Journal of Constructional Steel Research 97 (2014) 24–38

Contents lists available at ScienceDirect

Journal of Constructional Steel Research

Response of partially-restrained bolted beam-to-column connections under cyclic loads

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ARTICLE INFO

Article history:
Received 2 July 2013
Accepted 18 January 2014
Available online 22 February 2014

Keywords:
Partially-restrained connection
Bolted beam-to-column connection
Finite element models
Components slippage
Parametric analysis
Rotational stiffness

ABSTRACT

The structural response of steel moment resisting frames (MRFs) is greatly dependent on the behavior of beam-to-column joints, according to a properly detailed beam-bolts-plates-column structural chain, in light of capacity design principles. A modeling procedure for bolted top-and-seat angle components and connections for potential use in seismic MRFs is presented herein. Although these partially-restrained (PR) connection systems have been demonstrated to provide economic savings, they are not currently certified to be used for moment resistance in any major building specification jurisdiction. Examples of full-scale moment resisting connection systems, experimentally tested in past programs, have been numerically analyzed, focusing on top-and-seat angle components, which were observed to control the global response of the joint in terms of failure mechanisms, limiting the displacement ductility capacity and dissipation energy capabilities of the whole resisting system. Refined nonlinear solid FE models, accounting for the influence of friction, pretension of bolts, prying and relative slippage of components through highly nonlinear contact elements, have been developed to reproduce the cyclic-reversal test protocol. Simplified approaches, based on one-dimensional inelastic force-based fiber elements, combined with nonlinear links, to globally represent connection elements interaction, have been developed and validated by comparisons with experimental response.

To propose an alternative and conservative method for quick rotational stiffness estimates of these PR bolted top-and-seat angle connections, a series of detailed parametric solid FE analyses have been performed and the effectiveness of this analytical preliminary-design-stage tool quantified in comparison with some of the most commonly known analytical approaches.

1. Introduction

Steel MRFs have traditionally been used as the lateral-force resisting system in high seismicity areas because of their significant potential for large ductility under seismic loading. In the past, fully welded moment connections have been assumed to provide the optimum combination of strength, stiffness and ductility in special MRFs; however, the 1994 Northridge and 1995 Hyogo-ken Nanbu earthquakes had revealed poor performance of these connection systems [1,2], since numerous brittle fractures were observed, due to large stress/strain concentrations in the critical joint region [3]. Indeed, traditional fully welded moment connections had exhibited several drawbacks, mainly related to the connection geometry, which causes large strain demands in critical zones, and to the design approach of modern MRF structures, which implies, according to economic reasons and performance-based design concepts [4], a concentration of the lateral resistance in a limited number of connections [5]. Sensitivity to fracture of traditional welded connection details resulted in low rotational ductility under cyclic load reversals, while the adoption of deeper beams to reduce drift demand gave rise to large shear demands at the connection level: in this case, to ensure the desired weak beam–strong column mechanism, the strain concentration problems inherent to the connection geometry worsened. Therefore, increasingly wide interest in the performance of bolted connections under seismic loading has occurred, as these solutions, characterized by promising performance in past seismic events [6], represent a particularly attractive choice, applicable both to new construction and to retrofitting of existing structures. In fact, bolted connections have the potential to be able to mitigate some of the major issues related to welded systems, potentially providing the required stiffness, strength and rotational capacity demanded by MRFs. With proper detailing, in many cases a high level of redundancy and a level of stiffness comparable to that of fully welded connections can be achieved. In addition, steel MRFs with more flexible beam-to-column connections ensure many economical and construction advantages over rigid frames [7]. Past studies [8,9] also demonstrated that the adoption of PR bolted beam-to-column connections does not necessarily imply a larger drift

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0143-974X/$ – see front matter © 2014 Elsevier Ltd. All rights reserved.
http://dx.doi.org/10.1016/j.jcsr.2014.01.014

response than that experienced in MRFs with rigid joints. Therefore, the belief that excessive deformations will take place or that instability under gravity loads and P-delta effect might occur during strong earthquakes was refuted. In particular, in many cases, low-rise buildings could reveal an improved structural response, according to the beneficial contribution of component slippage, thus showing increased energy dissipation capabilities, without any added plastic deformation exploitation of the components, directly implying lower force demands at the connection level.

In the light of the aforementioned observations, this paper investigates the response of bolted clip angle connections for potential use in low and medium seismicity areas, where limited ductility in MRF is expected and, consequently, stiffness rather than strength of such connections is crucial, particularly in the satisfaction of the building performance objectives for the immediate occupancy performance level or the serviceability limit state. Hence, detailed FE models have been developed to properly capture the experimental behavior of two full-scale PR bolted top-and-seat angle connections, tested in past programs. Based on this validation, the connection response has been investigated, by varying friction coefficients, pretension of bolts, and bolt-plates gap to assess their influence both on global load-displacement curve and local stress/strain distributions, observed in the critical joint region; finally, 16 different PR bolted beam-to-column connections with flange angle cleats have been designed, according to European standards [10] and their rotational stiffness has been compared to those predicted by conventional methods [10,11], in order to propose a closed-form expression, depending on components and bolts geometry only.

2. Literature review

Through the years, bolted connections have been widely investigated to assess their performance under cyclic-reversal pseudo-static loading [12–14]. Even though they may experience a slight loss of elastic stiffness due to slippage, this was gradual and stable and almost the whole initial elastic stiffness was recovered at the end of the slip plateau. Also, larger rotational ductility, achieved through slippage of faying surfaces, prevented the severe local buckling induced by welded connections; semi-rigid bolted connections were observed to be robustly ductile and slippage was evidenced to improve the response of steel MRF subjected to earthquakes. Piluso and Rizzano [14] carried out an experimental program devoted to assess the cyclic force-displacement response of 28 bolted T-stubs, subjected to both constant and variable amplitude cyclic loading histories. Stiffness and strength degradation rules were derived as a function of the displacement amplitude required at any cycle and of the energy dissipated in the previous loading history, in order to propose a semi-analytical model for predicting the cyclic behavior of bolted T-stubs starting from the knowledge of their geometrical and mechanical properties. Maggi et al. [13] experimentally studied several full-scale specimens of bolted extended end plate connections, observing that the T-stub analogy presents limitations for the accurate prediction of the yield lines at the end plate, when combined to bolt tension failures, due to the component interaction; criticalities in accounting for prying action also emerged, since the center of rotation for the plate was not placed at the compression flange level, as conventionally accepted in the design process, but at the level of the first bolt in compression. Furthermore, Girão Coelho et al. [12] statically tested 8 extended end plate moment connections, designed to trigger failure in the end plate and/or bolts without exploitation of the full plastic moment capacity of the beam; the influence of the end plate thickness and steel grade on the observed monotonic response was highlighted. In addition, Eq. 6.27 of Eurocode 3 (EC3) [10] was proven to largely overestimate the initial rotational stiffness experimentally observed, as roughly doubled predictions were obtained. This quite large and unsafe mismatch was motivated by the “unbalanced” connection systems tested, characterized by some of their components designed to be much weaker than the remaining; however, the need of further investigations, by assessing joint configurations, designed in accordance with common design practice [10], seems to be at least justified.

Although PR bolted beam-to-column connections usually present complex component interaction, characterized by mechanisms such as slip, bearing, and ovalization, the feasibility of the FE models to accurately simulate connection response has been verified during the last two decades by a number of research efforts [15–23], mainly addressed to capture behavioral changes as a consequence of geometric variations. The majority of these studies was focused on end plate connections and even though simplification in components geometry, bolts, contact conditions and friction effects have been employed in many early studies [15–17], accurate prediction of experimental results is shown. In particular, Bose et al. [15] characterized the moment carrying behavior of end plate joints, identifying their potential failure modes. More recent researches explicitly recognize bolt head-angle contact, while neglecting bolt shank–hole interaction [18] or apply a general nonlinear contact scheme to represent contact conditions between each component of the connection system, anyway analyzed in a monotonic fashion only [19,20]. Over the last years, significant improvements have been achieved in simulating the interface between the end plate and the column flange, as well as the pretension force in the bolts [21]; Gerami et al. [22] investigates the cyclic behavior of bolted end plate joints, while Girão Coelho [23] specifically applies ductile fracture models to predict the rotation capacity of bolted end plate joints subjected to large scale plasticity.

3. Nonlinear FE analyses

In light of this scenario, advanced nonlinear FE analyses, based on the use of refined 3D solid models with highly nonlinear contact algorithms, accounting for friction and relative slippage of components, appear to be an attractive tool in providing stiffness and strength estimates for a large variety of connection geometries, after reliable validation with experimental tests. In addition, these detailed FE studies aim at capturing the evolution of local quantities, such as principal stresses/strains, crucial in interpreting damage patterns and failure modes of the bolted connection under investigation, as well as at developing simplified connection modeling approaches, generally based on inelastic spring elements and addressed to represent connection element interaction in an equivalent and simplified manner, that can be easily incorporated into modern structural analysis programs. Hence, an exhaustive past research program [3], carried out at the Georgia Institute of Technology was selected in order to obtain a suitable experimental database of destructive tests.

3.1. Cases of study definition from past experimental database

As a part of the SAC Program [3], a series of 10 full-scale beam–column connection tests under cyclic loads was performed and collected in [24]; their calibration was achieved in light of an initial series of 48 T-stub and 10 clip angle components tested in cyclic tension and compression. In particular, this paper is focused on two full-scale top-and-seat angle experimental tests conducted by Schrauben [24], respectively named FS-01(CA-02) and FS-02(CA-04), to quantify the sensitivity of the response, in terms of failure mechanism, flexural and shear strengths, rotational stiffness, displacement ductility capacity and energy dissipation capabilities with respect to the connection system configuration. Specimen FS-01(CA-02) consisted of a W18 × 40 (W460 × 60) beam bolted to a W14 × 145 (W360 × 216) column by a 9″ × 3-1/8″ × 5/16″ (229 × 79 × 8 mm) shear tab and clip angles cut from a L8 × 6 × 1 (L203 × 152 × 25). Two 7/8″ (22 mm) diameter, 3-1/4″ (83 mm) long A490 high-strength bolts with one washer were used to fasten one leg of each clip angle to the flange of the column. The gage distance of these “tension” bolts was 2-1/2″ (63 mm). Four 7/8″ (22 mm) diameter, 3″ (76 mm) long A490 high-strength bolts with two washers were used to fasten the other leg to the beam flange