Journal of Constructional Steel Research 137 (2017) 180–191

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

journal homepage: www.elsevier.com/locate/jcsr

Experimental investigation of panel zone in rigid beam to box column connection

Ebrahim Jahanbakhti[a], Nader Fanaie[a,*], Alireza Rezaeian[b]

[a] Department of Civil Engineering, K.N. Toosi University of Technology, Tehran, Iran
[b] Department of Civil Engineering, Karaj Branch Islamic Azad University, Karaj, Iran

ARTICLE INFO

Keywords:
Experimental investigation
Rigid connection
Continuity plate
Yield lines theory
Box column

ABSTRACT

This research presents the results of an experimental effort conducted on three full-scale rigid connections between I-beam and box-column. Accordingly, it is focused on the elimination of continuity plates from such connections through controlling the thickness of column flange. The appropriate thickness is determined by analytical method and then verified through experimental investigations of three different full-scale connections under AISC protocol of cyclic loading. Pre-qualified connections, presented in AISC (WUF-W) are selected for the test. They have the beam flange width to column flange width ratios of 0.53, 0.6 and 0.8, and beam flange width to beam depth ratios of 0.48, 0.72 and 0.65. According to the obtained results, panel zone remains elastic in all tested samples; and plastic hinges are formed in the beam and near the column flange. Moreover, the samples reach to the 6% story drift before experiencing the permissible strength degradation. Therefore the tested connections have satisfied criteria of special moment resisting frame (SMRF) according to AISC. According to the FEMA requirements for SMRF, no crack should be observed in the connection up to the story drift of 4%. However, in a sample of this experimental research, crack is observed in story drift of 3%.

1. Introduction

Column stiffener connections were used with full penetration groove weld and conservative design in the moment steel frames before Northridge earthquake (1994). The intention to conservatively design of such connections, regarding their significant effects on the distribution of strain and stress in the panel zone, resulted in their installations where no continuity plates were needed, even with the thickness over than needed. These plates are connected to the column flange by full penetration groove weld. Such welds are apt to the stress concentration and usually experience cracks in the root during execution. The flexural connections between beams and columns mostly experienced weld fractures in the Northridge earthquake [1]. In 1997–2002, the seismic guidelines pointed out the necessity of installing stiffeners and removing the design criteria [2]. Accordingly, more concise discussions are found about the stiffeners in the version 1999 of AICS seismic code, comparing to its previous versions, and more detailed in the following one [3]. Tremblay et al. studied the connection failure in the Northridge earthquake, comparing it with the expected behavior, and recommended using the continuity plates in the flexural connections [4]. Kaufmann et al. focused on the brittle or ductile behavior of connections and expressed that the connections with electrodes of higher stiffness and continuity plates have more frangible behaviors [5]. Reoder applied finite element analysis and verified the improvement of stress distribution in the connection region with installing continuity plates [6]. Yee et al. recommended fillet weld instead of full penetration groove weld for preventing brittle fractures [7]. Engelhardt studied several beam to column connections with reduced beam sections and recommended the same thickness of beam flange for continuity plate [8]. Ricles expressed that the connection showed better seismic behavior in case of continuity plate installation; however, this installation could be ignored if the column flange had sufficient thickness [9]. Ghobadi et al. studied experimentally and analytically two single-sided full scale I-beam to box-column connections with the development of detail of T-stiffeners added to existing moment connections and resulted specimens with new proposed procedure performed well during test and also no crack propagation was seen [10]. Kiamanesh et al. investigated both experimentally and analytically the effect of stiffeners (column stiffeners, side-stiffeners, top and bottom flange stiffeners) and also effect of column flange thickness on the connection performance and energy dissipation. The specimens with both column and top-flange stiffeners had the highest values of energy dissipation. Decreasing the column thickness, in general, results in the decrease of connection stiffness and the increase of stress. Moreover, higher values of plastic

* Corresponding author at: K.N. Toosi University of Technology, Civil Engineering Department, No. 1346, Vali-Asr Street, P.O. Box. 15875-4416, 19697 Tehran, Iran.
E-mail address: fanaie@kntu.ac.ir (N. Fanaie).

http://dx.doi.org/10.1016/j.jcsr.2017.06.025
Received 6 May 2016; Received in revised form 19 April 2017; Accepted 24 June 2017
0143-974X/

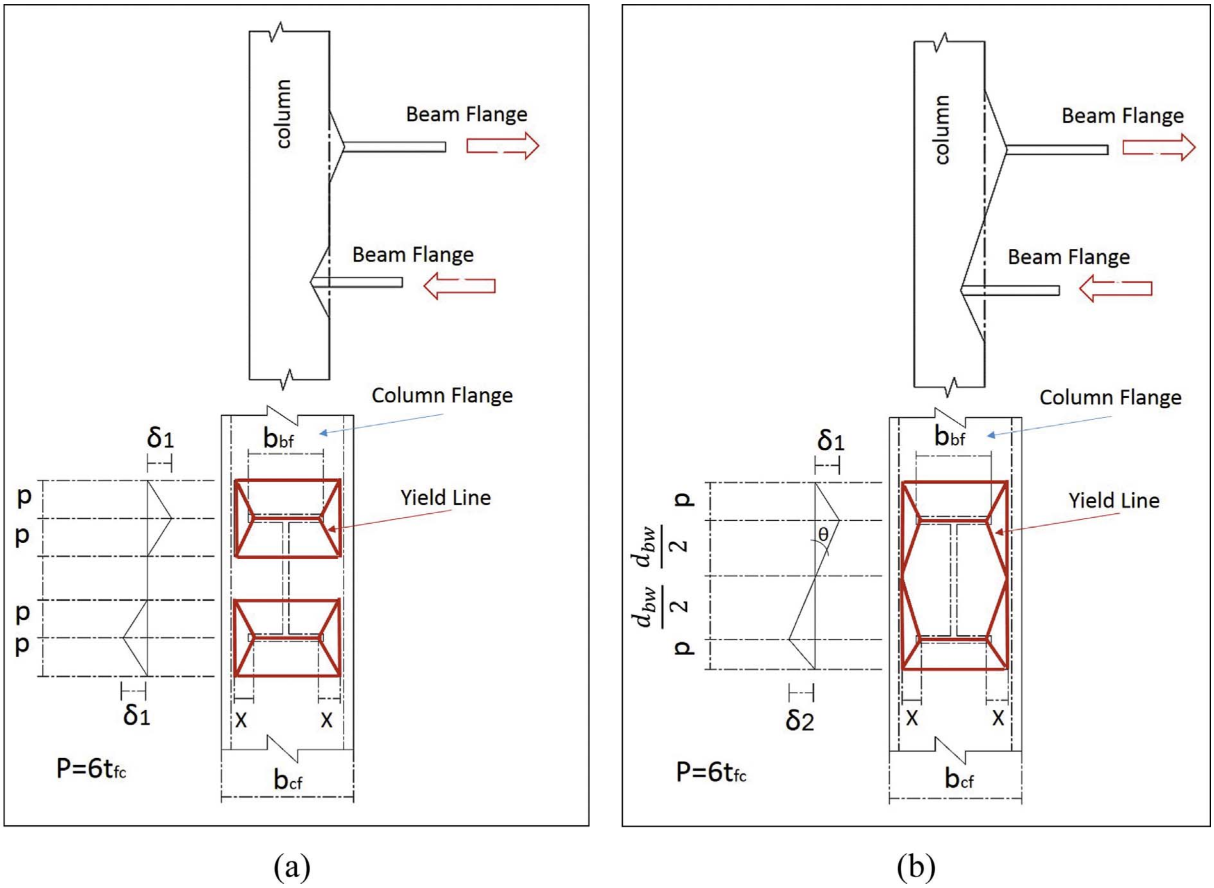

Fig. 1. Yielding mechanism: a) first mechanism; b) second mechanism.

Table 1
The specifications of the samples tested by Saneei Nia et al. [16].

Specimen	b_f (cm)	t_f (cm)	d_b (cm)	t_w (cm)	d_c (cm)	t_c (cm)	$\Sigma M_{pc}/\Sigma M_{pb}$
DC-S	16	1.5	33	0.8	30	1.5	1.03
DC-M	24	1.5	33	0.8	40	2.0	1.34
DC-L	24	2.0	38	0.8	50	2.5	1.74

Table 2
The thickness needed for column flange in case of not installing the continuity plates.

Column flange thickness	Sp-S	Sp-M	Sp-L
Needed thickness with respect to the first mechanism (mm)	30	36.7	42.3
Needed thickness with respect to the second mechanism (mm)	26.5	33.7	37.9
The used thickness (mm)	30	35	45

strain are observed in the side-stiffeners in case of removing column stiffener [11]. Mirghaderi et al. proposed a new moment connection for connecting I-beam to Box-column consisting of a vertical through plate instead of continuity plates. They studied two cyclically loaded specimens and suggested a design method to determine the dimension of through plate and also to evaluate the seismic performance of proposed connection. The specimens reached at least 0.06 rad of total story drift before experiencing strength degradation during the test [12]. Torabian et al. proposed an I-beam to Box-column moment connection without continuity plates consisting of vertical plates passed through the diagonal axes of a square box-column and welded to the box corners. Two cyclically loaded specimens were tested to evaluate the seismic performance of the connection. The obtained results showed that the specimens reached the 0.06 rad total story drift; and the proposed connection could be used as a prequalified connection in the special moment resisting frames [13]. The requirements of continuity plate installation in the panel zone were primarily presented by Graham et al. The studies with the purpose of stiffener requirements and rotation capacity were conducted on the two-way and four-way interior I-beam to I-column connections with typical sizes in the building frames. The results obtained for monotonically loaded specimens showed that stiffeners might be omitted in many beam to column connections. A formula was derived to control the stiffening requirements in I-beam to I-column connections by use of theoretical analysis, tests results and typical connections in building frames [14].

2. Analytical calculation of loading capacity of box column flange

Several clauses are presented in AISC-341-10 for controlling the requirements of installing continuity plates in the connections of I-shape beam to I-shape and boxed I-shape columns [15]. These relations are respectively results of analytical and laboratory investigations of Graham et al. and those of laboratory investigations of Ricles et al. [9,14]. Graham applied yield lines theory for the flange of I-shape column to calculate loading capacity. He considered several assumptions and calculations in his analysis with respect to the applicable connections. Following the attempts and calculations of Graham, in this research, two possible mechanisms, presented in Fig. 1a & b, are investigated based on the yield lines theory.

2.1. Analytical calculation through the first mechanism

Based on yield line theory the local bending resistance capacity of column flange (Q) for the first mechanism shown in Fig. 1a is calculable by equating the external work due to beam flange force on column flange calculated in Eq. (1) with internal work due to the yield lines