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Extended End-Plate Link-Column Joints in Eccentrically Braced Frames

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ABSTRACT

Seven link-column flange joints, representing parts of a typical eccentrically braced frame (EBF), were built and tested using a controlled cyclic displacement applied to the link. In these subassemblages, the links were connected to the column flanges using bolted extended end-plate (EEP) connections. The tests were conducted to investigate the stiffness, strength, tendency to slip and ductility of the bolted joint and the behaviour of its components. It was found that EEP connections are suitable for link-column joints of EBFs. Links with properly designed EEP connections sustained the same cyclic displacement history and reached the same ductility levels as welded connections.

INTRODUCTION

The most important members of an eccentrically braced frame (EBF) are its shear links. The shear links are eccentric elements formed in the beam by deliberately offsetting the brace from the beam-column joint. Limiting the length of the links maintains the lateral stiffness of the EBF close to that of a concentrically braced frame (CBF). Stiffened shear links can sustain severe deformations and dissipate energy from the input ground motion.

Malley and Popov (1983) tested few link to column flange bolted-web, welded-flange connections in a set-up simulating an EBF joint. Based on the results of a limited number of experiments, they concluded that for cases of large ductility demand, welded link-column connections are recommended. As the Canadian design code for steel structures (CSA, 1989) recognizes EBFs as a reliable framing system for seismic application, fully welded link-column connections in EBFs are recommended.

It is normally difficult to ensure the weld quality of a field welded connection. In addition, field welds are affected by weather conditions. Poor welds can result in serious brittle fractures at this sensitive joint of the link element which is required to behave in a ductile manner. The experience with the steel connection failures during the Jan. 1994 Northridge earthquake, suggests that the design and installation of welded connections in seismic applications, requires review. The use of bolted extended end-plate connection (EEP) may provide a viable alternative to the field welded connection in seismic applications.

All the available research on the EEP connection dealt with beam-column joints in moment resisting frames (MRFs). Significant differences exist in the loads and deformation. of the connection in cases of a MRF joint and a link-column joint in an EBF. There is need

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and current interest in investigating the behaviour of bolted EEP connection for link-column joints.

TEST SETUP, LOADING ROUTINE AND TEST SPECIMENS

The investigation of the behaviour of bolted link-column joints is pursued experimentally. Tests are conducted on seven connection specimens. The test setup is schematically shown in Fig.1. The left end of a replaceable link (end A) is connected to a large column stub using the EEP connection. The right end of the link (end B) is connected to a long beam of the same cross-section as the link section. The link and the beam attachment are secured together by 12-25.0 mm diameter A490 bolts connecting two steel end blocks. To prevent failure from occurring outside the link and its joint, the beam is strengthened by flange doubler plates of 6 mm thickness. Applying two equal displacements at points B and C simultaneously will cause the initial elastic link end-moments to be unequal. This represents the elastic moments in a link in a typical EBF. The applied load is displacement controlled and consists of cycles of displacements applied quasi-statically until failure of the specimen is reached. It starts with a displacement of ± 6 mm in the first cycle. This is followed by two cycles of ± 12 mm displacement and two cycles of displacement of ± 20 mm. After the first five cycles, the displacement is increased by an increment of 6 mm every two cycles until failure.

The link sections are selected to be W200 X 15. The material for all the specimens, the link stiffeners and the column stiffeners is CSA-G40.21-M300W steel (equivalent to 44 ksi structural grade). Design of the link length "e" and web stiffeners spacing "a" is based on the criteria given by CSA (1989). The web stiffeners are designed according to the criteria given by Malley and Popov (1983). These stiffeners are one sided, full depth, 6 mm thick and all around fillet welded to the link section flanges and web.

The link-column connections are designed according to several design approaches such as CSA (1989), Ghobarah et al. (1990) and the American Institute of Steel Construction method taken after Tsai and Popov (1988). The purpose is to check the suitability of available EEP design approaches for link-column joints. A summary of the design features of each specimen is given in Table 1.

EXPERIMENTAL OBSERVATIONS

The performance of the specimen during the test is evaluated from measurements of displacements, applied load and strains. The shear-displacement and moment link-deformation angle were developed from the test data (Ghobarah and Ramadan, 1994)

Specimen 1: This specimen was the only design that employed end-plate stiffeners. It suffered flange local buckling at the second panel of the link at the toe of the end-plate stiffener. Flange buckling at end B was initiated and increased especially at the upper flange. Severe tearing of the upper flange was observed during cycle 6 at which the test was terminated. The maximum shearing force reached is 1.25 V_p where V_p is the plastic shear capacity of the link section. The maximum moment developed at end A at the start of the second panel (at the toe of the end-plate stiffener) reached 0.9 M_p. The maximum moment developed at end B reached the plastic moment capacity of the link section, M_p.

Specimen 2: For this specimen, initial yield and subsequent flange local buckling occurred at end A first. Failure of the specimen occurred prematurely when the weld connecting the link flange to the end-plate at end A, fractured. It is worth noting that the weld was made in controlled laboratory environment by qualified welders.

Specimen 2R: The initial phase of this link performance was similar to specimen 2. However, this specimen sustained a severe load history similar to the fully welded specimen 6. It also failed in a typical shear link manner where severe flange local buckling at end A induced web buckling. Finally, the link lost its load carrying capacity. The maximum shear developed is $V_{max} = 1.4\ V_p$, the maximum moment developed at end A is $MA_{max} = 1.1\ M_p$ and at end B is $MB_{max} = 1.1\ M_p$.

Specimen 3: Initial yielding of this specimen was similar to specimens 2 and 2R. However, failure of this link was localized in its end-plate joint in the form of bolt fractures. Fig. 2 shows the moment-link deformation angle hysteresis loops for specimen 3. The V_{max}, MA_{max} and MB_{max} developed by this link are 1.3 V_p, 1.0 M_p and 1.1 M_p, respectively.

Specimen 4: The initial yield of this specimen occurred in the flanges of the link at the first panel similar to previous specimens. Failure occurred at end A, when the weld connecting the end-plate to the link flanges, fractured. Fig.3 shows specimen 4 at failure where the end-plate deformation is visible. The V_{max}, MA_{max} and MB_{max} developed by this link are 1.35 V_p, 1.1 M_p and 1.1 M_p, respectively.

Specimen 5: The initial yield of this specimen occurred at end B which is different from observations of specimens 2R and 4. Failure of the specimen occurred at the connection where the end-plate fractured at the toe of the weld between the end-plate and the link flange. The shear-displacement hysteresis loops for this specimen are shown in Fig. 4. The V_{max}, MA_{max} and MB_{max} developed by this link are 1.25 V_p, 0.9 M_p and 1.1 M_p, respectively.

Specimen 6: The behaviour of this fully welded link specimen is similar in every detail to the response of specimen 2R. This includes the initial yield stage, mode of failure and maximum developed shear forces and moments.

STIFFNESS OF EEP CONNECTIONS

From the stiffness point of view, the comparison between the degradation of the connection stiffness with the number of inelastic excursions for the different links is shown in Fig. 5. The stiffness of a connection is calculated from its moment-rotation hysteresis loops. The stiffness is defined as the slope of the moment-rotation curve during the elastic stages of loading and unloading. The stiffness (K) during different loading cycles of the test for each specimen is normalized by the initial stiffness of specimen 2R (K_{i2R}) since each specimen had its own K_i depending on the degree of fixation of the connection. The variation of stiffness of different specimens is shown in Fig. 5. Specimen 2R suffered no loss of its stiffness and behaved in a similar fashion to the fully welded joint, specimen 6. Its final stiffness at failure was exactly equal to its initial stiffness. Specimen 4 suffered a moderate loss of its initial stiffness. Its stiffness at failure was a little more than 50% of the initial value. Specimen 3 suffered a more pronounced loss of stiffness. Its final stiffness deteriorated to about 35% of its initial value which is equivalent to 30% of K_{i2R} (Fig. 5). However, the poorest behaviour of all specimens was that of specimen 5. Its performance is regarded as unacceptable since the loss of its stiffness started early during the test at relatively low link deformation angle. At the same time, its final stiffness was less than 20% of its initial value, which is equivalent to 12% of K_{i2R}.

SLIPPAGE OF EEP CONNECTIONS

A feature of the behaviour of bolted connections is the possibility of slippage when subjected to high shear. Krawinkler and Popov (1982) tested several connections under cyclic loading and reported that slippage causes bolted connections to dissipate less amounts of energy. In case of shear links, the slippage problem may be further aggravated by the high shear forces that the connection is subjected to.

In the current research, vertical slippage of the connections was measured by 2 Linear Variable Displacement Transducers (LVDTs). In specimen 1 connection, end plate stiffeners were used. Its bolts suffered no slippage and even performed elastically during the entire test. The bolts in specimen 2R were designed according to the Canadian standards CSA (1989) to prevent slippage. Experimental observations confirmed that this connection suffered virtually no slippage. On the other hand, specimen 3 had the smallest bolts designed according to AISC specifications (taken after Tsai and Popov, 1988). The bolt vertical slip in mm is plotted against the loading steps in Fig.6 for specimen 3. Specimen 3 and 4 suffered slippage during the tests while specimen 5 with the underdesigned end-plate suffered slight slip. The reason for this may be that the excessively flexible connection of specimen 5 shifted the higher moment to the other link end thus reducing the forces and moments on the bolts. It is noted that specimen 4 suffered slippage that is comparable to that of specimen 3 while specimen 2R showed the least slippage.

DUCTILITY OF BOLTED SPECIMENS

There are several ways to measure the ductility of a link and assess its performance. One of the common measures of ductility for shear links, is γ_{max} or the maximum sustained link deformation angle defined as the displacement at end B divided by the link length. Specimen 2R, 4 and 6 reached the highest γ_{max} among all specimens at a value of 0.084. Ricles and Popov (1987), conducted experimental investigations on shear links attached to a concrete slab. They concluded that the allowable link deformation angle γ_u should not exceed 0.06. Links developing γ_{max} in excess of that value caused considerable damage to the attached floor system. Later, they relaxed the allowable γ_u to 0.08. Specimens 1 and 2 suffered premature failures, reaching γ_{max} values of only 0.057 and 0.044 respectively. Specimens 2R and 4 were able to achieve the allowable link deformation angle reaching γ_{max} = 0.084. This is an indication of their superior ductility which is comparable to that of a fully welded link connection (the welded specimen 6 had γ_{max} = 0.084). All bolted specimens (except 1 and 2) were able to achieve high values of γ_{max}. Specimens 3 and 5 achieved γ_{max} equal to 0.071 in spite of suffering a sudden brittle failure in their connection components.

CONCLUSIONS

1. Links with rigid connections develop higher ultimate forces and dissipate larger amounts of energy in a ductile manner than links with flexible connections.
2. Links with properly designed bolted EEP connections as proposed by the no slip bolt design provisions of CSA (1989), sustained the same cyclic displacement history and dissipated equal amounts of energy as the carefully shop welded links.
3. It is advisable to design EEP connections for shear link-column joints to preform elastically even under severe load conditions. It is prudent not to rely on the connection to share in the energy dissipation mechanism.
4. High shearing forces developed by the shear links should be included in the

connection design.

5. Bolt holes of diameters near the minimum allowable size minimize the bolt slippage problem.

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Table 1 Design Features of the Tested Connections

Specimen No.	Bolt Size (mm)	End-plate thickness (mm)	Design Criteria
1	20	16	End-plate stiffeners
2	20	16	CSA (1989), no slip
2R	20	16	CSA (1989), no slip
3	12	12	Tsai and Popov (1988)
4	16	12	Ghobarah et al. (1990)
5	16	8	Under-designed end-plate
6	Fully welded link-column joint		

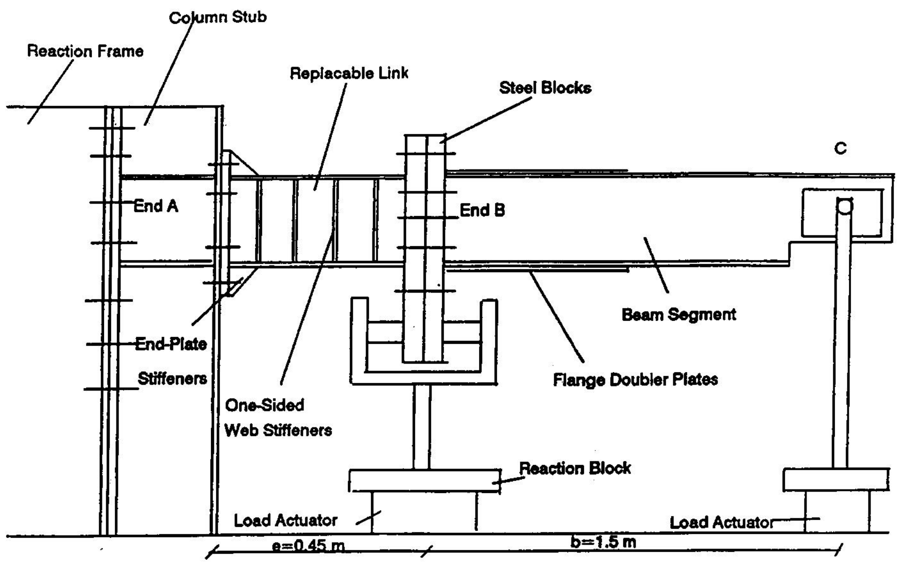

Fig. 1 Schematic of test setup

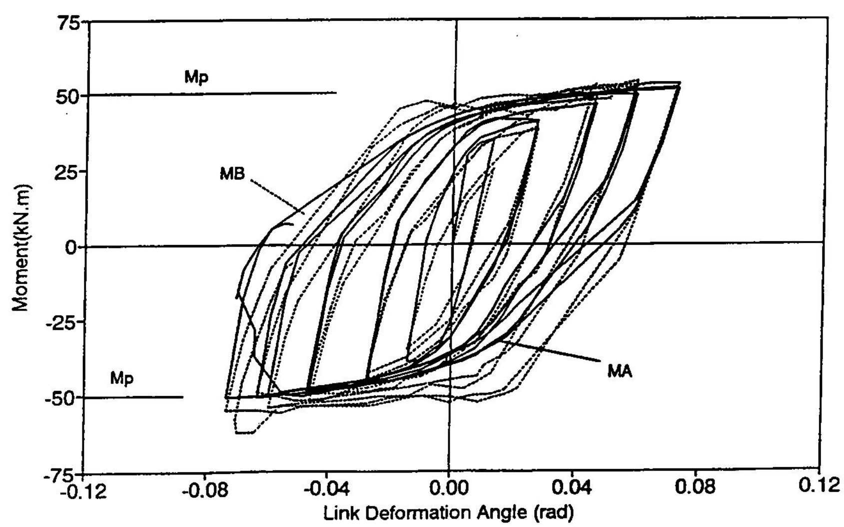

Fig. 2 Moment-link deformation angle hysteresis loops for specimen 3

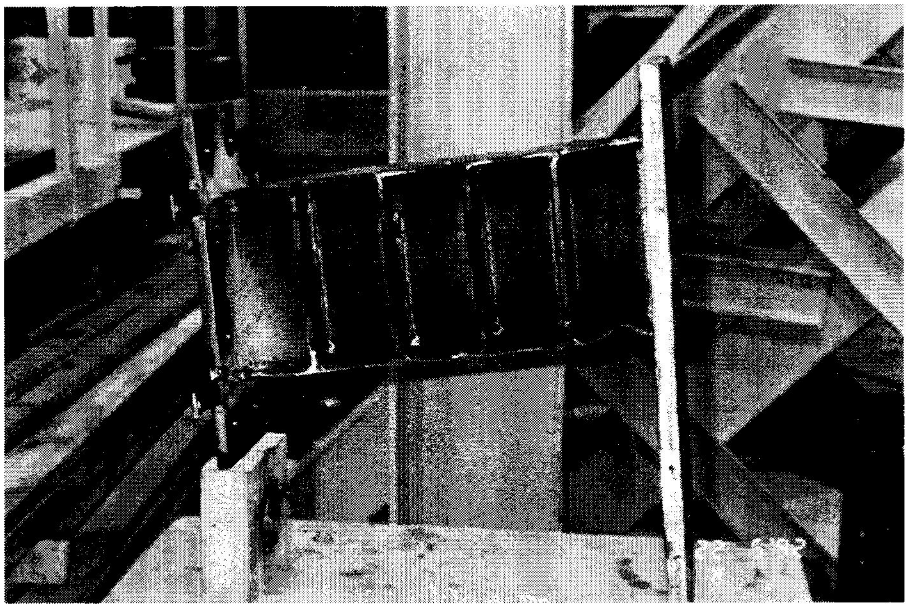

Fig. 3 Specimen 4 at failure

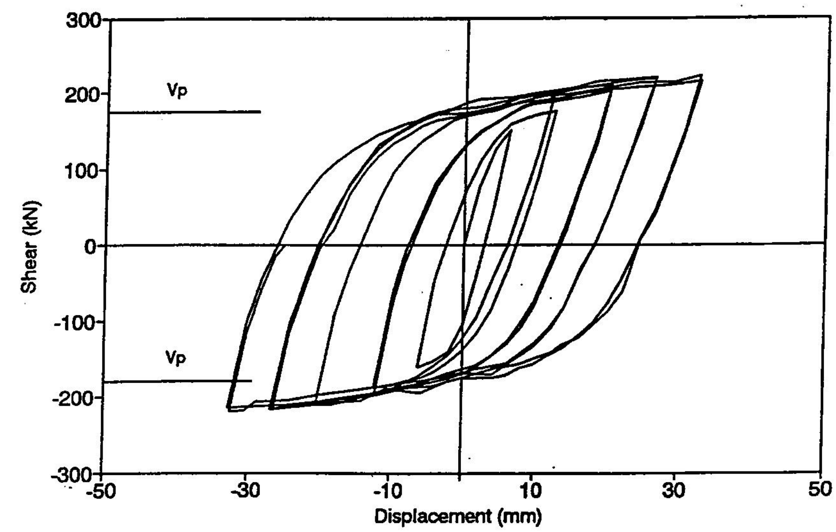

Fig. 4 Shear-displacement hysteresis loops for specimen 5

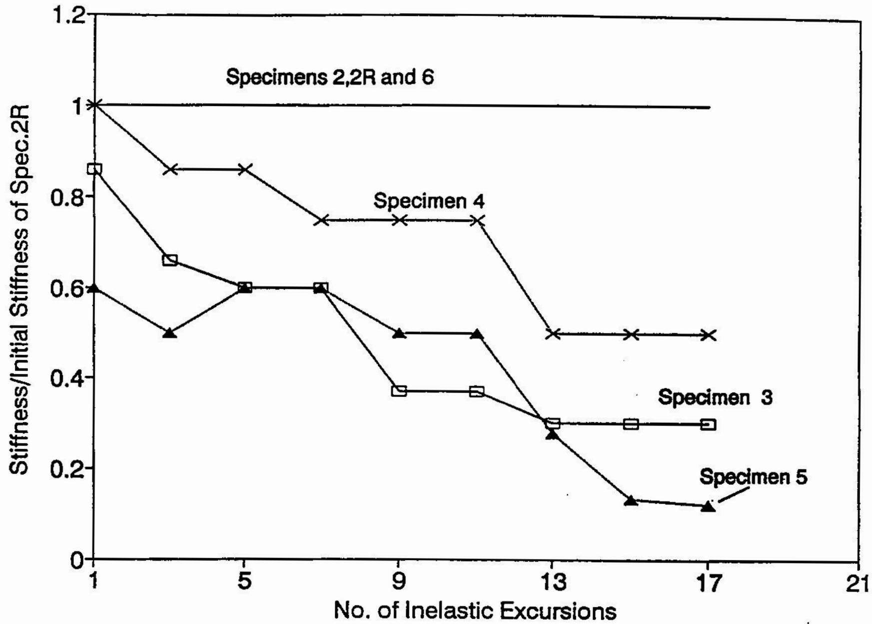

Fig. 5 Variation of connection stiffness K as a percentage of initial stiffness K_{i2R} of specimen 2R

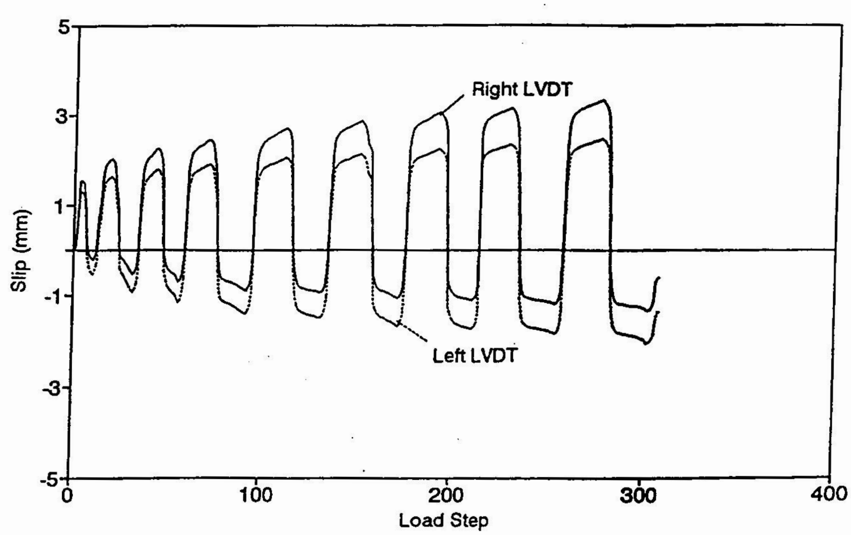

Fig. 6 Variation of vertical slippage with loading steps for specimen 3