

Replaceable reduced web link section for link-to-column connections in EBFs

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Abstract

The use of eccentrically braced frame (EBF) in high seismic regions is increasing day by day as EBF possess high elastic stiffness, stable inelastic response under cyclic lateral loading, and excellent ductility and energy dissipation capacity. The ductility and energy dissipation capacity of EBF depends on the active link beams. Link-to-column connections in EBFs are prone to failure at low drift levels lower than the AISC recommendations, due to their susceptibility to fracture at the link flange-to-column welds. In this study a replaceable link with reduced web section was developed that overcomes this limitations of link-to-column connections in EBFs. Since the failure is controlled at the reduced portion of web, the fracture at the link flange-to-column welds will be avoided. Quasi-static loading test was carried out to evaluate the cyclic inelastic performance of the proposed link. The loading test results show that the developed link satisfies the plastic rotation level recommended by the AISC provisions.

Keywords: EBFs, Replaceable active links, Reduced section, Quasi-static loading

INTRODUCTION

Eccentrically braced frames (EBFs) have high ductility, as in moment resisting frames and high stiffness as in concentrically braced frames. The research works on EBFs started in the mid-1970's with Roeder and Popov (1977) and then with Manheim and Popov (1982). The principle of EBF design is to confine all the inelastic activities within active links only. The design is directly related to active link forces in plastic state and plastic design is considered as most rational approach for EBFs. EBFs are designed to remain elastic in small to moderate earthquakes, while in severe earthquakes the systems form a mechanism, with inelastic demand being focused into the active link, thereby protecting the braces, columns and beams from damage. The damages in EBFs noticed during Christchurch earthquake series of 2010/2011, which has been the first event worldwide to push modern EBFs into the inelastic range, the limitations of conventional EBFs.

The disadvantage of using the conventional EBF is that the active link and the collector beam are part of a common floor beam element or the damaged link member is not isolated from the main structures, hence the repair of damaged links can be an expensive operation and time consuming task. Furthermore, it may also affect the use of the building. The concept of the removable link addresses the disadvantage of the conventional EBFs.

In 1994 Ghobarah and Ramadan started the concept and experimental investigation on bolted extended end-plate connections for eccentrically braced frames with link-to-column connection configuration. The inelastic performance they found was similar to fully welded connections. In 2003 Balut and Gioncu (3) in their suggestion for an improved dog-bone solution and they identified the advantage and disadvantage of replaceable dog-bone. They suggested that in order to control the formation of plastic mechanisms which provides a ductile behaviour of steel frame, the dog-bone should be weaker in strength or less cross sectional area than the connected beams so that it undergoes post-elastic deformation while the rest of the structural members remain in the elastic range. However, the disadvantage of using replaceable dog-bone according to Balut and Gioncu is the difference in section will make difference in yielding strength so that it needs exhaustive control (2). The concept of dog-bone can also apply for removable active link. The disadvantage in section difference is not only difference in yielding strength but also it has some difficulties in construction. A deck support is required at the replaceable links having less cross sectional during construction as shown in Fig. 1.

Link-to-column connection in eccentrically braced frames (EBFs), tend to fracture in the link flange prior to large link rotations due to column connection attracts greater moment because the axial stiffness of the column is stiffer than the flexural stiffness of the beam. AISC provisions also warns designers of this problem which indicates that it is an ongoing research. In this research, replaceable links with reduced web section were proposed and investigated in order to address the limitations of EBFs. When the plates with holes are subjected to shear, the elastic stress at the edge of the hole is four times the stress at the other portions as shown in Fig. 2. Thus, under cyclic shear loading, the plasticity of the reduced web link section starts in the web. Therefore, in link-to-column connection of eccentrically braced frame, the failure supposed to occur at the flange welds because the reduced sections of web reduces stress and strain values in the links flange at the connection. The web section reduction also helps to control the plastic deformation only at the link without total section difference between the link and collector beam.

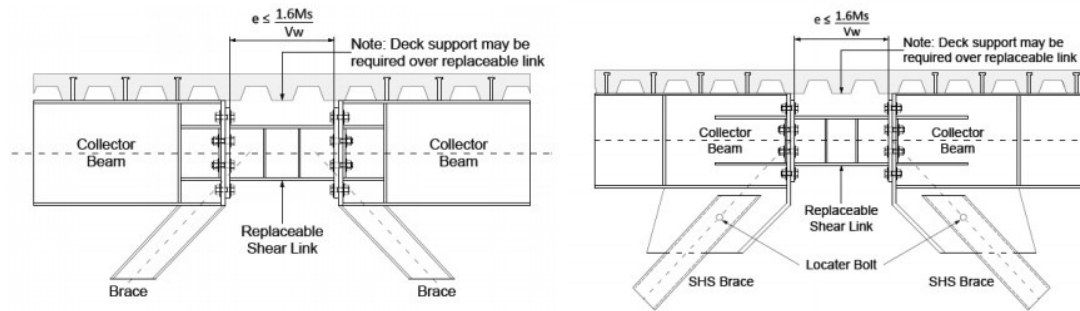


Fig. 1. Configuration of replaceable links. [3]

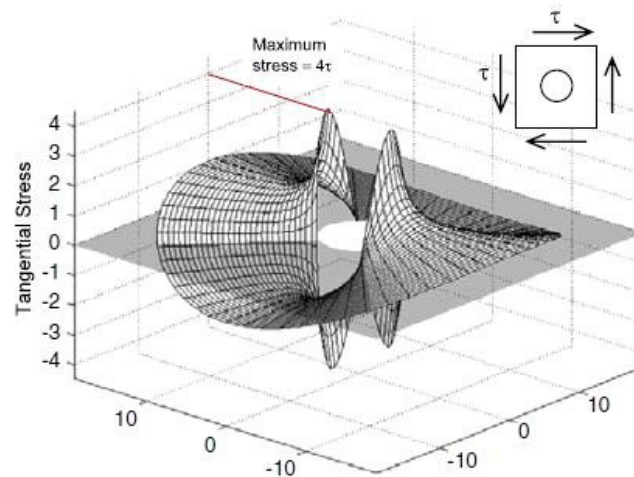


Fig. 2. Tangential elastic stress distribution in a plate with a hole subjected to shear.

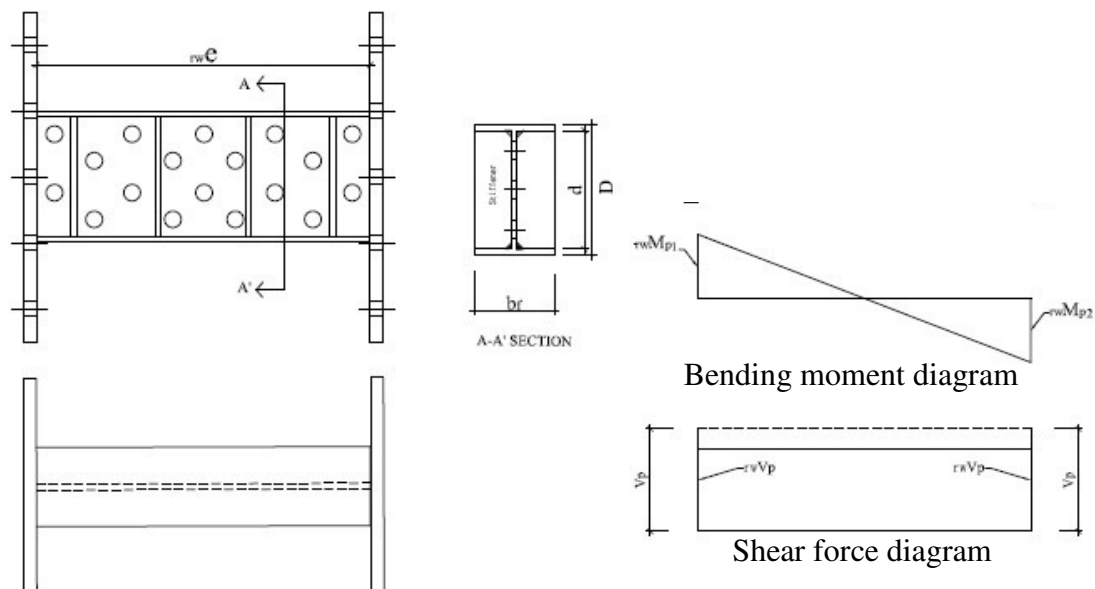


Fig. 3. Detail of reduced link section

PROPOSED LINK DETAILS AND DESIGN PROCEDURES

Detail of proposed links

Figure 3 shows the detail of the developed links with shear and bending moment distribution. As shown in the figure, the plastic shear of reduced web section is less than the plastic shear of unreduced section. The web section reduction has a slight effect on section plastic moment. Hence, the link length (${}_{rw}e$) is will be increased with a decrease of shear force. The behaviour of link is depends on link length factor given in Eq. (1) and (2). The rotation limit of link with respect to normalized link length is: short (shear) link ${}_{rw}\rho \leq 1.6$ is 0.08rad, long (flexural link for ${}_{rw}\rho \geq 2.6$ is 0.02rad and for intermediate link $1.6 < {}_{rw}\rho < 2.6$ the rotation limit is found by interpolation between 0.02rad and 0.08rad.

Design Procedures

The shear force and bending moment distribution diagram is drawn considering equal moments in both end for simplicity purpose although it is known that the moments in column face is bigger than moment in beam faces. As shown in Fig. 3 the plastic shear of reduced link section is less than the plastic shear of unreduced link. The plastic bending moment of reduced link section is taken at the reduced flange section, thus the reduced link length is smaller than the link length of unreduced link.

$$\rho = \frac{e * V_p}{M_p} \quad (1)$$

$${}_{rw}\rho = \frac{{}_{rw}e_p V_p}{M_p} \quad (2)$$

where ${}_{rw}\rho$: non-dimensional link length factor, ${}_{rw}e$: is the length of reduce link section, V_p and ${}_{rw}V_p$: plastic shear of unreduced and reduced link section defined as in Eq. (3) and Eq. (4) respectively. M_p and ${}_{rw}M_p$ is the plastic moment of unreduced and reduced link section defined in terms of yield strength and section modulus as in Eq. (5).

$$V_p = 0.6 f_y * d t_w \quad (3)$$

$${}_{rw}V_p = 0.6 f_y t_w (d - n\phi) \quad (4)$$

$$M_p = f_y * Z_p \quad (5)$$

where f_y : is yield strength, d : link depth, n : is number of perforations in a vertical alignment and ϕ : diameter of perforation, as shown in Fig. 3.

The behaviour of link is depends on link length factor given in Eq. (1) and Eq. (2) for unreduced and reduced link section respectively. According to AISC seismic provision, the yielding behaviour and the rotation level of link is governed with respect to link length factor. For shear links (short link): $\rho \leq 1.6$ and the rotation limit is 0.08rad, for flexural links (long link): $\rho < 2.6$ and the rotation limit is 0.02rad and for intermediate link $1.6 < \rho \leq 2.6$ the rotation limit is found by interpolation between 0.02rad and 0.08rad. The same can be applied for reduced link section as well. That means ${}_{rw}\rho$ dictates the yielding behaviour of reduced link section (${}_{rw}e$) or the following limits must be satisfied:

$$\text{Shear Link:} \quad {}_{rw}\rho \geq 1.6 \quad (6)$$

$$\text{Intermediate Link:} \quad 1.6 < {}_{rw}\rho \leq 2.6 \quad (7)$$

$$\text{Flexural Link:} \quad {}_{rw}\rho > 2.6 \quad (8)$$

The equilibrium equation of moment for unreduced and reduced link section in Fig. 3 is given as in Eq. (9) and Eq. (10) respectively.

$$M_{p1} + M_{p2} = e * V_p \quad (9)$$

$${}_{rw}M_{p1} + {}_{rw}M_{p2} = {}_{rw}e * {}_{rw}V_p \quad (10)$$

As it has been discussed above, since the end moments are assumed to be equal for simplicity, Eq. (9) and Eq. (10) can be rewritten as in Eq. (11) and Eq. (12) respectively.

$$M_p = \frac{eV_p}{2} \quad (11)$$

$${}_{rw}M_p = \frac{{}_{rw}e {}_{rw}V_p}{2} \quad (12)$$

Web section reduction has less effect on the bending moments. Thus, the plastic moments of unreduced and reduced link sections expressed in Eq. (11) and (12) are equal. Equalizing these two expressions, we can find the relationship between the plastic shear for reduced and unreduced web as Eq. (13).

$${}_{rw}e = \frac{V_p e}{{}_{rw}V_p} \quad (13)$$

Substituting Eq. (3) and (4) in Eq. (13), we can find:

$${}_{rw}e = \frac{de}{d - n\phi} = \left(\frac{1}{1 - \frac{n\phi}{d}} \right) e \quad (14)$$

where e: is the link length of unreduced section calculated in terms of plastic moment, plastic shear of unreduced section.

Equation (14) defines the relationship between reduced link length and unreduced link length. The parameter that controls the behaviour of link in both unreduced and reduced link is the link length factor (ρ and ${}_{rw}\rho$), which is given by Eq. (1) and (2) respectively.

TEST PLAN AND PROCESS

In order to evaluate the performance as well as the behaviour of the proposed links, quasi static loading test was carried out. The test setup used in these experiment was designed to reproduce the forces and deformations that will occur in a link installed between the loading beam and a plate fixed on the H-section beams. Fig. 4 shows the features and dimensions of the setup. As shown in the figure, the two columns pinned at both ends are used not only to support the loading beam they prevent the axial loads on the test specimens. In order to prevent the rotation at the top of the specimen pantographs was installed. The instrumentations that permits the calculation of forces and all link deformations is shown in Fig. 4. Link forces were derived from forces measured at the load cells and link deformations were computed from the difference between the displacement meters DT1 and DT2.

The loading protocol used for loading test was shown in Fig. 5. The loading protocol in the finite element analysis includes 6 cycles of link rotation γ at an amplitude of 0.00375 radians, 6 cycles of link rotations γ at an amplitude of 0.005 radians, 0.0075 radians and 0.01 radians, 4 cycles of link rotations γ at an amplitude of 0.015 radians and 0.02 radians, 2 cycles of link rotation γ at an amplitude of 0.03 radians, 1 cycle of link rotation γ at an amplitude of 0.04 radians, 1 cycle of link rotation γ at an amplitude of 0.05 radians, and the following each cycle of link rotation γ increased in the amplitude by 0.02 radians up to the limit on the actuator capacity or the specimen failure, whichever came first.

Table 1 Test specimen detail

Specimen ID	$\frac{b_f}{t_f}$	t_f mm	$\frac{d}{t_w}$	t_w mm	Open area (%)	r_{we} (mm)	Stiffener	V_p (kN)	M_p kNm	ρ	$r_w\rho$
S10S1	9	10	22.5	8	10	300	3	193.4	55.2	1.0	1.0
U10S1	9	10	22.5	8	10	300	-	193.4	55.2	1.0	1.0
U15S1	9	10	22.5	8	15	300	-	174.7	54.2	1.0	1.0
U10Sf1	13	9	22.8	8	10	500	-	205.9	69.1	1.65	1.5
S10Sf1	13	9	22.8	8	10	500	4	205.9	69.1	1.65	1.5
S10Sf2	13	9	26	7	10	500	4	180.2	68.4	1.85	1.3

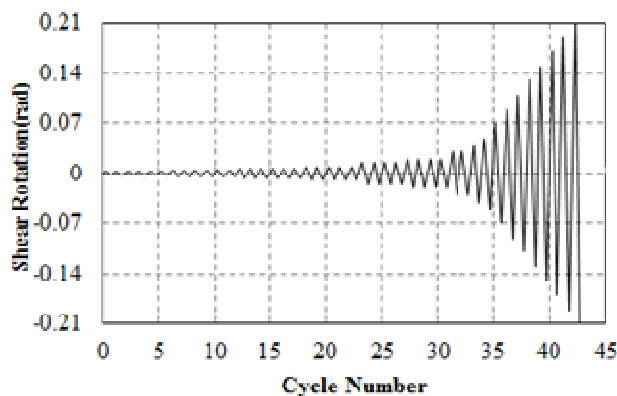


Fig. 5. Loading protocol

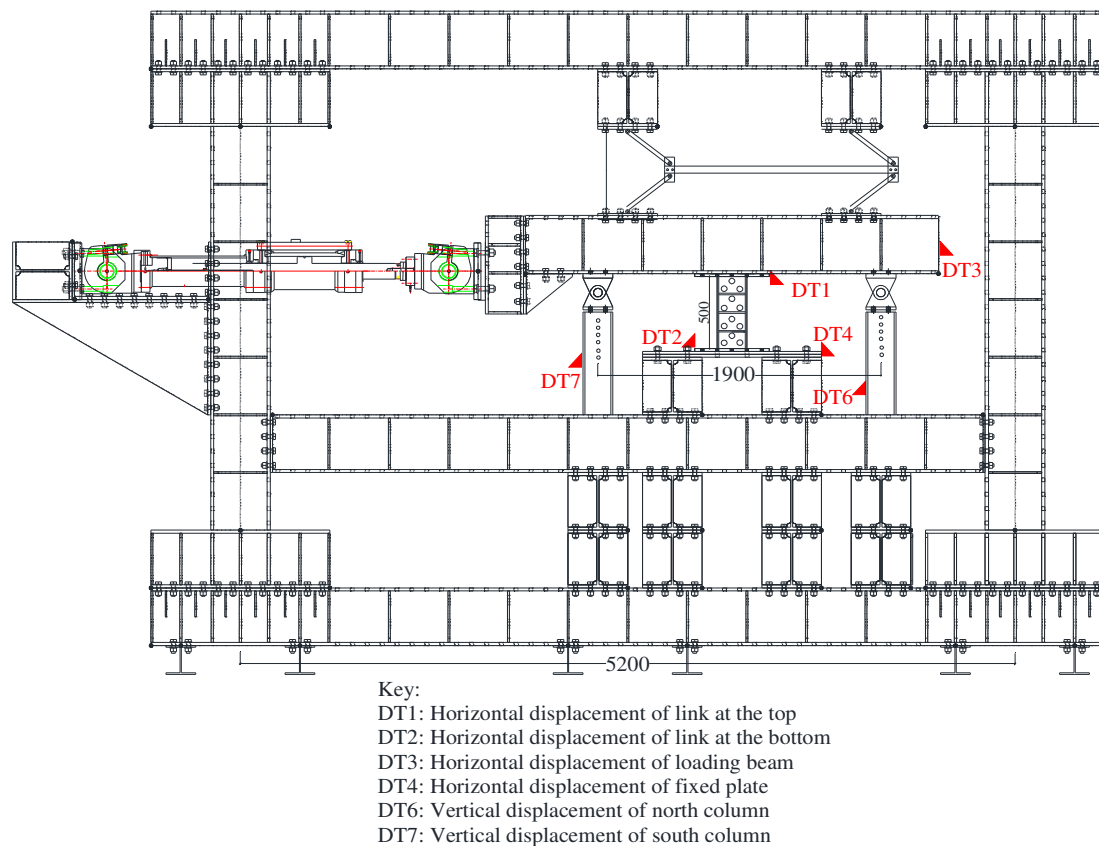


Fig. 4. Test set up and location of displacement meter

RESULTS OF EXPERIMENT

Hysteresis Response

Fig. 6 shows the hysteresis response of shear force versus shear rotation relationship of test specimens. As shown in the hysteresis response, the hysteresis loops of all specimens considered are stable in the range of $\pm 0.08\text{rad}$ which is the target plastic shear rotation. Note that not all specimens were loaded up to final failure such as U10Sf1, that stopped loading at 0.13rad due to the shock limitation of the actuator. The nominal value of plastic strength V_n was calculated plotted on the hysteresis loops using Eq. (4) which can be taken as a modified AISC 341-10 (AISC2010) provision.

Plastic rotation

According to the AISC seismic provision, one of the design requirements of links in eccentrically braced frames is their capacity of plastic deformation. The plastic deformation of link is expressed in terms of inelastic (plastic) rotation (γ_p) which defined as $\gamma_p = \gamma_t - \gamma_e$ where γ_t is the total shear rotation and γ_e : elastic link rotation. The plastic rotation of the test results are presented in Fig. 7 with the inelastic rotation collected from tests on steel links of various length ratios from literatures (Hjelmstad and Popov 1983; Malley and Popov 1984; Kasai and Popov 1986; Ricles and Popov 1986; Engelhardt and Popov 1989; McDaniel et al. 2003; Okazaki and Engelhardt 2007; Okazaki et al. 2009; Dusicka et al. 2010; Mansour et al. 2011). As shown in the Fig. 7, the plastic rotation of reduced link section evaluated in this paper exceeds the inelastic rotation requirements recommended in AISC 341-10 (AISC 2010).

Failure Mode

Two types of failure mode was observed during loading testing and are presented in Fig. 8. Stiffened specimens shows cracks around the edge of the web holes and fractures of web starting at the edge of holes. Web fracturing starting at the edge of hole and local buckling of flanges were observed for unstiffened specimens. During cyclic loading the reduced web hole changed their shape to elliptical before cracks and final fracture was occurred. As it has been assumed that the perforated plates under shear force fails at the edge of the hole, the obtained results proved the assumption we stands from. The test results were summarized in Table 2.

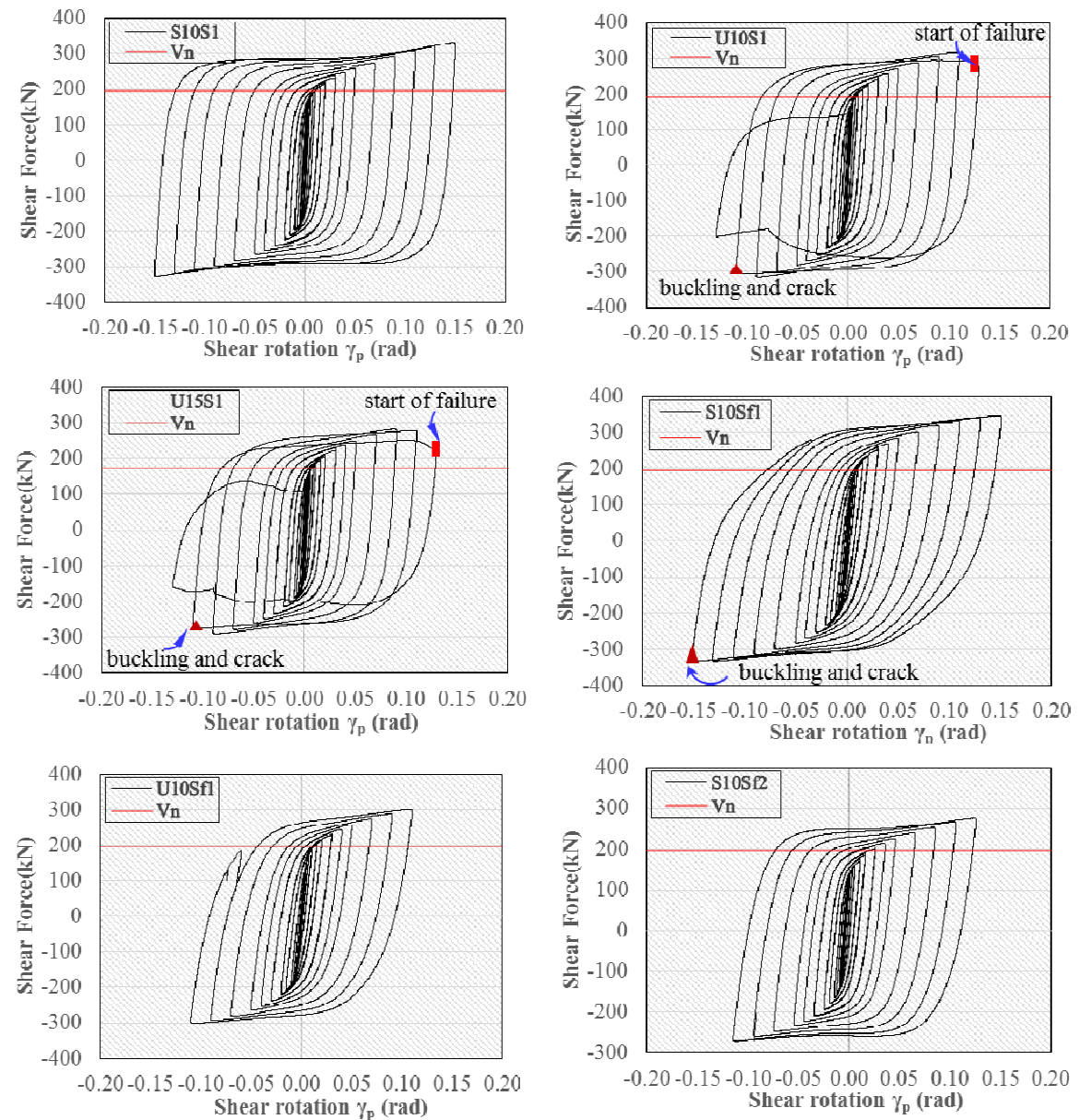


Fig. 6. Hysteresis response of test specimens

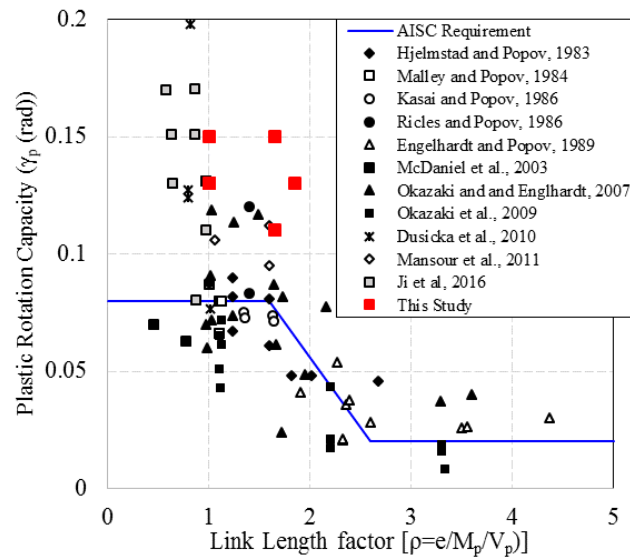


Fig. 7. Plastic rotation capacity versus link length ratio

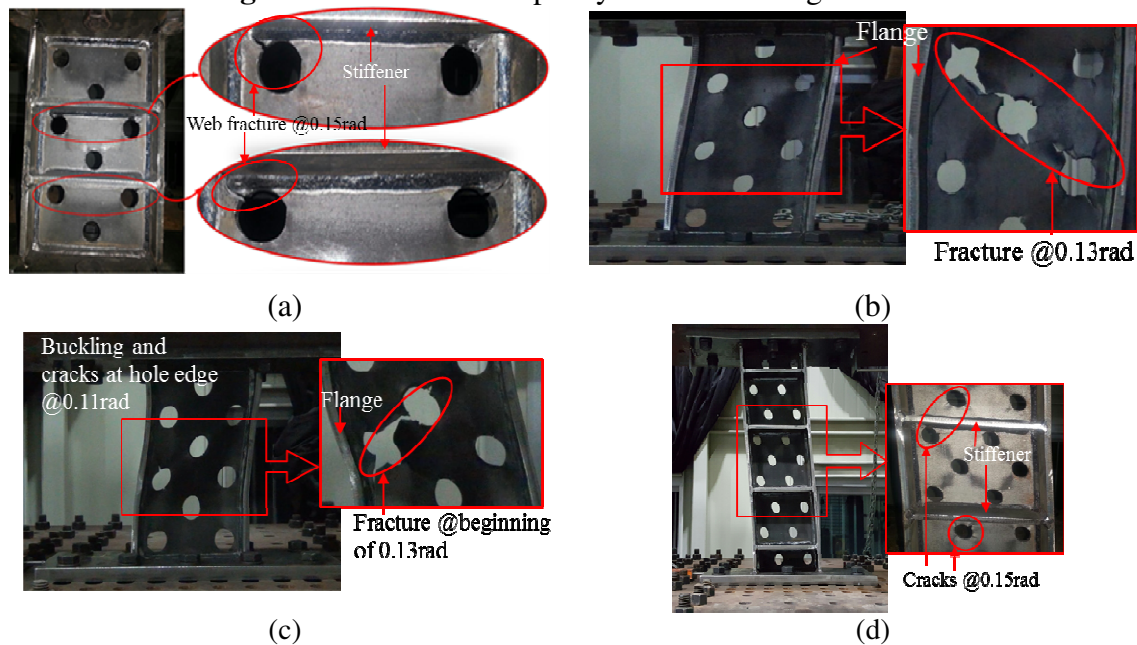


Fig. 8. Damage of test specimens: (a) web cracks at hole edge (Specimen S10S1); (b) web cracks at hole edge and flange buckling (Specimen U10S1); (c) web cracks at hole edge and flange buckling (Specimen U15S1); (d) web cracks at hole edge (Specimen S10Sf1)

Table 2 Summary of test results

Specimen number	Target plastic shear γ_p (rad)	Measured shear rotation (rad)	Observed buckling/cracks/failure
S10S1	0.08	0.15	Cracks around web holes
U10S1	0.08	0.13	Web fracture and flange plasticity
U15S1	0.08	0.13	Web fracture and flange plasticity
U10Sf1	0.08	0.11	Cracks around web holes
S10Sf1	0.08	0.15	Cracks around web holes
S10Sf2	0.08	0.13	Cracks around web holes

CONCLUSION

From the design procedure and experimental investigation on reduced web link section, the following conclusions are drawn.

1. The reduced link length factor and unreduced link length factor are different and calculated in different equations. The reduced link section length factor is generally smaller for the same link than unreduced link section.
2. The strength of reduced link section is smaller than unreduced section. Which enables reduced link section to control the plastic deformation only at the link with same overall section with collector beam so that no deck support is required at the link during slab construction.
3. The failure in reduced link section is started at the edge of the holes thus the fracture supposed to occur at the welds of flange is avoided. The considered links achieved a very large plastic rotation capacity of 0.13rads in average which is greater than 0.08rads recommended in AISC 341-10.

ACKNOWLEDGEMENT

This work was financially supported by Basic Science Research Program through the National Research Foundation of Korea (NRF) funded by the Ministry of Education, Science and Technology (No. 2015-053557) and with the support of Jeollanam-do (2016 R&D supporting program' operated by Jeonnam Techno park)

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