

The Effect of Vertical Shear-Link in Improving the Seismic Performance of Structures with Eccentrically Bracing Systems

Mohammad Reza Baradaran, Farhad Hamzezarghani, Mehdi Rastegari Ghiri, Zahra Mirsanjari

Abstract—Passive control methods can be utilized to build earthquake resistant structures, and also to strengthen the vulnerable ones. In this paper, we studied the effect of this system in increasing the ductility and energy dissipation and also modeled the behavior of this type of eccentric bracing, and compared the hysteresis diagram of the modeled samples with the laboratory samples. We studied several samples of frames with vertical shear-links in order to assess the behavior of this type of eccentric bracing. Each of these samples was modeled in finite element software ANSYS 9.0, and was analyzed under the static cyclic loading. It was found that vertical shear-links have a more stable hysteresis loops. Another analysis showed that using honeycomb beams as the horizontal beam along with steel reinforcement has no negative effect on the hysteresis behavior of the sample.

Keywords—Vertical shear-link, passive control, cyclic analysis, energy dissipation, honeycomb beam.

I. INTRODUCTION

VERTICAL shear-links system or shear panel system (SPS) [1] is one of the passive energy dissipation systems that is installed vertically between the node of the two chevron braces and the lower flange of upper floor beam (Fig. 1). I-shaped cross section is often used in the design of vertical shear-links. These components appearance is similar to a short beam which is connected to the eccentric braces and dissipates seismic energy by yielding in the design intended circumstances. In the vertical link beam system, shear yielding of this beam's web, dissipates the seismic energy and causes the remaining elements to stay elastic; therefore, these components are made of mild steel [2]. One of the most effective, yet simple passive control methods is the use of vertical shear-links (VSL) in systems with eccentric bracing. This system has a stable hysteresis curve and doesn't cause stress concentration, and can dissipate the energy uniformly. After the failure of thinner sections, dampers will still have the

capacity to resist. In fact, in this system, hysteresis curves remain stable without any decrease in strength. Vertical shear-links also properly dissipates the input energy. This system has a good ductility as well as significant stiffness. Because of this good ductility of the system, relative displacements of floors and maximum displacement of structure cannot easily damage the building. In fact vertical shear-link acts like a ductile fuse and dissipates the seismic energy, and prevents the damage to the main structural elements such as beams, columns and bracing [3].

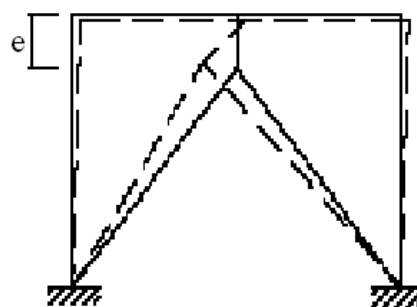


Fig. 1 Eccentric bracing system with vertical shear-links

If the design use bolts to attach the shear link beam to the main beam, it can be easily replaced and so it will be considered as a disposable member. This means that in case of a situation in which the link beam gets damaged by a severe earthquake, it can be removed from the structure and replaced by a new similar one. Another major advantage of this method is the possibility of its relatively easily utilization in the improvement and strengthening existing buildings. Using this vertical shear-link or shear panel technique can help to improve the seismic reaction of structures which lack an earthquake resistant system, especially in countries where the vast majority of buildings lack such systems, provided that those buildings possess the appropriate structure for the implementation of vertical shear-links.

Vertical shear-links in eccentric bracing systems, unlike the horizontal shear-links, are not located inside the structure and can be easily replaced. So, after the earthquake, assuming that other structural elements have remained elastic, only the vertical shear-links must be replaced, and then structure can continue its normal function. A knee brace should be designed and constructed with high accuracy and a slight change in its characteristics reduce the ductility without increasing the

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stiffness, but unlike knee brace, vertical shear-link can be easily designed and implemented.

Based on the results of nonlinear finite element analysis by Dusicka on the vertical shear-links which was performed in accordance with the AISC Seismic standards, It was found that the ultimate shear capacity had a good conformity with the modeling and testing values, It was also confirmed that the web of these link beams yields in shear stress [4], [5].

II. NOTES ON DESIGNING VERTICAL SHEAR-LINKS

Properly designed, vertical shear-links (VSL) can dissipate the earthquake energy, show high ductility, and keep other structural elements elastic. Length of the link beam is an important factor in the design of VSL. Weaker performance of long link beams compared with short ones has been proven in numerous experiments [6].

Regulations have limited this length to the following values in order to make sure that the shear failure occurs before the bending failure [7], [8].

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

In the above equation, e is the length of VSL, M_p is the plastic moment capacity, and V_p is the plastic shear capacity of the beam. According to the work done by some researchers in order to ensure the better performance of these components, it is recommended for the link beam to be shorter than the value mentioned above. For example, according to studies conducted by [9], as a conservative equation for the case in which the moment is identical on both sides of the link beam, it is recommended to limit the Link beam length to the following values [10], [11].

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

Since in the vertical link beam system, shear yielding of this beam's web dissipates the seismic energy and causes the remaining elements to stay elastic, these components should be made of mild steel. According to UBC 1997, the minimum yield strength of steel used in link beams should be limited to 3,500 kilograms per square centimeter [12].

Rotation angle is another important factor in the design of vertical link beams. For link beams shorter than the value of (1)-(7), rotation angle of the Link beam is limited to 0.08 according to AISC 97, 0.09 according to UBC1997, and 0.06 according to CISC1991 [8], [13]-[15]. As we know, vertical shear-link should be designed in a way that its web would yield in the design intended circumstances so it can dissipate the seismic energy, and keep the other structural elements such as beams, columns and bracings elastic. So each brace must have a compressive strength equivalent to 1.5 times the axial force corresponding to the control strength of vertical link beam. Control strength of the link beam decreases with the shear strength V_r , or bending strength M_r , and is equal to

the one that would lead to lower force in bracing. Also some Stiffeners should be considered in the design, namely across the web of beam and on either side of it where the brace and link beam reach each other. These Stiffeners transfer shear forces and are also protect the web of beam against buckling. Middle Stiffeners prevent web buckling in vertical link beam, so they must be connected to web across the length of it. The regulations related to stiffeners can be seen in several valid standards [13]-[15].

III. PROCESS OF MODELING AND ITS DETAILS

Shell elements SHELL181 and SHELL43 were used in the process of modeling. The first element was used for the mesh of regular surfaces, and the second element was used for the mesh of irregular surfaces, such as link plates, that required a triangulated network. However, these elements both have four nodes and six degrees of freedom at each node, and generally have similar properties. But the "SHELL43" is more recommended for triangulated networks. Plasticity model applied in the analysis was based on the von Mises yield surface. Plastic hardening was defined using the multi-linear kinematic hardening rule. Steel with yield strength of 350 mega Pascal and ultimate strength of 500 mega Pascal was used in the model.

Boundary conditions applied to the columns were full rigid connections, and the beam to column connections were modeled as hinge joints. Gusset plates in the center and those that connect bracing to columns were placed in the form of double plates on either side of the bracing channel bars (U-profile or UNP). Other details of the model are shown in Fig. 2. The connection of vertical shear-links to the lower flange of upper main beam is in reality a bolted friction-type joint to allow the replacement of component. But in this model, this connection was created by coupling degrees of freedom in the adjacent nodes.

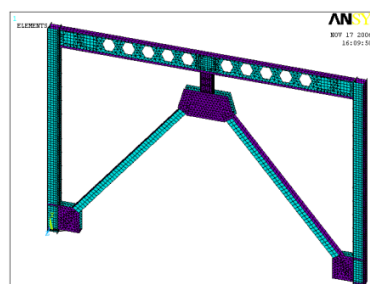


Fig. 2 Details of the frame sample modeled in ANSYS software

IV. DESCRIPTION OF STUDIED SAMPLES

In order to perform numerical studies on frames with shear panel (VSL), first we designed five single-story single-span frames. Designs were based on the laws of UBC97 and related seismic regulations. Since designing a single-story single-span frame based on actual loads usually results in small sections for structural members, it was decided that we should assume the characteristics of a vertical shear-link and then design other members proportional to the shear capacity of the link

beam. The length of the beam and height of the column were considered 3 meters and 4.2 meters respectively. Table I shows the characteristics of frames and link beams in 5 studied samples. Stiffeners thickness in all samples was 10 Millimeter.

V. PROCESS OF PUTTING LOAD ON SAMPLES

In accordance with AISC 1997 Seismic regulations, we used displacement control method to apply loading cycles in 0.125, 0.25, 0.5, 1, 2, and 3 times the yield displacement on sample in that order and in three cycles for each of them, and then applied loading cycles from 4 times the yield displacement up to sample failure in two cycles for each of them (Fig. 3). This loads applied to the structure in the form of a macro.

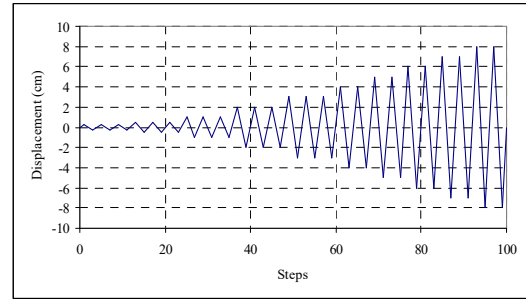


Fig. 3 Displacement curve applied to the frame with vertical shear-link

TABLE I
 CHARACTERISTICS OF STUDIED FRAME SAMPLES

Characteristics of Vertical Shear-Link (shear panel)			Characteristics of Frame						Sample
Stiffener distance (cm)	Length (cm)	Section	Brace		Beam		Column		
			Length (cm)	Section	Length (cm)	Section	Length (cm)	Section	
10	20	IPE160	345	2UNP80	420	IPB140	300	IPB120	SP1
10	20	IPE140	345	2UNP80	420	IPB140	300	IPB120	SP2
---	20	IPE140	345	2UNP100	420	IPB140	300	IPB120	SP3
10	30	IPE140	337	2UNP80	420	IPB180	300	IPB140	SP4
10	30	IPE160	337	2UNP100	420	CPE180	300	IPB140	SP5

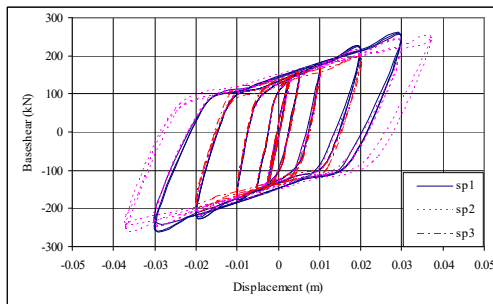


Fig. 4 Base shear-displacement hysteresis curve for the first three samples

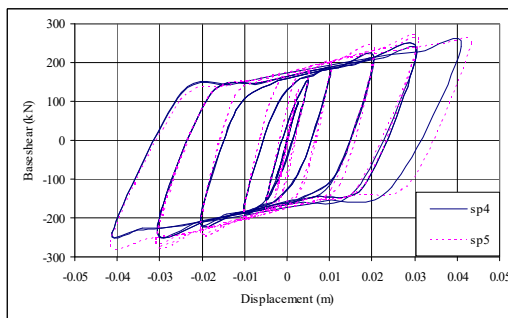


Fig. 5 Base shear-displacement hysteresis curve for the fourth and fifth samples

VI. RESULTS OF THE ANALYSIS OF STEEL FRAMES WITH VERTICAL SHEAR-LINK

Figs. 4 and 5 show the hysteresis curves of 5 analyzed samples. In these figures, frame's base shear is depicted in terms of story displacement. The frames of the first three samples are similar to each other, and this can also be said about the other two samples. Length of the Link beam is 20 centimeters in the first three samples and 30 centimeters in the fourth and fifth sample. Therefore, curves of the similar frames are depicted in one graph to allow a better comparison between curves.

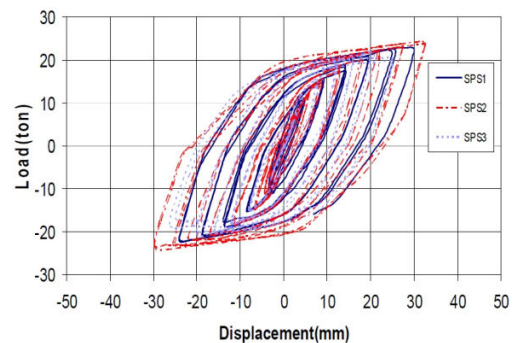


Fig. 6 Force-displacement curves of the first three samples made in laboratory

Figs. 6 and 7 show the force-displacement curves of the samples made in laboratory, and as it can be seen, these results are similar to the results of those samples modeled by finite element software. [16] As it can be seen, all samples have a

stable energy curve and high energy dissipation. The ductility coefficient of these samples was calculated between 6.25 and 7.23. Of course the value 6.25 belongs to the sample which lacked any stiffener. If we rule out this sample, the ductility coefficient of frames will be between 7 and 7.23, which have a lower dispersion range and is a more consistent value. The third samples had no Stiffener. Before the buckling of the web of the link beam, Lack of stiffener had no significant effect on the behavior of the frame, but after that, frame's load capacity decreased.

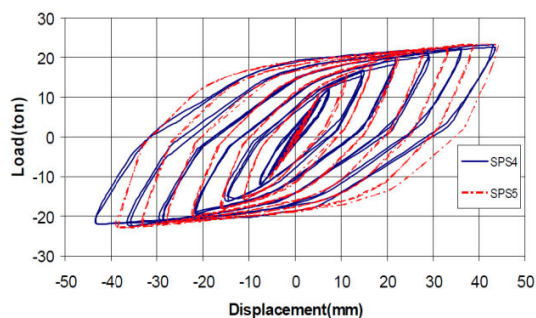


Fig. 7 Force-displacement curves of the fourth and fifth samples made in laboratory

As mentioned earlier, length of the link beam increased to 30 centimeters in the fourth sample. According to the calculations based on the UBC 1997 regulations, in order to make sure that beam's shear failure occurs before its bending failure, the use of IPE140 longer than 39 centimeters as link beam, is not allowed. But the analysis of third and fourth samples shows that increase in link beam length from 20 to 30 centimeters has no significant effect on the shear behavior of link beam. Considering the frequent use of Honeycomb beams in our country, it was decided that honeycomb beam should be used in the fifth sample. Therefore, a honeycomb beam reinforced with two 8 millimeters thick plates on openings and another plate on the junction of the vertical shear-link was modeled (Fig. 8). As can be seen from the results and graphs, by reinforcing the web of the honeycomb beam, we can use it without any negative effect on sample's hysteretic behavior.

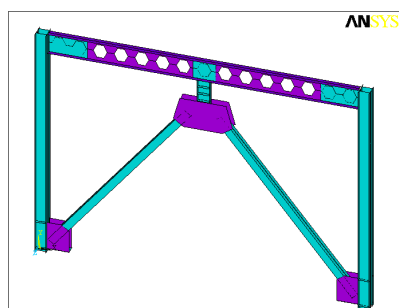


Fig. 8 Geometric model of eccentric frame with vertical shear-link in ANSYS software

Figs. 9 and 10 show the hysteretic behavior of the link beam in all of the 5 samples. As it can be seen, all samples have fat stable hysteresis curves and there is no shrinkage. Comparing

the frame's base shear-displacement curve with link beam's shear-rotation curve shows that a large portion of the force on the frame is carried and supported by link beam. So it can be said that designed samples have shown excellent performance and the expectation of other components remaining elastic is quite realistic and true, and analysis results also confirms it. Also the maximum rotation angle of link beam in these samples was between 0.09 and 0.152, the value 0.09 belonging to the sample without any stiffener. If we rule out this sample, the maximum rotation angle will be between 0.122 and 0.159. However AISC and UBC seismic regulations have limited this value to 0.08 and 0.09 respectively. Ductility of the shear panel component was also calculated and its value for the first, second and third samples was 30, 32.43 and 20.93 respectively. Lower ductility of the third sample compared with the first and second samples can be attributed to the absence of stiffener in that sample. Ductility of the fourth and fifth samples was 10.6 and 18.37 respectively. As it can be seen, shorter shear panel component and increased role of shear will increase ductility. Considering that the SP4 sample uses IPE 140 and the SP5 sample uses IPE160 and also that the maximum length allowed for shear in UBC1997 regulations is 38.7 centimeters and 44.5 centimeters for SP4 and SP5 respectively, by approaching the upper limit of shear, ductility of fourth sample is significantly reduced compared with the fifth one. While the first two samples have similar conditions to the fourth and fifth samples, the length of their shear panels are 20 centimeters, but there is not much difference between the ductility of these samples. So it is clear that the function of short link beam is far more satisfying than the longer link beam, even when both of them are in the full shear zone. While the required axial force for the yielding of the column was about 800 kilo Newton, The axial force of columns did not exceed 173 kilo newton. The axial force in braces also did not reach more than half of the axial force required for yielding. So the assumption on energy dissipation by the link beam and other components remaining elastic is confirmed. Another study on the these 5 samples by the application of increasing static load on them, showed that behavior factors of the 5 samples were 10.71, 10.69, 7.27, 8.43 and 10.93 in numerical order. Here it can also be seen that the third sample has a smaller behavior factors compare with first and second samples which is also caused by the absence of stiffeners.

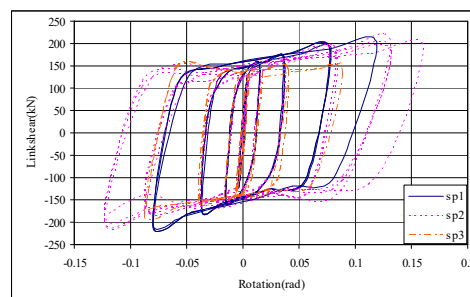


Fig. 9 Shear-rotation hysteresis curve for the first three samples

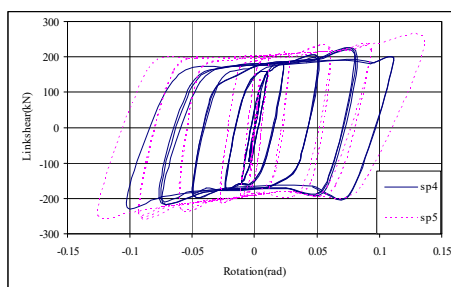


Fig. 10 Shear-rotation hysteresis curve for the fourth and fifth samples

VII. CONCLUSION

Vertical shear links have fat and stable hysteresis loops. These components act as a minor structural member and like a ductile fuse; they dissipate the seismic energy, increase the structures ductility and prevent the damage to the main structural elements such as beams, columns and bracing by preventing other members to reach yield point. The performance of short length vertical shear links, (which must yield in shear stress), is much better than the long vertical shear links. If the design use bolts to attach the link beam to the main beam, it can be replaced and will be considered as a disposable member. Before the buckling of the web of the link beam, lack of stiffener has no significant effect on the behavior of the sample, but after that, structures' strength decreases very quickly. The use of stiffeners increases ductility and also provides lateral support for the link beam and stabilizes the hysteresis curve. Because of the expected ductile behavior of the vertical shear links, these components should be made from mild steel. The use of honeycomb beams as the floor beam albeit reinforced with two plates on openings and another plate on the junction of the VSL has no negative effect on sample's hysteretic behavior. According to the calculations based on the UBC 1997 regulations, in order to make sure that beam's shear failure occurs before its bending failure, the use of IPE140 longer than 39 centimeters as link beam, is not allowed, but this study showed that increasing the length of the IPE140 link beam from 20 to 30 centimeters had no significant impact on the shear behavior of link beam.

The maximum rotation angle was between 0.122 and 0.159 while AISC and UBC seismic regulations have limited this value to 0.08 and 0.09 respectively. Local ductility coefficient of link beam ranged from 10.6 to 32.43, depending on the length of the beam and the presence or absence stiffeners. Ductility coefficient of frames with vertical shear links was between 6.25 and 7.3. This significant difference in the ductility of frame and the ductility of link beams is resulted from the link beam's superior ductility. Obtained behavior factors were between 7.27 and 10.9. In summary, we concluded that the initial assumption which stated that the utilization of vertical shear link keeps other structural members such as beams, columns and braces elastic is true and this component has proper ductility and energy dissipation.

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