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Seismic performance of retrofitted WFP connections joined to box column using ribs

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A B S T R A C T

In this paper, design methodology and retrofitting detail of welded flange plate (WFP) moment resisting connections have been proposed. Existence of defected complete joint penetration (CJP) groove weld at the only top flange of the connection and presence of the box column with different ductility were the main construction challenge of the mentioned connections, which are less investigated in the technical literature. Reference experiment was carried out based on the proposed design methodology. Experimentally validated analytical study was performed to evaluate the applicability of the mentioned design approach of retrofitting connection by rib plates. Ductility of the column and the number of utilized rib plates were considered as the main criteria to be focused. Potential of fracture in welds of connection, seismic performance of connection and failure mechanism were investigated in cyclically loaded specimens. The experimental and analytical results proved the applicability of the proposed design method. Also, highly ductile box columns guaranteed the acceptable performance of retrofitted connection based on AISC special moment resisting frame requirements. Finally, the acceptable limits of rib plate's design methodology were developed.

1. Introduction

A large number of various structures have been damaged during the Northridge earthquake in 1994. Thereby, different types of damages observed in steel structures. Brittle failure, existing of defective CJP groove weld, stress concentration in weld access holes and formation of plastic hinges in the column face were the most significant damages observed, although the engineering approaches had predicted that moment resisting frames had ductility and could withstand inelastic deformation in acceptable values [1].

Based on more investigations have been performed on the moment connections, two theories revealed for improvement of seismic design: strengthening the connections and weakening the beams [2]. Rib reinforcing is an efficient procedure for retrofitting the moment resisting connections. Technical literature introduces two types of rib reinforcing: rib with curved and extension parts, and rib with the triangular shape.

Chen et al. [3] tested six full-scale specimens to study the effects of reinforcing of pre-Northridge moment connections by rib with curved and extension parts in built-up box columns. The results revealed that retrofitting method has an acceptable level of performance. Although, the stress distribution in the box columns comparing to H-shaped columns was different. Two webs in the box columns caused approximately continuous stress distribution along the CJP groove weld, but the concentration at the edge of the flange was evident. In contrast, The H-shaped columns with one web had peak stress in the centerline of beams. It should be noted that this study has taken with a single rib plate.

Another investigation by Chen et al. [4] presented similar results using rib with the triangular shape in H-shaped columns. Two unreinforced specimens were compared with two single rib retrofitted specimens. The results showed that the local stress concentration has been reduced; therefore, the potential of fractures in weld access holes became less [5,6]. Plastic hinges were conducted to beam section away from the column face.

Lee et al. [2,7] by studying on the principal stress distribution in rib plate with the triangular shape, represented theory that called 'Equivalent strut model.' To prove this theory, four large-scale specimens were tested and the numerical models were used to verify them. Actually, in equilibrium formulations, the rib was replaced with a strut element. Based on their proposal theory, the difference of flexural stress between beam theory and finite element model was observed. The results of beam theory by considering the beam and rib plate as a joined part have shown unreliable stresses in CJP groove weld of the beam to the column. In other words, the force transferred from beam to column might be underestimated. All specimens showed ductile behavior

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during the experiments and passed 4% the story drift angle.

According to the high capacity of biaxial bending resistance [8], and also the lack of H-shaped column in some countries, box column is the most commonly used ones. Moreover, application of box column in the tall steel buildings with the high level of forces is crucial; however limited research for the beam to box column connections had been conducted [9].

Ghobadi et al. [10,11] tested seven WFP connections joined to box column including one with defective weld and six reinforced connections. Cyclic loading history was applied to the specimens, and T-stiffener retrofitting method was utilized.

This study retrofits the first defective specimen of Ghobadi et al. [10] by the Rib reinforcement method and tries to answer the question that "Whether the method of Rib reinforcing of connections can be applicable in presence of box column?" In this regard, the significance of column section compaction, the number of rib plates and the rib design methodology were considered in research. For this reason, the design methodology of the retrofitted connection was developed, and the experimental specimen was planned as a reference of investigation which showed an acceptable level of AISC [12] requirements for special moment resisting frames. Then parametric investigations were implemented to cover acceptable limits of design parameters of rib plate and column ductility. To reach the validation of design approach in parametric study, fracture potential of rib welds, the acceptable seismic level of connection and yield stress distribution of connection area were considered to interpreter the results. Finally, the convenient construction detail and the applicability limits of design methodology were presented.

2. Design methodology of rib plate and design of reference experiment

The WFP connections as shown in Fig. 1 have simple design steps based on FEMA-355D [13]. The area and length of top and bottom flange plate designed based on couple forces resulted from expected plastic bending moment of the beam. For transferring forces completely, these plates should be connected to column face by CJP groove welds. The shear forces of the beam were exerted to shear tabs because the stiffness of top and bottom flange plates were much less than shear tabs [14].

Sometimes in a WFP connection, in order to reducing stress in weld of top flange plate to the beam, the top flange plate has wider width and becomes narrower with the angle < 30°. The CJP groove welds are the most vulnerable part of the connection that it's behavior directly depends on welding quality. Fig. 1 shows studied type of connection with defective CJP groove weld or fillet weld with insufficient strength.

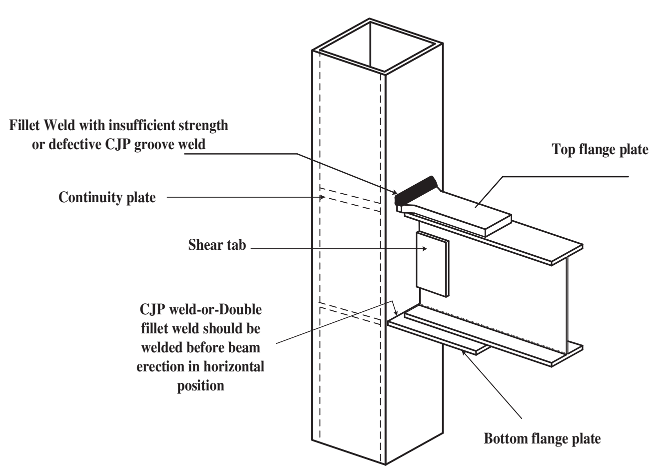

Fig. 1. Schematic detail of WFP connection with fillet weld or with defective CJP groove weld at top flange.

Based on the inconvenient welding detail of the top flange plate to the column, applying rib plates as a retrofitting program can be efficient. The rib plate plays the stiffener role that joins top flange plate to the column while the defective weld is failed.

As shown in Fig. 2, rib plate is connected to the top flange plate and column, using the fillet weld. To elaborate the rib plate design concept, forces distribution in rib plate sides should be illustrated. Axial force (T) resulted from the beam expected plastic moment in top flange plate acts as a shear force on the horizontal side of rib plate while it produces tension force on the vertical side. The moment caused by shear force on the horizontal side, due to its eccentricity from the centerline of vertical side, is converted to shear force along the vertical side and compression force along the horizontal side. This event derives from the displacement restraint of horizontal side.

Fig. 3 illustrates the applied forces diagram of rib plate. In concept of Rib design, the moment resulted from the shear force on the horizontal side of rib and is given in Eqs. (1)–(7). The fillet weld in the horizontal side carries the shear force and acts as longitudinal fillet weld. Meanwhile, fillet weld of the vertical side behaves as transverse fillet weld and also longitudinal fillet weld due to presence of the tension force and shear force, respectively. Therefore, in the calculation of weld design, the strength of transverse loading fillet weld was considered 1.5 times of longitudinal fillet weld in accordance with AISC specifications [12] and AWS [15]. The summary of connection design steps for the reference experimental specimen is presented in Table 1. The specification of the steel material was selected from Table 2. The design methodology is set based on LRFD method of AISC specification [12]. In the example calculation, the real yield stress was used instead of the expected yield stress of the material due to the availability of coupon test results.

$$P = w \times L \rightarrow w = \frac{P}{L} \tag{1}$$

$$\frac{H}{L} = \tan\alpha \rightarrow Arc\tan\frac{H}{L} = \alpha \tag{2}$$

$$\frac{h_x}{x} = \tan\alpha \rightarrow h_x = x\tan\alpha \tag{3}$$

$$M_x = w \times \frac{h_x}{2} \quad \rightarrow \quad M = \int_0^L w \times \frac{h_x}{2}dx \tag{4}$$

$$M = \int_0^L w \times \frac{x\tan\alpha}{2}dx = \int_0^L \frac{P}{L} \times \frac{\tan\alpha}{2}xdx = \frac{P}{L} \times \frac{\tan\alpha}{2} \times \frac{L^2}{2} \tag{5}$$

$$M = \frac{P}{4}L\tan\alpha = \frac{PH}{4} \tag{6}$$

$$\text{Shear force on vertical side} \rightarrow F_{v,\ \text{due to } M} = \frac{M}{L} \tag{7}$$

where L is length of horizontal side, H is length of vertical side, α is the angle of corner of Rib plate, P is shear force on horizontal side, w is linear force on horizontal side, and M is moment due to shear force on horizontal side.

3. Experimental program

To prove the accuracy of proposal design procedure, the exterior experimental specimen (RC6) was elaborated (see Fig. 4). The structural steel used in this experiment was ST37 type (similar to A36 steel). The mechanical properties of all used parts were determined by coupon tests, which is shown in Table 2. The electrode E6013 and E7018 were utilized for different welds. The electrode E7018 is known as high toughness weld metal which has a minimum specified Charpy V Notch toughness of 70 J at − 30 °C. Moreover, it provides more ductile welds and is essential for welding of critical connections, so the rib plates were welded by E7018 to the top flange plate and column [16,17]. On the other hand, low toughness electrode E6013 was used for old