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Full strength extended stiffened end-plate joints: AISC vs recent European design criteria

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ABSTRACT

Extended stiffened end-plate bolted joints are widely used in seismic resistant steel frames. In the United States of America (USA) this type of joint is seismically pre-qualified according to AISC 358-16. At the present time, prequalification criteria for different types of bolted joints are also under development in Europe within the framework of the EQUALJOINTS (i.e. European pre-QUALified steel JOINTS) research project. The design criteria and detailing rules proposed by this European project for extended stiffened end-plate joints differ from AISC criteria in some respects. Therefore, the aim of this work is to verify and to compare the effectiveness of both design procedures through the results of a comprehensive parametric study based on finite element (FE) simulations. The FE results show that both AISC and European design procedures can guarantee the formation of a plastic hinge in the beam under cyclic loading. However, under column loss scenarios the European connections are more ductile than those designed according to both AISC 358-16 and AISC 341-16. In addition, it is investigated the possibility to use heavy columns satisfying the resistance requirements without stiffeners (i.e. continuity plates and supplementary web plates). The comparison between the response of the joints with and without stiffened columns shows that heavy unstiffened columns can be adopted without appreciably modifying the joint response.

1. Introduction

Extended stiffened end-plate bolted (ESEPB) joints are widely used in seismic resistant moment resisting frames, mainly due to the simplicity and the economy of their fabrication and erection.

In the last thirty years, a large number of studies were carried out to investigate the seismic performance of ESEPB joints, especially in United States of America (USA). In 1990 Murray [1] presented design procedures for the eight-bolt extended stiffened end-plate moment connections. Afterwards, Sumner and Murray [2,3] carried out an extensive theoretical and experimental campaign to qualify seismically ESEPB joints. Their tests demonstrated that this type of bolted joints can guarantee satisfactory energy dissipation capacity, without appreciable degradation of strength and stiffness. Provided that the beam-to-column hierarchy would be satisfied, the design philosophy was envisioned to provide adequate strength in both the connection and the column web panel to allow the formation of a plastic hinge near the beam extremity. In 2004 Murray and Sumner [4] presented a unified method for the design of ESEPB joints subjected to both wind and seismic loading that uses yield line theory to predict the end-plate and column flange strength. Their findings represent the background document of the current AISC 358-16 [5] for ESEPB joints.

Outside from the USA, Shi et al. [6,7] carried out both monotonic and cyclic tests in order to investigate the seismic behavior of ESEPB joints and to validate an analytical method to predict the joint response. In particular, Shi et al. [6] highlighted the key role of rib stiffeners, concluding that extended end-plate connections with end-plate stiffeners can provide better rotation capacity and larger stiffness than unstiffened joints, provided that rational design criteria are used. Analytical models and design rules for the rib stiffener were developed by Lee et al. [8,9] for welded rib-stiffened joints. More recently, Abidelah et al. [10] carried out an experimental and numerical study in order to investigate the influence of rib stiffeners on the cyclic behavior of bolted end-plate joints. In particular, they demonstrated that the ribs modify the position of the center of compression increasing the resisting lever arm of the bolted connection.

In Europe (EU), the current versions of the Eurocodes (i.e. EN1993:1-8 [11] and EN1998-1 [12]) provide neither specific requirements nor codified prequalification procedures for seismic resistant extended stiffened end-plate joints. However, in 2016

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Nomenclature

List of symbols

C	maximum distance between the beam flanges
C_{pr}	overstrength factor accounting for the strain hardening according to AISC codes. It is equivalent to γ_{sh} assumed by European design rules
F_{nt}	nominal tensile strength of bolt according to AISC codes
F_u	tensile strength of the yielding element according to AISC codes
F_y	yield stress of the yielding element according to AISC codes
F_{yc}	yield stress of column flange material according to AISC codes
F_{yp}	yield stress of end-plate material according to AISC codes
I	second moment of area
K and K'	out-of-square of the flange tips of the hot-rolled profiles
L_h	distance between the plastic hinges
$M_{con,Rd}$	design flexural strength of the connection
M_f	probable maximum moment at the column face
M_{pr}	probable maximum moment at plastic hinge
M_{wp}	probable maximum moment at the column axis
M^*	moment at the column axis under column removal
N	catenary action
N_{pr}	probable axial strength of the beam
$R_{t,Rd}$	tensile strength of the bolt
$R_{p,Rd}$	tensile strength of the equivalent T-Stub
R_u	required shear strength of the column web panel
S_h	distance from face of column to the plastic hinge
S^*_h	distance from center of column to the plastic hinge
V_{col}	shear force into the column
$V_{gravity}$	shear force due to the gravity loads
V_u	required shear strength of the beam and beam web-to-column connection
W_{pl}	plastic modulus
Y_c	column flange yield line mechanism parameter
Y_p	end-plate yield line mechanism parameter
Z	plastic section modulus
b_{bf}	width of the beam flange
b_p	width of the end-plate
d	depth of the beam
d^*	distance from the centroid of the beam flange in tension and the centroid of the compression center
d_e	vertical distance from the external bolt row to the end-plate edge
$d_{b,required}$	required minimum diameter of bolts
h_i	distance from the centerline of compression flange to the centerline of the *i*-th tension bolt row
h_o	distance from the centerline of compression flange to the tension-side outer bolt row in four-bolt extended end-plate moment connections
h_p	depth of the end-plate
p_b	vertical distance between the inner and outer row of bolts in eight-bolt stiffened extended end-plate moment connections
p_{fo}	vertical distance from the outside of the beam tension flange to the nearest outside bolt row
t_{bf}	thickness of the beam flange
t_{cf}	thickness of the column flange
t_p	thickness of the end-plate
$t_{p,requried}$	required thickness of the end-plate
t_s	thickness of the rib
t_{SC}	thickness of the continuity plate
t_{SWP}	thickness of the supplementary web panel
δ_o	maximum allowed out-of-square value of the beam flange tips
γ_{sh}	overstrength factor accounting for the strain hardening according to European design rules. It is equivalent to C_{pr} assumed by AISC design rules
γ_{ov}	overstrength factor accounting for the variability of yield stress (i.e. the ratio between the average and the characteristic yield stress) according to EN1998-1
ϕ_d	resistance factor for ductile limit states according to AISC codes
ϕ_n	resistance factor for non-ductile limit states according to AISC codes

prequalification procedure and design criteria for seismic resistant ESEPB joints have been developed by D'Aniello et al. [13] within the framework of the EQUALJOINTS (i.e. European pre-QUALified steel JOINTS) research project [14] (hereinafter referred as "EJ" for brevity sake).

It should be noted that in Europe there is currently an action program promoted and coordinated by CEN Technical Committee 250 (CEN/TC250) to amend all current Eurocodes. In this framework, several working groups and committees, e.g. Technical Committee 13 – seismic design (TC13) and Technical Committee 10 – steel connection (TC10) of the European Convention for Constructional Steelwork (ECCS), are working to revise, update and harmonize the design rules for seismic resistant joints addressed by EN 1998-1 and EN1993:1-8.

The aim of this study is to compare the design rules for full strength ESEPB joints recently developed in Europe [13,14] with those recommended for fully restrained flexural ESEPB joints by AISC 358-16 [5] in order to develop a further background document to support the revision of the relevant parts of current Eurocodes. The paper is organized into three parts. In the first part the main differences between AISC and EJ design rules are described and discussed. In the second part the effectiveness of both approaches on joint response is investigated by means of parametric finite element analyses. In third part the results are discussed and concluding remarks are inferred.

2. Design criteria for extended stiffened end-plate bolted joints

Both AISC 358-16 [5] and EQUALJOINTS [13,14] design procedures for full strength joints aim at ensuring the formation of a plastic hinge in the beam. This purpose is differently achieved, and the main differences concern the configuration of the connection (i.e. distribution of bolts and requirements on rib stiffeners), the calculation assumptions (i.e. capacity design rules, position of center of compression, active bolt rows, yield line pattern), and some ductility criteria (i.e. limitations on the thickness of end-plate compared to the diameter of bolts). These aspects are discussed in this section. In addition, for the sake of comparison, the symbols used by AISC codes [5,15] are adopted in the design equations described hereinafter, except for some terms introduced within EJ and not explicitly defined in the relevant sections of AISC 358-16 [5].

AISC 358-16 [5] procedure imposes limits on the allowed size for both beam and column, while EJ procedure does not impose any limitation provided that the beam-to-column hierarchy (i.e. weak beam-strong column) is satisfied. According to AISC 358-16 [5], either 4-bolt row or 8-bolt row joint configurations can be adopted (see Fig. 1), but the selection should be based on geometrical limitations, e.g. the distance of bolts rows, the thickness of end-plate, the size of the connected beam, etc. On the other hand, only one joint configuration with 6 bolt rows (see Fig. 1) is considered by the EJ procedure [13,14]. Both