long period region (1-3 sec), and this can have serious effects on structures with period of vibration in the range of 1-2 sec. On the other hand they noticed an increase of ductility requirements for more rigid structures, which is due to the higher value of the ratio of pulse duration to the period of the structure. The diversity of earthquake effects in the near fault area has already been taken into account by some of the most recent code provisions such as UBC97 [26] that introduces specific near fault amplification factors to increase seismic coefficient value. Still, a great effort should be done for an exhaustive comprehension of the phenomenon and for the correct quantification of its effects on structures.

Effect of zipper column on displacement of CBF

In this study, the target point for displacement in each structural model was set at the top story. The displacement of target point during the analysis was chosen to display the CBF and CBF with zipper column seismic behavior when the time history is finished. Also the maximum displacement of target point were derived for comparing reasons. Figures 8 and 9 present the variation of these parameters according to each earthquake record for 5, 12 and 20 story structural models.

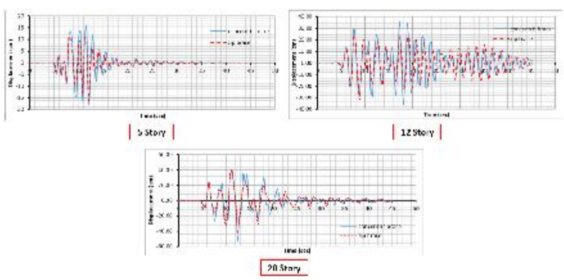

Figure 8: Displacement of target point for structural frames using Kobe record

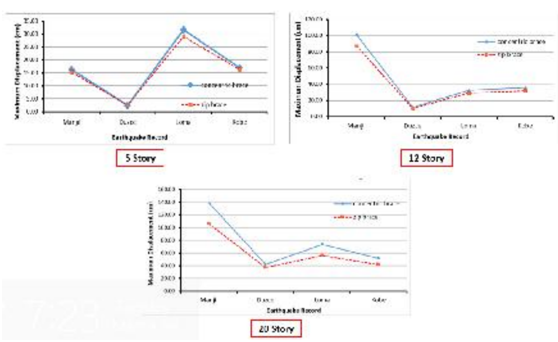

Figure 9: Maximum displacement of target point for structural frames

It can be observed in the figures that in most cases, the displacement of target points in CBFs are decreased when zipper columns are used, while there are noticeable difference when taller models are used. It also can be seen that the maximum roof-displacements of CBFs are decreased when zipper columns are used.

For comparing reasons, the maximum inter-story drifts in each story of the structural models with and without zipper columns were obtained from time-history analysis and are shown in figures 10 to 12.

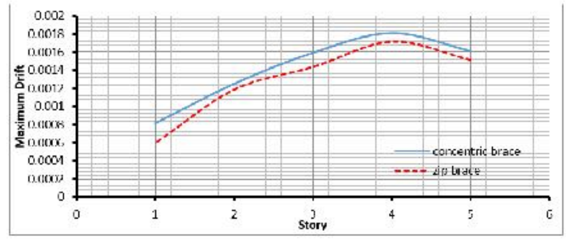

Figure 10: inter-story drift for 5 story model

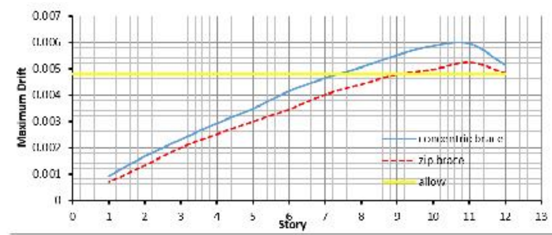

Figure 11: inter-story drift for 12 story model

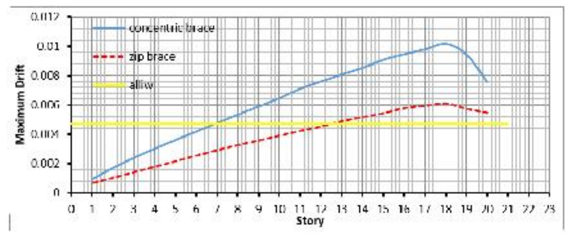

Figure 12: inter-story drift for 20 story model

It can be observed that as the zipper columns are used, maximum inter story drift decreases especially in the higher part of the structure. Current study shows that the maximum inter-story drifts of zipper frames are smaller than the maximum allowable drifts of codes when concentrically braced frames cannot satisfy this code requirement.

Forces at supports

For comparing reasons, the values of base shear in the frame supports were derived and presented in figures 13 and 14.

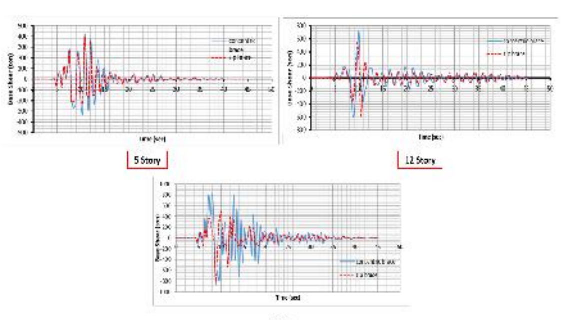

Figure 13: Base shear time history for frames using Kobe record

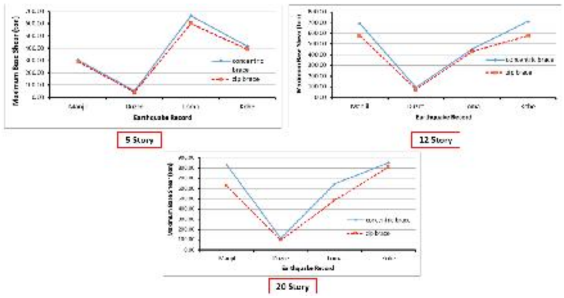

Figure 14: Maximum base shear for frames

It can be observed in the figures that when zipper columns are used in CBF systems, the nonlinear base shear decreases. It also can be seen that as models with higher story numbers are considered, the differences between these values get higher.

Tension braces axial force and energy dissipation

In this section, the effect of zipper column on the distribution of internal forces in braces such as axial loads are investigated. It is assumed that the critical points for braces in CBFs and zipper frames are in the mid height of braces. Figure 15 plots the change in internal forces of 5, 12 and 20 story frames.

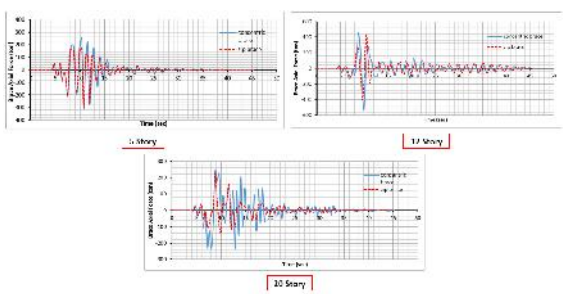

Figure 15: Distribution of brace internal force for frames using Kobe record

It can be observed in the figure that in most cases, axial force in braces are decreased when zipper columns are used.

For comparing reasons, the maximum strain energy in each structural models with and without zipper columns were obtained from time-history analysis and are shown in figure 16. It can be derived that as zipper columns are used, the energy dissipation process in models are enhanced so the maximum strain energy are increased.

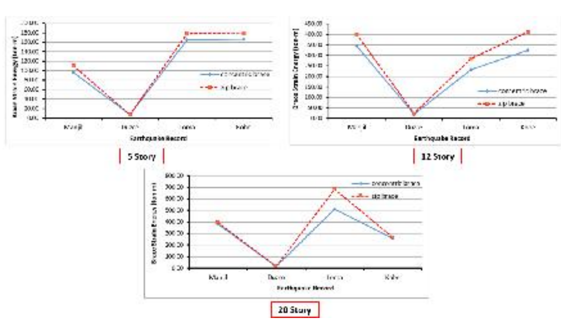

Figure 16: Maximum strain energy in braces

Conclusion

As one of the widely used seismic force resisting systems, chevron braced frame provides high stiffness and moderate ductility. However, under strong ground motion excitations, the structure is prone to story mechanism formation and reduced energy dissipation capacity due to the concentration of damage within one floor. To overcome this drawback, an innovative system is to add a zipper column at the brace to beam intersection points with the aim of carrying the unbalanced force resulted from brace buckling. According to the results, zipper frames have more ductile behavior and higher strength than ordinary concentrically braced frames. The zipper frames appear to reduce the tendency of chevron-braced frames to form soft stories and to improve seismic performance without having to use overly stiff beams.

It can be concluded that in CBF systems, the maximum displacement are decreased as zipper columns are utilized. The maximum inter story drift decreases especially in the higher part of the structure. Also current study shows that the maximum inter-story drifts of zipper frames are smaller than the maximum allowable drifts of codes requirement.

It also can be mentioned that the maximum base shear in zipper frames are decreased so the distributed earthquake forces in story height are decreased. Using zipper column has great effects on decreasing moments at supports and rotations in stories that controlling overturning forces of frames could be possible.

With utilizing zipper columns, the axial force in tension braces are decreased and the strain energy in braces are increased so the energy dissipation process in zipper frames are improved and the seismic behavior or CBFs are enhanced.

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