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Experimental Study on the Seismic Behavior of Replaceable Shear Links Connected to Coupling Beam

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***Abstract*— A seismic device for steel core frames of high-rise buildings has been developed. The system consists of two concentrically brace frames connected with coupling beams and shear links. The shear link has been designed to yield and dissipate the energy while the coupling beam and other elements of the frames remain elastic. The experimental work had been conducted on a model of half-scale sub-assemblage of core steel frame with its links made in Japan and Indonesia. The result of testing using a prescribed cyclic loading (AISC 341-10) showed that there was no indication of buckling nor fracture of the link on the assemblage as its plastic rotation reached 0.08 rad and beyond, and no decreasing strength indicated in the hysteretic curve. At the same time, there was no indication of any damage (yielding, buckling) in coupling beam (web, flange, end-plate, bolts) and column. Yielding is localized only in web and flange of the link. The Indonesian shear link specimen showed higher values of shear resisting force due to the higher value of its material yield-strength as compared to the Japan link specimen. However, both specimens showed the same ductility when they reached the maximum strength with its plastic rotation beyond 0.08 rad. as specified in the code (AISC 341-10).**

***Keywords*— damage controlled element, energy dissipation, replaceability, shear link**

I. INTRODUCTION

The use of RC core-wall in high-rise building has been very common, as to effectively reduce the lateral drift of the building due to the earthquake. Due to openings on the perimeter of core-wall, the coupling-beam at the opening between separated walls becomes the weak element that needs to be designed carefully. Lately, this system is also adopted in the steel construction. However, in the latter case, a link beam is added to connect coupling beam framing to the core columns. Several advantages of having this link beam are (1) additional energy dissipation device and (2) the replaceability of shear link after damage.

Research of the applicability of link beam in the eccentric braced frame system has been widely studied [1], [2], [3]. In 2014, Ji et al. [4] tested several shear links connected to coupling beam made of low yield strength steel (LY225). They found out that these shear link can be used effectively as a fuse and achieved inelastic deformation up to 0.15 rad. However, the experimental study of link beam using common steel material in Indonesia (BJ 37) is still limited. The purpose of this study is to evaluate experimentally the seismic behavior of shear links connected to coupling beams framing to the steel core frames. Furthermore, as the shear link is designed as a bolt-connected link which is inexpensive and simple to fabricate, it is expected that the damaged shear link can be replaceable after a severe earthquake. In this way, the proposed coupling beam with shear link system is expected to be an excellent seismic device for core-frames in a high-rise building.

II. MATERIAL AND METHOD

The sub-assemblage tested in this study is based on a hypothetical building with core frames consisting of two concentrically braced frames connected by a coupling beam (Figs. 1 and 2). The structure is a 20-story building located in Jakarta, Indonesia, with building plan shown in Fig. 1. In the x-direction, this building adopts concentrically braced frame system, while in the y-direction, it uses core frame system where a steel link beam is placed in the mid-span of each coupling beam so that the seismic energy is expected to be dissipated by steel link beams (Fig. 2). The link beams as mechanical damper could be designed as the only element to perform plastic deformation, while all other structural elements in the building are designed to remain elastic. This will certainly control the building damages due to a severe earthquake to be located mostly at the link beams, similar to the mechanism of the eccentrically braced frame system.

The gravity loads assigned to each floor were: the dead load due to self-weight; the super-imposed dead load of 50 kg/m^2 for roof story and 150 kg/m^2 for other stories; the live load of 100 kg/m^2 for roof story and 250 kg/m^2 for other stories. Meanwhile, earthquake load was analyzed using response spectrum analysis. It was assumed that the soil type of this building is soft soil (SE). The analysis and design of these buildings were carried out using SAP 2000 [6] according to Indonesian Seismic and Steel Building Code SNI 1726:2012 [7] and SNI 1729:2015 [8]. There are four major steps involved in the design procedure: (1) determination of building geometry, initial element size, and load criteria, (2) design of link element as a deformation-controlled element, (3) design of other elements as force-controlled elements using capacity design concept, and (4) checking of failure mechanism through pushover analysis. More in-depth explanation related to the numerical modeling could be found in Moestopo et al. [9]

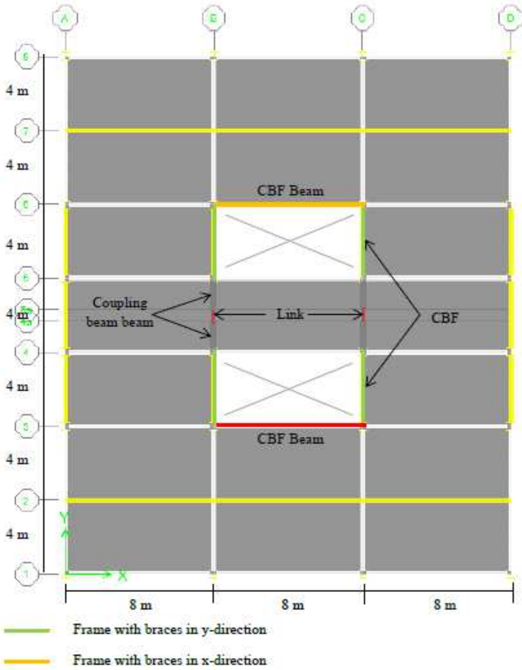

Fig. 1 Building plan

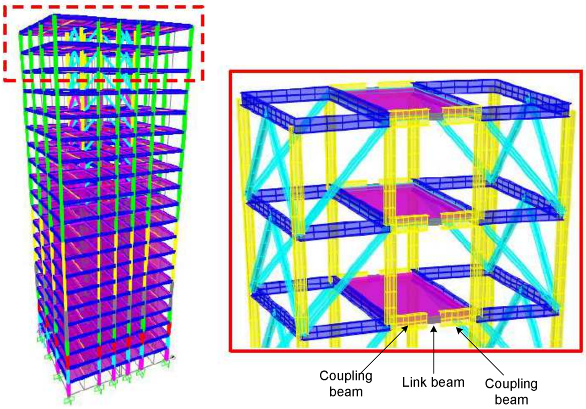

Fig. 2 Three-dimensional view of core frame system

A. *Test Setup*

Based on the numerical analysis described in the previous section, a half-scale sub-assemblage was fabricated and loaded with cyclic loading test to investigate the seismic performance of the shear link and the coupling beam. The span between two columns of the sub-assemblage is 2000 mm, whilst the height of the column is modeled from mid floor to mid floor and taken as 2000 mm as well (Figs. 3 and 4). The columns use steel section WF 350x350x12x19, the coupling beams use WF 400x200x8x13, and link beam uses WF 150x75x5x7. The material for columns and coupling beams was made from Indonesian steel (BJ 37 or A36 equivalent).

As mentioned in the introduction, research of link beam connected to coupling beam using A36 steel is still limited. The only test parameter is the source of material for the link. Specimen JPN used Japanese steel (SS400) and fabricated in Japan, while Specimen INA used Indonesian steel (BJ 37) and fabricated in Indonesia. The material used for the sub-assemblage is summarized in Table 1.

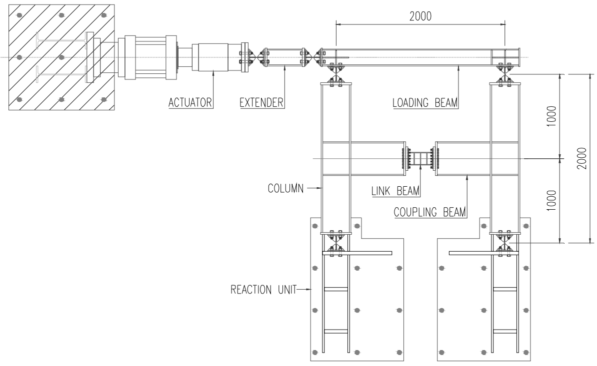

Fig. 3 Plan of sub-assemblage

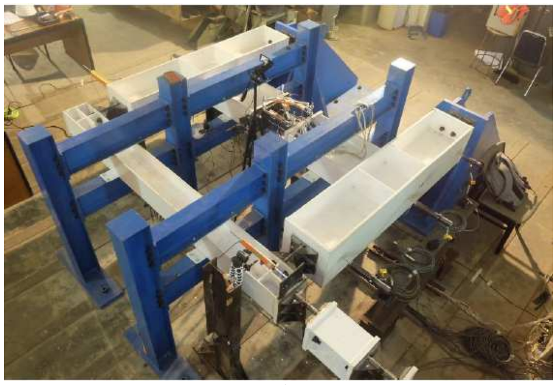
Fig. 4 Photo of the test setup

TABLE I
MATERIAL FOR SUB-ASSEMBLAGE

Spec	Link Beam	Coupling Beam	Column
JPN	SS400	BJ 37	BJ 37
INA	BJ37		

The link beam is bolt-connected to coupling beam using high tension bolt (Fig. 5). After the test, the link specimen can be replaced with the new one. In order to ensure the replaceability, coupling beams and columns will be designed to remain elastic throughout the test as will be presented in the following section.

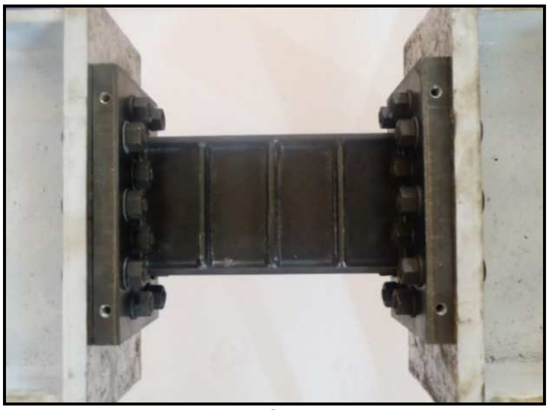

Fig. 5 Link specimen

B. Specimen Design

To ensure the replaceability of link specimen, columns and coupling beams will be designed to remain in elastic throughout the loading history and therefore, a capacity design is adopted. The following paragraphs show the capacity design check to ensure no yielding of coupling beams and columns would occur during the test.

The nominal shear strength capacity, V_n of a link beam (WF 150x75x5x7) can be calculated using Equation (1):

$$V_n = 0.6 \times F_y \times \left[\left(D - 2t_f \right) \times t_w \right] \quad (1)$$

where F_y is the nominal yield strength of steel section, D is the height of section, t_f is the thickness of the flange, and t_w is the thickness of the web. The term $(D\text{-}2t_f)\times t_w$ represents the area of the web. By substituting all the values, the nominal shear strength capacity of a link beam V_n is 97.92 kN as shown in Equation (2):

$$V_n = 0.6 \times 240MPa \times \left[\left(150 - 2 \times 7mm \right) \times 5mm \right] \quad (2a)$$

$$V_n = 97.92kN \quad (2b)$$

Using the capacity design concept, the required shear for coupling beam, $V_{u,cb}$ is taken as the calculated nominal shear strength capacity of a link amplified by two factors, i.e., R_y and over-strength factor, as shown in Equation (3):

$$V_{u,cb} = V_n \times R_y \times overstrength\ factor \quad (3)$$

where the value of R_y is taken as 1.5 which represents the ratio of expected material yield strength to the minimum yield strength and the over-strength factor is taken as 1.5. Hence, the required shear force for coupling beams, $V_{u,cb}$ is 220.32kN as shown in Equation (4):

$$V_{u,cb} = 97.92kN \times 1.5 \times 1.5 = 220.32kN \quad (4)$$

Knowing the shear force acting on coupling beam, the moment acting on coupling beam $M_{u,cb}$ can be calculated using Equation (5):

$$M_{u,cb} = V_{u,cb} \times \ell_{cb} \quad (5)$$

where ℓ_c is the length of a coupling beam, and is taken as 900 mm. Hence, the required moment acting on coupling beam, $M_{u,cb}$ is 198.29kN-m.

On the other hand, the design plastic moment of a coupling beam (WF 400x200x8x13) is equal to 277.77 kN-m as calculated in Equation (6):

$$\phi M_{n,cb} = \phi \left(Z_x \times F_y \right) = 277.77kN - m \quad (6)$$

where Z_x is plastic section modulus and taken as 128592 mm^3.

Comparing the design plastic moment and required moment at coupling beam, $\phi M_{n,cb} / M_{u,cb} = 1.26$, it can be concluded that the design of the coupling beam is sufficient and no yielding of coupling beam would occur when the link beam reaches its shear capacity.

Meanwhile, the acting axial and moment force on columns were obtained from sub-assemblage analysis using SAP2000. The values of axial and moment forces are then amplified using two factors, i.e., R_y and over-strength factor as shown in Equations (7) and (8):

$$P_{u,c} = 119.55kN \times R_y \times overstrength\ factor \quad (7a)$$

$$P_{u,c} = 119.55kN \times 1.5 \times 1.25 = 224.16kN \quad (7b)$$

$$M_{u,c} = 179.33kN - m \times R_y \times overstrength\ factor \quad (8a)$$

$$M_{u,c} = 179.33kN - m \times 1.5 \times 1.25 = 336.24kN - m \quad (8b)$$

where the over-strength factor for the column is taken as 1.25 for this study.

The design axial force and moment of a column are calculated using Equations (9) and (10), respectively:

$$\phi_c P_{n,c} = 0.9 \times A_g \times F_{cr} \quad (9a)$$

$$\phi_c P_{n,c} = 0.9 \times 17390mm^2 \times 233.82MPa \quad (9b)$$

$$\phi_c P_{n,c} = 3659.73kNm \quad (9c)$$

where A_g is the gross area, and F_{cr} is the critical inelastic compressive stress.

$$\phi_b M_n = 0.9 \times Z_x \times F_y \quad (10a)$$

$$\phi_b M_n = 0.9 \times 2493182mm^3 \times 240MPa \quad (10b)$$

$$\phi_b M_n = 538.53kNm \quad (10c)$$

Finally, the capacity of the column is checked using axial-moment interaction equation as shown in Equation (11):

$$\frac{P_u}{2\phi_c P_c} + \frac{M_u}{\phi_b M_n} = 0.03 + 0.62 = 0.65 < 1.0 \quad (11)$$

Equation (11) shows that column capacity is adequate to resist combined axial and moment load.

C. Instrumentation

There are two main data that were acquired through the LVDT, which configuration is shown in Fig. 6. First, is the link rotation which is acquired through the diagonal configuration of LVDT 2, 3, 4, 5, 6, 7, 8, and 9 of both faces of the link (Fig. 7). The second one is frame drift. Frame drift or column drift is calculated as the average between the left and right column drift. Left column drift is measured by using LVDT 12, 13, and 14 while right column drift is measured by using LVDT 15, 16, and 17. LVDT 1 and 18 is used to measure the horizontal displacement on actuator and loading beam.

There are also LVDTs that were used to measure potential slip. LVDTs 10 and 11 are used to measure potential slip on link-to-coupling beam connection. LVDT 21 and 22 are used to check potential slip between the reaction unit and strong floor.

There were ten single gauges and four rosettes that were used on the specimen. Four rosettes (SG 7, 8, 9, 10) were used on one side of the web. SG 11, 12, 13, and 14 were used to measure the strain of the flange of the link. One gauge was used on each column (SG 1 and 4). Two gauges were used on top, and bottom flange of coupling beam ends near column (SG 2, 3, 5, and 6). The locations of strain gauges are indicated in Figs. 6 and 7.

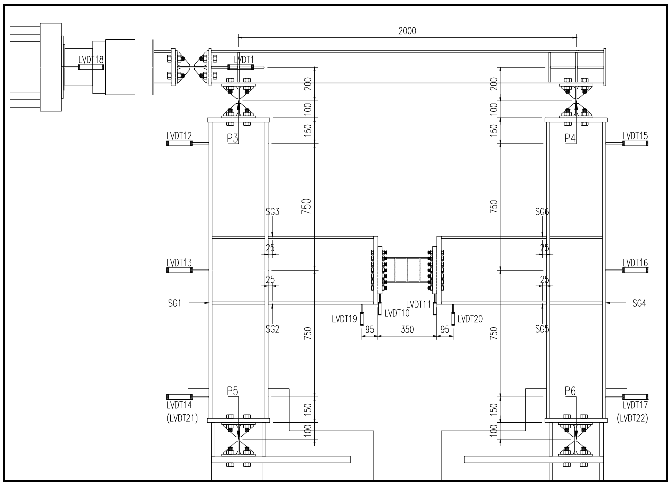

Fig. 6 Instrumentation plan

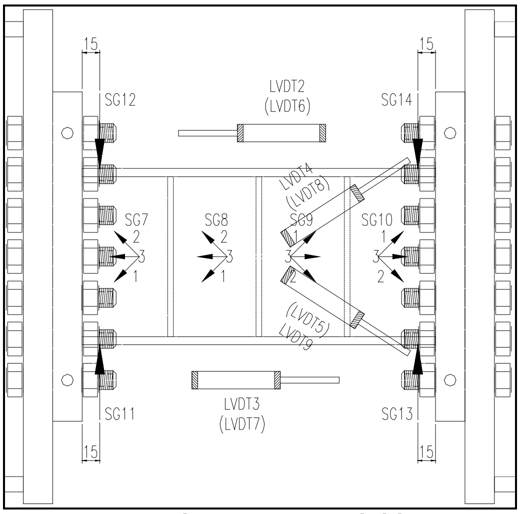

Fig. 7 Detail instrumentation on link beam

D. *Loading Protocol*

The AISC [9] loading protocol adopted in this study is the one used for cyclic tests of link-to-beam moment connections in eccentrically braced frames as tabulated in Table 2.

However, the source of input protocol for the horizontal actuator is from horizontal displacement. So, a transformation rule from link rotation to horizontal displacement must be derived using the geometry. The link rotation angle (γ) corresponds to the horizontal displacement at LVDT 12, (Δ_{LVDT12}) can be expressed in Equations (12) to (14) and illustrated in Fig. 8.

TABLE III
LOADING PROTOCOL

No.	Number of Cycles	γ [radian]
1	6	0.00375
2	6	0.005
3	6	0.0075
4	6	0.01
5	4	0.015
6	4	0.02
7	2	0.03
8	1	0.04
9	1	0.05
10	1	0.07
11	1	0.09
12	1	0.02

$$\gamma = \frac{L}{e}\theta \tag{12}$$

$$\theta = \frac{\Delta_{LVDT12}}{H_{12}} = \frac{\Delta_{LVDT12}}{1750mm} \tag{13}$$

$$\Delta_{LVDT12} = \frac{\gamma \times e}{L} H_{12} = \frac{\gamma \times 300mm}{2000mm} 1750mm \tag{14}$$

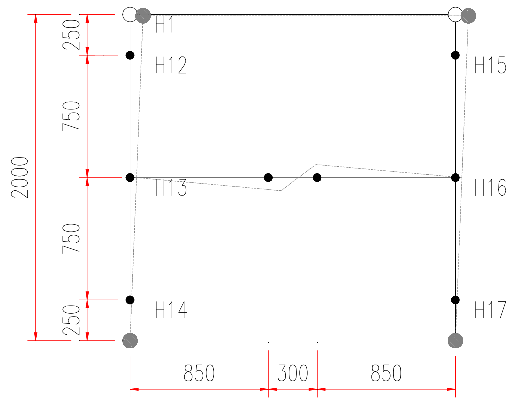

Fig. 8 Transformation from link rotation to displacement target

Hence, the input displacement into the actuator is set as shown in Fig. 9.

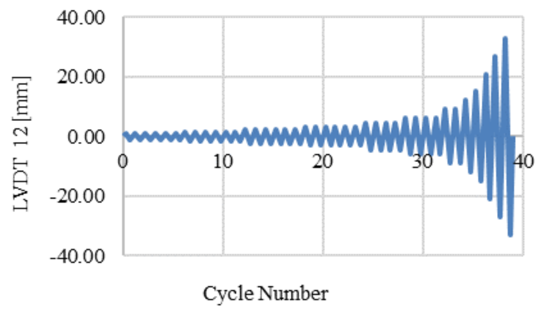

Fig. 9 Loading protocol (horizontal displacement parameter)

III. RESULT AND DISCUSSION

A. Material Test

Tensile tests were carried out for all material used in the experiment. Columns, coupling beams, and all link specimens from Indonesia (INA) were fabricated from BJ37 steel (F_y = 240 MPa, F_u = 370 MPa), equivalent to ASTM A36 steel. All link specimens from Japan (JPN) were fabricated from SS400 grade steel (F_y=235 MPa, F_u = 400 MPa). Tensile tests were conducted following the Indonesian standard SNI 07-0371-1998 [10] which refers to JIS Z 2201 [11]. The results of the tensile test are shown in Tables 3 to 6. The average web yield strength for specimen INA was 365.32 MPa, while that for specimen JPN was 339.63 Mpa. The average flange yield strength for columns and coupling beams were 290 MPa and 388 MPa, respectively.

TABLE IIIII
AVERAGE TENSILE TEST RESULT OF LINK BEAM

Specimen	Location	F_y	F_{ue}	ε_u	F_{ye}/F_{ue}
		[MPa]	[MPa]	[mm/mm']	[-]
INA	web	365.32	498.14	0.3049	0.73
INA	flange	344.46	444.75	0.2450	0.78
JPN	web	339.63	452.41	0.3007	0.75
JPN	flange	323.64	449.66	0.4023	0.72

TABLE IVV
AVERAGE TENSILE TEST RESULT OF COUPLING BEAM

Specimen	Location	F_{ye}	F_{ue}	ε_u	F_y/F_u
		[MPa]	[MPa]	[mm/mm']	[-]
INA	web	435.7	545.6	0.1392	0.80
INA	flange	388.8	518.3	0.1998	0.76

TABLE V
AVERAGE TENSILE TEST RESULT OF COLUMN

Specimen	Location	F_{ye}	F_{ue}	ε_u	F_{ye}/F_{ue}
		[MPa]	[MPa]	[mm/mm']	[-]
INA	web	299.8	459.6	0.2214	0.66
INA	flange	290.0	453.94	0.1699	0.64

B. Specimen JPN

The plot of hysteretic loops between link shear vs. total link rotation is shown in Fig. 10. Specimen JPN was loaded until the link collapsed. The maximum link shear force was 247.6 kN, which occurred at total link rotation of γ = 0.121 rad. The maximum link plastic rotation was obtained: γ_p = 0.109 rad at last pull cycle and γ_p = 0.1160 rad at last push cycle or average of γ_p = 0.112 rad. The test was terminated after fracture occurred at the bottom left of the flange (Fig. 11).

Based on the failure mechanisms described in the previous chapters, the link element is yielded as expected, while all other elements (columns and coupling beams) remain in an elastic stage. Strain gage measurement presented in Figs. 12 and 13 indicate the stresses occurred at columns and coupling beams respectively were still in the elastic range.

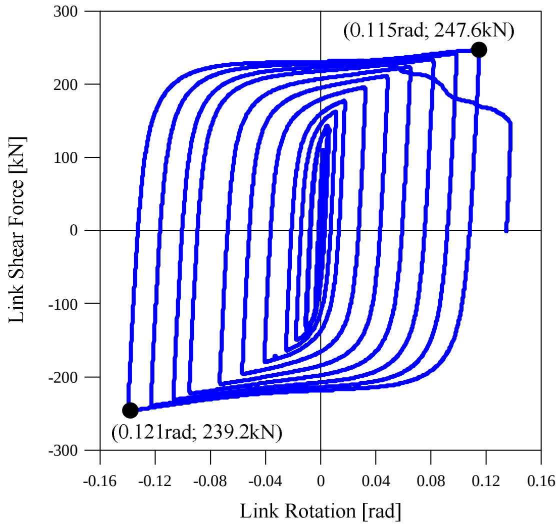

Fig. 10 JPN: Link shear vs. Total link rotation

Fig. 11 Fracture at the bottom left flange and final deformation of link specimen JPN

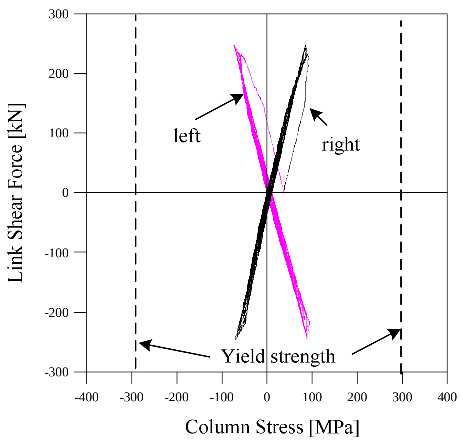

Fig. 12 JPN: load vs. column stress

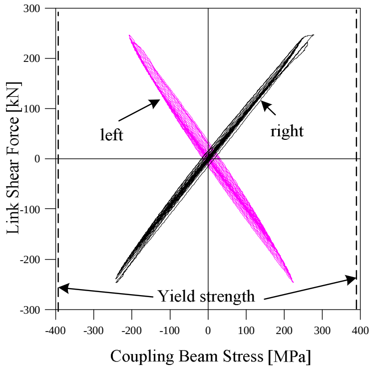

Fig. 13 JPN: load vs. coupling beam stress

C. *Specimen INA*

The plot of shear link force against link rotation (Fig. 14) shows that the link attained its maximum shear-force of 267.2 kN at total link rotation of 0.113 rad or plastic rotation of 0.111 rad, as the test was terminated after fracture occurred at the bottom link flange (Fig. 15). Rosette strain gages measurement indicated that the first yielding of link web occurred at cycle 24, which corresponds to link rotation of 0.0028 rad.

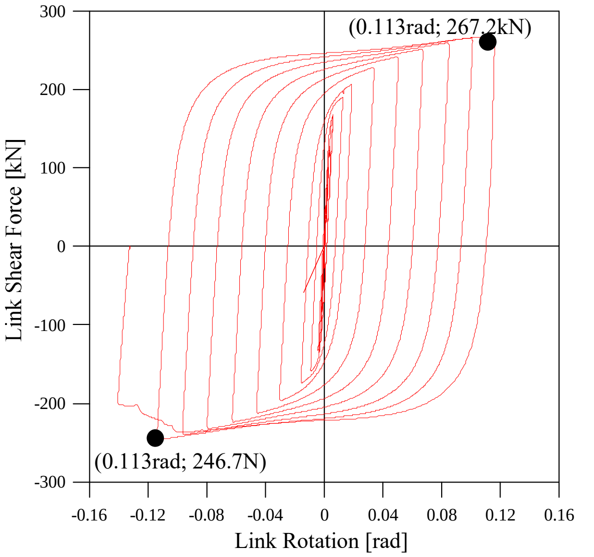

Fig. 14 INA: Link shear force vs. total link rotation

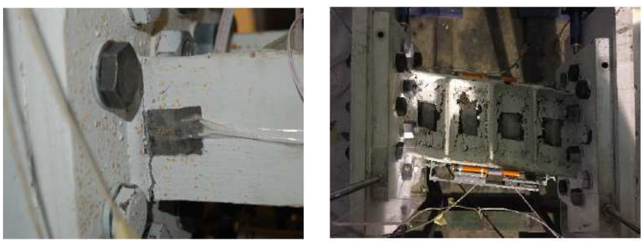

Fig. 15 Fracture at the bottom left of the flange and final deformation of link specimen INA

Figs. 16 and 17 indicate that no yielding of columns and coupling beams occurred during the test.

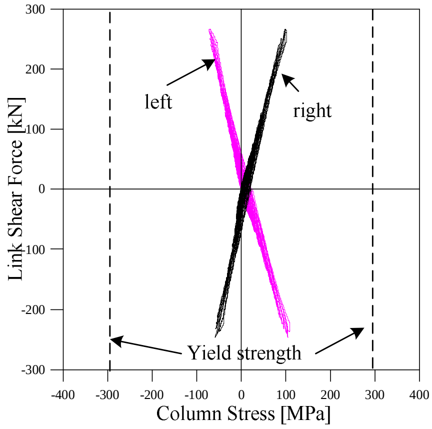

Fig. 16 INA: load vs. column stress

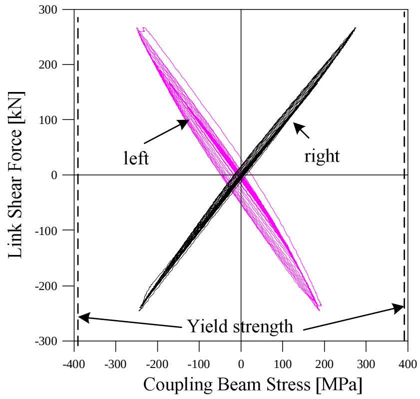

Fig. 17 INA: load vs. coupling beam stress

D. *Comparisons of Seismic Performance*

Fig. 18 shows the comparison of two specimens (Specimen JPN and Specimen INA) after cyclic loading test. Specimen made of Indonesian steel (Spec INA) showed a slightly higher shear capacity than that made of Japanese Steel (Spec JPN). This is due to a slightly higher yield strength of the material (Tables 3 and 4). The average web yield strength of BJ 37 was 365.32 MPa, while that of SS400 was 339.63 MPa. Meanwhile, both links performed satisfactorily in the AISC specified loading test and were capable to sustain larger cyclic deformation beyond the target of 0.08 rad plastic link rotation angle.

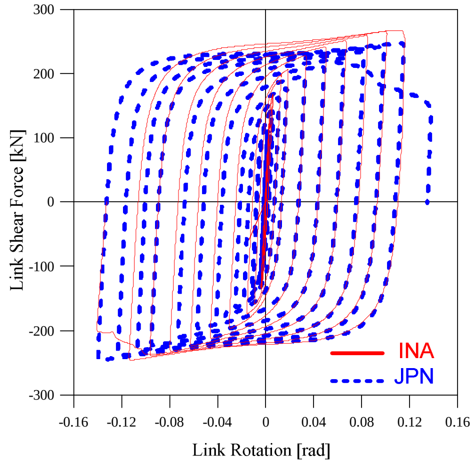

Fig. 18 Comparison of hysteretic loops

Moreover, comparisons of secant stiffness degradation and cumulative energy dissipation of these two specimens are shown in Figs. 19 and 20, respectively. They indicate that these two specimens performed identically and showed no appreciable difference regardless of the difference in material. This verifies a good seismic performance of the link beam attached to coupling beam system.

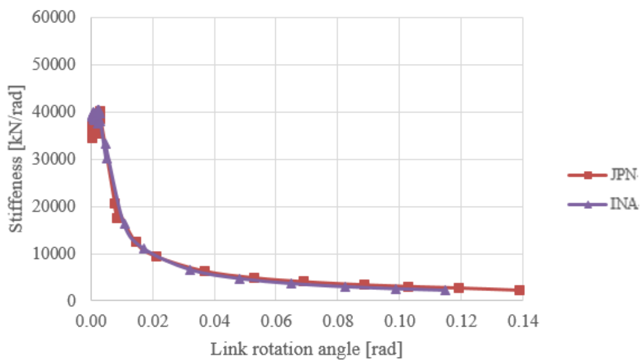

Fig. 19 Comparison of secant stiffness degradation

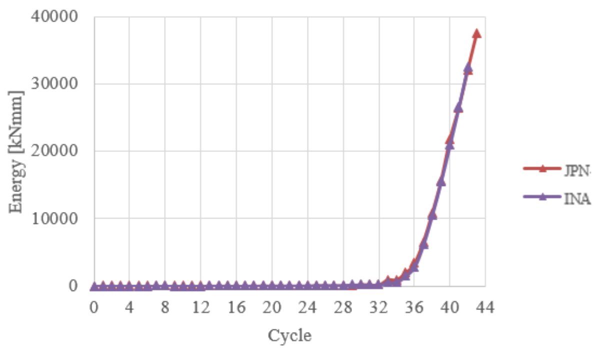

Fig. 20 Comparison of energy dissipation

E. *Replaceability of Link Beam*

In this experimental work, the sub-assemblage (column and coupling beam) was used throughout the experimental study. After each test, the link specimen was removed, and the inspection was conducted to the column, coupling beam, and the end-plate connection between the coupling beam and the link specimen (Figs. 21 and 22).

As discussed previously, after each testing, no indication of yielding in coupling beam and column of the sub-assemblage were observed. Thus, the sub-assemblage did not experience any permanent deformation.


Fig. 21 Link to coupling beam coupling beam connection before testing


Fig. 22 Coupling beam after testing and before the next testing (top view)

Further inspection of the end-plate connection of the coupling-beam after the completion of each testing showed that the followings are consistently performed:

- There was no indication of damage on the end-plate surfaces as well as around the perimeter surface of all bolt-holes in both end-plates of coupling beam.
- There was no indication of damage in all bolts that were used in connecting end-plates of both coupling beams and link specimen.
- There was no significant slip occurred between end-plate of coupling beam and end-plate of link specimen.

The experimental work verified the replaceability of the link in the sub-assemblage and confirmed the use of the seismic device developed in this research project.

IV. Conclusion

Based on the cyclic test of two specimens made of the Japanese and Indonesian material, results and conclusions have been obtained as follows:

- A seismic resisting structural system of steel core-frames has been developed. The system consists of two concentrically braced frames connected with coupling beams and shear links. The shear link has been designed by a capacity design to yield and dissipate the energy while the coupling beam and other elements of the core frames (beam, column and bracing) remain elastic.
- The experimental work had been conducted on a model of half-scale sub-assemblage of core steel frame structure with its links made in Japan (specimen JPN) and Indonesia (specimen INA). The result of testing using a prescribed cyclic loading (AISC 341-10) showed no indication of buckling nor fracture of the link on the assemblage as its plastic rotation reached 0.08 rad and beyond, and no

decreasing strength indicated in the hysteretic curve. At the same time, there was no indication of any damage (yielding, buckling) in coupling beam (web, flange, end-plate, bolts) and column. Yielding is localized only in web and flange of the link. This verified the behavior of the link and the sub-assemblage as a seismic device, as specified in the code (AISC 341-10).

- The Indonesian shear link specimen showed higher values of shear resisting force due to its slightly higher value of yield-strength as compared to the Japan link specimen. However, both specimens showed the same ductility when they reached the maximum strength with its plastic rotation beyond 0.08 rad.
- The result of cyclic loading test has confirmed that damaged link can be easily removed and replaced with the new link with the same geometry and the same bolt configuration since the remaining structure (coupling beams, columns and its connections) remain elastic and they always returned to its original position after the damaged link was removed.

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