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Seismic Performance of Vertical Shear Links

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Abstract: One way to reinforce steel structures is to use braces which are both concentric and eccentric. Concentric braces have high stiffness and low ductility while eccentric braces have initial stiffness and proportional ductility. In eccentric braces, link beam is an area under plastic deformation which leads to earthquake energy absorption. These beams are either horizontal as a part of story beam or vertical located between story beam and two braces. Link beam length is one of the most important parameters in analysis and design of these structures. Behavior of link beam will be closer to shear link when its length is lower. If length of a link beam exceeds a certain amount, its behavior will shift to shear link. This study tends to evaluate seismic performance of frames with vertical link beam by using DRAIN-2DX which is the most powerful software for non-linear static and dynamic analyses.

Key words: Energy dissipation, shear link, pushover analysis, amount, length

INTRODUCTION

Two important criteria in designing buildings in seismic areas are resistance and ductility. EBFs not only provide the considered stiffness but also they have high ductility and energy absorption capability. There are two types of EBFs; frames with Horizontal link (H-EBF) and frames with Vertical link (V-EBF). Since, total energy of these frames is absorbed by link beam, V-EBFs are preferred over H-EBFs because link beam of H-EBF is a part of story beam which experiences plastic deformations during an earthquake therefore, it is required to replace the story beam after an earthquake which is difficult and impractical. In V-EBF systems however, link beam is out of the story beam between story beam and two braces therefore, it can be replaced easily after an earthquake. Figure 1 shows position of horizontal and vertical link beams in EBFs.

Literature review: Vetr and Bou kamp examined V-EBFs. Zahrai and Bruneau (1999) improved seismic performance of bridges by using Shear Panel System (SPS). They modelled a single span bridge and a three-span bridge by using ADINA and DRAIN-2DX and tended to improve these bridges by using TADAS, SPS and EBF systems. Bruneau and Sarraf performed tests for improving resistance of bridges by using vertical shear links. In designing SPS, they tended to have lower SPS flow resistance than capacity of other structural elements in order to dissipate energy by flowing the element. Stiffness of elements should not be too low because this leads to large lateral displacement in the structure and may cause damages to non-ductile elements. Moreover, very highly stiff SPS increases the need for ductility (Sarraf and Bruneau, 1998). Ghobarah and Elfath (2001) studied seismic performance of non-ductile concrete structures reinforced by divergent steel bracing with vertical link.

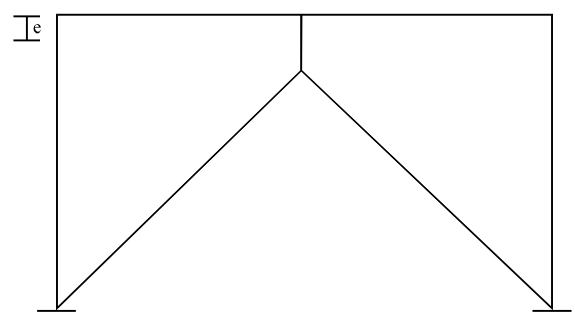

Fig. 1: Position of vertical link beam

Link length: Link beam length is one of the important parameters in design of Vertical Shear Links (VSLs). To cause hinge rupture in shear before bending, hinge length is limited to AISC (1997):

$$e \leq 1.6\frac{M_{PL}}{V_{PL}} \quad (1)$$

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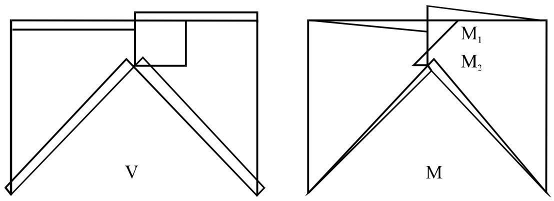

Fig. 2: Shear and bending distribution in V-EBF

Where:
M_{PL} = Plastic moment capacity of the beam section
V_{PL} = Plastic shear capacity of the beam section

Angle of rotation for link beams shorter than Eq. 1 is suggested at 0.08 radian (AISC, 1997). Energy dissipation capacity, yield mechanism and collapse all greatly depend on link length. For short link beams shear yield is the governing behavior while bending yield is deterministic in long link beams. This study tends to examine behavior of a frame with both types of link beams. Link beam is generally under high shear force along and considerable bending moment at the end. As shown by shear and bending distribution in vertical shear hinge (Fig. 2), moment is not equal at two ends; proportional to rotational stiffness of the beam or braces, upper moment is more than lower moment. Plastic shear force (V_p) and plastic bending moment (M_p) of link beam can be calculated by AISC (1997):

$$V_P = 0.6F_y(d-2t_f)t_w \quad (2)$$

$$M_P = ZF_y \quad (3)$$

Where:
t_f = Flange thickness
t_w = Web thickness
Z = Plastic link section
d = Total height of the section
F_y = The link yield stress

Stiffness, resistance and energy dissipation capacity of long link beams under severe periodic loads are not comparable to short link beams.

MATERIALS AND METHODS

Modelling: Several three-span three-story frames in which the middle of span was braced with different link lengths (110, 100, 40, 30, 20 cm) were considered for analysis. The Software SAP2000 V12 was used for designing frames. Connections were considered simple except for the braced span. Structures were loaded based on the Iran National building regulations (AISC, 1997). Earthquake force was based on equivalent static method of the Iran seismic design code (the standard 2800) (Building and Housing Research Center, 2006). The studied structures were residential and located in a relatively high-risk area. The ground was type 2. The considered frames are described in Table 1. Based on the standard 2800, seismic coefficient was calculated as follows:

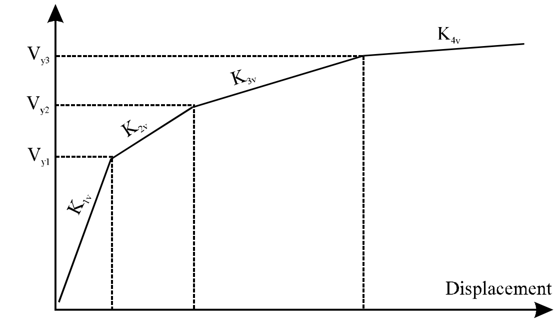

Fig. 3: Shear force-displacement curve for steel link

Table 1: Specifications of sections

Story	Column (IPB)	Beam (IPE)	Brace (2UNP)	Shear link (IPE)
1	240	300	220	220
2	200	300	220	220
Roof	200	270	220	220

$$W = 94358 \text{ kg}$$
$$H = 9 \text{ m}$$
$$T = 0.08H^{3/4} \rightarrow T = 0.415,\ T_0 = 0.1,\ T_S = 0.5,\ S = 1.5$$
$$T_0 \leq T \leq T_S,\ B = S+1 \rightarrow B = 2.5$$
$$C = \frac{ABI}{R} \rightarrow C = 0.125$$
$$V = CW \rightarrow V = 11.795$$

Then, DRAIN-2DX was used for non-linear analysis. Type 02 beam-column element was selected for elements of beam, column and braces. Since, DRAIN-2DX which is used in non-linear analyses does not detect plastic shear hinge, modeling of link beam seems critical. Therefore, the links considered by Ramadan and Ghobarah (1995) but with some modifications were used for modeling shear links. The link considered by Ghobarah and Ramadan is a column-beam element with two transitional springs at both ends. Each spring contains two internal and external nodes which are bound to each other in transition and transfer along X or Y-axis. To model vertical link beam, it was considered as a column-beam element with elastic springs at both ends and internal and external nodes of springs were bound to each other in transition and transfer along Y-axis. Figure 3 shows shear force displacement curve of shear links considered by Ramadan and Ghobarah (1995):

$$K_{1V} = \frac{GA_{web}}{e} \tag{3}$$

$$K_{2V} = 0.03\, K_{1V} \tag{4}$$

$$K_{3V} = 0.015\, K_{1V} \tag{5}$$

$$K_{4V} = 0.002\, K_{1V} \tag{6}$$

Where:

G = Shear module of the section
e = Link length
A_{web} = Web cross-section of vertical link

RESULTS AND DISCUSSION

Different levels of performance and performance point

Performance point (target): Performance point is determined from intersection of capacity curve and displacement needs. In other words, performance point is stopping point of a structure along structural capacity curve that is this structure under design earthquake or any other earthquake used has a response which reflects performance point of the structure. To obtain this point, performance level of structural or non-structural members can be determined under forces and deformations caused by the required displacement.

Capacity curve: Capacity curve is obtained by pushover curve of the structure.

The required displacement curve: The required displacement of the structure under the design risk-level earthquake is obtained by using non-linear time-history analysis. Figure 4 shows capacity curve, bilinear capacity curve and the reduced 2800 curve for the model with 30 cm link.

Performance level: FEMA356 suggests three performance levels (ANON, 2005).

Immediate occupancy: Immediate occupancy refers to a performance level in which it is predicted that resistance and stiffness of a structure under earthquake do not considerably change.

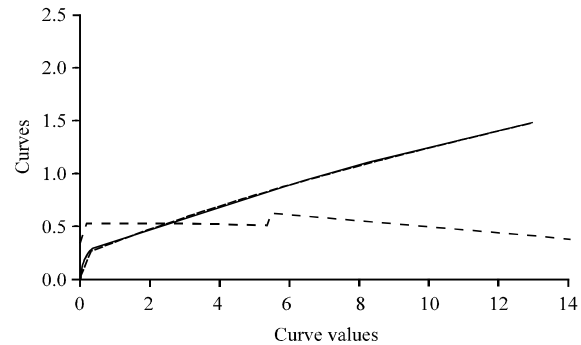

Fig. 4: Capacity curve, bilinear capacity curve and the reduced 2800 curve

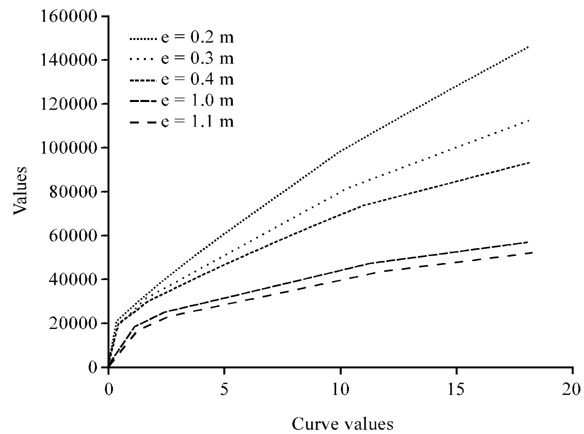

Fig. 5: Pushover curve of the considered models

Life safety: Life safety refers to a performance level in which it is predicted that earthquake does not cause considerable damages to the structure and damages do not lead to fatality.

Collapse prevention: Collapse prevention refers to a performance level in which it is predicted that earthquake causes extensive damages to the structure while the building does not collapse and fatality is minimized. Since, a certain performance level was not intended in this study, various performance levels were considered for different link lengths (Table 2).

Behavior of the modeled examples: This study considered five models with shear and bending links for non-linear static analysis. For better understanding of results, behavior of the links used in models was completely shear or bending. For this purpose, five link lengths were used three of which (20, 30 and 40 cm) had shear behavior and two others (100 and 110 cm) had bending behavior. For convenience, the models are shown by capital letters (Table 3-5). Figure 5 shows pushover curves of the

Table 2: Modelling parameters and acceptance criteria for non-linear procedures

	Modeling parameters				Acceptance criteria (Plastic rotation angle, radians)			
	Plastic rotation angle, radians		Residual strength		Primary		Secondary	
Component/Action	a	b	ratio (c)	IO	LS	CP	LS	CP
EBF link beam								
$e \le \frac{1.6M_{CE}}{V_{CE}}$	0.15	0.17	0.8	0.005	0.11	0.14	0.14	0.16
$e \ge \frac{2.6M_{CE}}{V_{CE}}$	Same as for beams	-	-	-	-	-	-	
c. $\frac{0.6M_{CE}}{V_{CE}} < e < \frac{2.6M_{CE}}{V_{CE}}$	Linear interpotation shall be used		-	-	-	-	-	

Modelling parameters and acceptance criteria for non-linear procedures-structural steel components (continued)

Table 3: Displacement and base shear in different performance levels

Models	e (cm)	Δ_{io} (cm)	Δ_{ls} (cm)	Δ_{cp} (cm)	V_{io} (kg)	V_{ls} (kg)	V_{io} (kg)
A	20	0.304	6.950	8.980	18750	75300	90900
B	30	0.425	11.028	13.189	19950	84450	93000
C	40	0.552	14.179	17.230	20850	82800	90750
D	100	1.110	1.172	1.193	17850	18450	18600
E	110	1.220	1.236	1.252	16650	16800	16950

Table 4: Roof displacement and base shear in target displacement

e (cm)	V_p (kg)	Δ_p (cm)
20	38850	2.285
30	39900	3.103
40	40350	3.743
100	43200	9.558
110	41700	10.67

Table 5: Performance level and stiffness

Performance level	IO	IO	IO	-	-
e (cm)	20.000	30.000	40.000	100.000	110.000
Δ_{io} (cm)	0.304	0.425	0.552	1.110	1.220
Δ_{ls} (cm)	6.950	11.028	14.179	1.172	1.236
Δ_{cp} (cm)	8.980	13.189	17.230	1.193	1.252
Δ_p (cm)	2.285	3.103	3.743	9.558	10.670

models in different colors. As shown in Fig. 5 increase in link length leads to a considerable decline in pushover curve that is a certain displacement with increased link length requires lower force applied to the structure. In addition, increase in link length reduces structural stiffness. For example, stiffness of the model D is 1.55 times greater than that of A. Table 5 shows performance levels of the considered models. As shown in Table 5, the models with shear link are within life safety levels while the models with bending link do not show any performance level. This indicates weak performance of bending links compared to shear links.

CONCLUSION

- The increase in link length reduces stiffness and energy dissipation capacity of the structure
- Shear link beams have stronger performance level than bending link beams

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