

Cyclic Behavior of High-Strength Steel Unstiffened Extended End-Plate Moment Connections

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Abstract

The paper presents experimental investigations on the cyclic behavior of unstiffened extended end-plate moment connections fabricated from high-strength steel. Five tests were conducted on connections using conventional (grade Q355, $f_y = 355$ MPa) steel beams and high-strength (grade Q690, $f_y = 690$ MPa) steel columns and end plates. The effects of end plate and column flange thicknesses, and weak panel zone design, were examined, thus resulting in failure modes being either end-plate fracture, bolt rupture, or combined mode. Experimental results are provided in detail, including observations of different failure modes, moment-story drift angle, moment-connection rotation, moment-shear distortion, and moment-beam rotation curves, as well as mechanical indices to characterize cyclic response. The moment resistances of these specimens are compared with Eurocode 3 Part 1-8, indicating consistent predictions. The maximum and cumulative plastic rotations, and cumulative energy dissipation, are evaluated, demonstrating limited plastic rotations (15–25 mrad) of the bolted end-plate but substantial plastic distortion (> 35 mrad) of the panel zone. Hence, the 30% limit on web panel contribution to joint plastic rotation is suggested to be relaxed to 70%. Inelastic response of the panel zone can be well predicted by the Krawinkler model. Thinner column flanges than the end plate of the same steel grade has an adverse affect and should not be used in seismic design.

Keywords: End plate connection, High-strength steel, Moment connection, Cyclic, Joint

1 Introduction

High-strength structural steels, defined as those with nominal yield strengths ≥ 460 MPa (e.g., Q460 in China or S460 in Europe) [1–5], enable lightweight structural systems that offer both economic advantages through material efficiency and environmental benefits via reduced embodied carbon. Their architectural versatility further enhances their appeal for modern construction [6]. To maximize these benefits while mitigating serviceability concerns, high-strength steels are particularly suitable for braced frame systems where deflection requirements are non-governing. In such applications, semi-continuous connections - especially bolted end-plate configurations - provide optimal solutions by combining fabrication efficiency with cost-effectiveness [7]. Their widespread adoption in steel semirigid frames built in seismic areas underscores the critical need for research into the behavior of high-strength steel end-plate connections to facilitate code-compliant engineering implementations. Because high-strength steels are characterized with high yield-to-tensile ratio and low ductility [8, 9], current seismic design criteria derived from prior research on conventional structural steels are challenged. To ensure seismic safety and realize optimized design, the knowledge on the plastic rotation supply and energy dissipation capacity of high-strength steel end-plate connections is important for semirigid frames that are expected to experience inelastic deformation demand under strong earthquakes [10].

When properly designed, extended end-plate connections can achieve substantial initial rotational stiffness and resistance. Their energy dissipation capacity may be further enhanced through balanced design between bolts and end plates [11, 12], thereby avoiding the premature weld fracture issues commonly observed in traditional welded moment connections under seismic loads [13, 14]. As for high-strength steels, the inherent low material ductility does not necessarily translate to unsatisfactory structural ductility. High-strength steels still exhibit stable

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cyclic responses at the material level [15–17]. Besides, the ductility of structural members and connections is also governed by geometry discontinuities and system-level behavior [18, 19]. Girão Coelho and Bijlaard [20, 21] demonstrated that bolted end-plate joints with S690 or S960 steel end plates could attain monotonic plastic rotations exceeding 35 mrad through optimized bolt-to-end-plate configurations. Dubina et al. [22] revealed that stiffened extended end-plate connections designed for Failure Mode 2 (per Eurocode 3 Part 1-8 [23]) using S460 or S690 steel end plates achieved cyclic plastic rotations of 25–38 mrad, meeting the ductility requirements for DCM (Ductility Class Medium) and even DCH (Ductility Class High) in Eurocode 8 Part 1 [24]. Notably, they also found that S460 steel column web panels contributed over 60% of the total joint rotation via shear distortions exceeding 25 mrad, suggesting that the 30% limit in Eurocode 8 Part 1 for panel zone deformation contributions may be overly conservative. Sun et al. [25] tested four Q690 steel flush and extended end-plate connections under cyclic loading, reporting ultimate joint rotations (at 85% peak moment) surpassing 35 mrad. Liang et al. [26] evaluated three Q690 stiffened extended end-plate connections with reduced beam sections (RBS) under cyclic loading. While tests were terminated due to beam torsional buckling or lateral restraint limits, the results could not fully characterize the connections' seismic performance. Recent European prequalification programs have substantially advanced the seismic design and qualification of conventional-steel end-plate beam-to-column joints: the EQUALJOINTS and EQUALJOINTS-Plus projects established a harmonized prequalification procedure for EC8-compliant joints [27], which has been applied to external extended stiffened end-plate joints [28] and to EC8-1-2-compliant unstiffened end-plate joints [29], while shake-table tests have further validated moment-resisting frames with detachable yielding zones [30]. These studies, however, concern conventional steels, and their design provisions have not been verified for high-strength steel end-plate connections.

Existing experimental studies under monotonic loading [20, 21, 31, 32] have demonstrated that the component method specified in Eurocode 3 Part 1-8 remains applicable for high-strength

steel end-plate connections. The authors' previous work [32] further revealed that the code provisions for joint rotation capacity tend to be conservative. However, despite the above prequalification studies on conventional steels, current research on the seismic performance of high-strength steel end-plate connections [22, 25] remains limited, particularly for unstiffened extended end-plate connections, where insufficient experimental data exist to establish the quantitative relationships between key design parameters (e.g., end-plate thickness, bolt configuration) and seismic performance indicators such as plastic rotation capacity and energy dissipation capability. Notably, the column web panel - even when fabricated from high-strength steel - exhibits significant plastic shear deformation capacity [33, 34]. Previous studies have not comprehensively investigated its interaction effects with bolted end-plate connections under cyclic loading conditions. To address these research gaps, this study conducts cyclic loading tests to systematically examine how the compatibility between Q690 high-strength steel end-plates, panel zones, column flanges and bolts influences the seismic performance of connections. The experimental results provide a basis for developing reliable seismic design recommendations.

2 Test program

2.1 Design of specimens

A reference one-sided end-plate connection was designed to represent typical joints between Q355 conventional-strength steel beams and Q690 high-strength steel columns in a prototype 3-bay, 6-story steel moment frame [33]. This material combination reflects practical Chinese construction practices where beam design is typically governed by deflection requirements under gravity loads, limiting the benefits of using high-strength steel in beams. Conversely, column design must satisfy strength/stability requirements under combined gravity and seismic loading as well as the strong-column-weak-beam (SCWB) mechanism, making high-strength steel advantageous for material efficiency. The prototype frame features 6 m beam spans and 3 m story heights, with welded H-sections for both beams ($h_b \times b_b \times t_{wb} \times t_{fb} = 320 \times 160 \times 8 \times 14$ mm) and

columns ($h_c \times b_c \times t_{wc} \times t_{fc} = 200 \times 160 \times 12 \times 16$ mm). These sections were designed according to Chinese Standard [35] and verified as Class 1 cross-sections per Eurocode 3 Part 1-1 [36]. The SCWB mechanism and panel zone strength requirements were carefully checked by,

$$\frac{\Sigma W_{pc} f_{yc} (1 - P/P_y)}{1.1 \eta_y W_{pb} f_{yb}} \geq 1 \quad (1)$$

$$\frac{\alpha_{pz} W_{pb} f_{yb}}{\frac{4}{3} \frac{f_{yc}}{\sqrt{3}} h_{pz} b_{pz} t_{pz}} \leq 1 \quad (2)$$

where W_{pc} and W_{pb} are plastic section moduli of the column and beam, respectively; f_{yc} and f_{yb} are material yield strength of the column and beam, respectively, and $\eta_y = 1.1$ accounting for material overstrength; P is the axial column load, taken as 30% of the yield axial load, $P_y = f_{yc} A_c$ (A_c is the column cross-section area); $h_{pz} = h_b - t_{fb}$, $b_{pz} = h_c - t_{fc}$, and $t_{pz} = t_{wc}$ (no doubler plates) are the height, width, and thickness of the panel zone, respectively; α_{pz} is a factor equal to 0.95 for one-sided connections tested in this study. The SCWB ratio and panel zone demand-to-capacity ratio for the aforementioned combination of beam and column cross sections are 1.6 and 0.8, respectively, allowing for controlled yielding in the panel zone.

The bolted end-plate connection was designed following Chinese Technical Specification [37] under design gravity load combinations. In this specification, the tensile bolt region of the end plate is idealized as an equivalent T-stub, and the design proceeds as follows. The tension and shear demands per bolt are, respectively,

$$N_t = \frac{M}{n_2 h_1} + \frac{N}{n} \leq N_t^b \quad (3)$$

$$N_v = \frac{V}{n_v} \leq N_v^b \quad (4)$$

where M is the connection moment, N is the connection axial tension ($N = 0$ under compression), V is the connection shear, n is the total number of bolts, n_2 is the total number of bolts in the two

rows symmetric about the tension flange, n_v is the number of bolts resisting shear ($n_v = n - n_2$), h_1 is the distance between the beam flange centerlines, N_t^b is the tension capacity of a high-strength bolt ($N_t^b = 0.8P$, where P is the standard pretension), and N_v^b is the slip-critical shear capacity of a high-strength bolt ($N_v^b = 0.9kn_f\mu P$, where $k = 1$ for a standard hole, $n_f = 1$ for a single shear plane, and the slip factor $\mu = 0.3$). The required end-plate thickness without prying action is

$$t_{ec} = \sqrt{\frac{4e_2N_t^b}{bf}} \quad (5)$$

where e_2 is the distance from the bolt center to the web edge of the T-stub, b is the effective end-plate width per bolt row, and f is the design strength of the end-plate steel. When prying action is considered, the required end-plate thickness and the associated coefficients are

$$t_e \geq \sqrt{\frac{4e_2N_t}{\psi bf}} \quad (6)$$

with the prying influence factor $\psi = 1 + \delta\alpha'$, the section coefficient $\delta = 1 - d_0/b$, $\rho = e_2/e_1$, $\beta = (1/\rho)(N_t^b/N_t - 1)$, and $\alpha' = 1.0$ for $\beta \geq 1.0$ or $\alpha' = \min\{(1/\delta)[\beta/(1 - \beta)], 1.0\}$ for $\beta < 1.0$, where d_0 is the bolt hole diameter, and e_1 is the distance from the bolt center to the free edge of the T-stub. The prying force and its coefficient are

$$Q = N_t^b \left[\delta\alpha\rho \left(\frac{t_e}{t_{ec}} \right)^2 \right] \quad (7)$$

with $\alpha = (1/\delta)[(N_t/N_t^b)(t_{ec}/t_e)^2 - 1] \geq 0$, and the bolt tension check is

$$N_t + Q \leq 1.25N_t^b \quad (8)$$

Following the prototype-frame internal forces ($M = 97.2$ kN·m, $N = 0$, $V = 194.4$ kN), a benchmark specimen satisfying all the Chinese code capacity requirements was designed,

adopting ten Class 10.9 M20 high-strength bolts ($P = 170$ kN, $d_0 = 22$ mm) and a 16 mm-thick Q690 end plate ($t_e = 16$ mm, $f = 630$ MPa), with demand-to-capacity ratios of 0.64 (end plate), 0.51 (bolt in tension), 0.71 (bolt in shear) and 0.80 (panel zone). This configuration, as shown in Figure 1, is designated as B355-C690-EP16 (denoting beam/column grades and end-plate thickness).

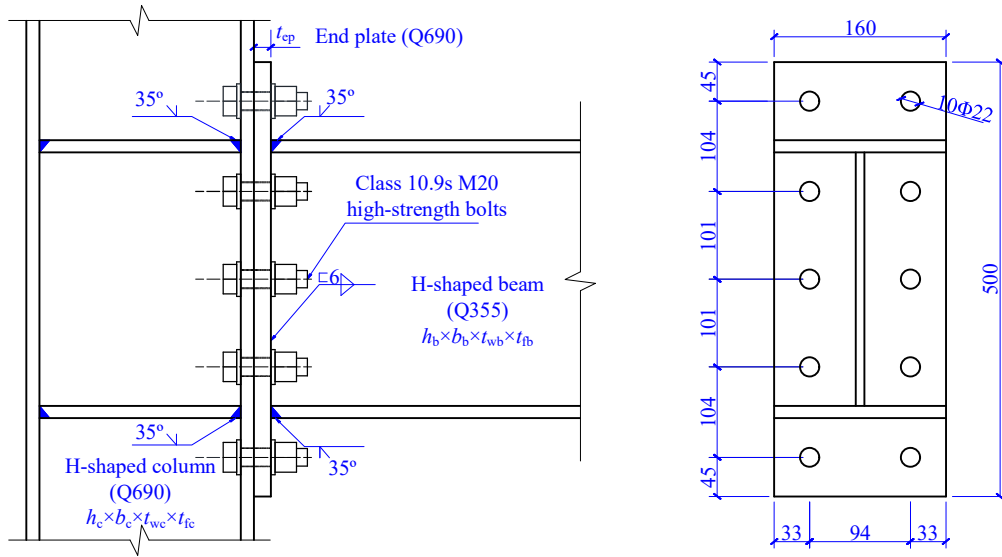


Figure 1. Extended end-plate connection details (unit: mm)

To investigate parameter effects, four variant specimens were developed: i) B355-C690-EP12 and B355-C690-EP8, featuring reduced end-plate thicknesses (12 mm and 8 mm, respectively); ii) B355-C690-EP16-WPZ (denoting Weak Panel Zone) with reduced column web thickness (8 mm) while maintaining SCWB, whose design ratios according to Eqs. (1) and (2) are 1.5 and 1.21, respectively, thus allowing substantial yielding in its panel zone; and iii) B355-C690-EP16-WCF (denoting Weak Column Flange) with thinner column flanges (12 mm), maintaining SCWB (the ratio of Eq. (1) is 1.3) and panel zone relative strength (the ratio of Eq. (2) is 0.79) but intentionally violating the column flange thickness requirement in Chinese Technical Specification [37] (which prescribes the detailing rule that the column flange thickness should not be less than the end-plate thickness) to explore economic optimization potential. Table 1 summarizes the test matrix.

Table 1. Test specimens

Specimen label	Beam section (mm)	Column section (mm)	End plate thickness (mm)	Bolt
B355-C690-EP16	H320×160 ×8×14	H200×160×12×16	16	10.9s M20
B355-C690-EP12		H200×160×12×16	12	
B355-C690-EP8		H200×160×12×16	8	
B355-C690-EP16-WPZ		H200×160×8×16	16	
B355-C690-EP16-WCF		H200×160×12×12	16	

It should be stressed that, the specimens are deliberately proportioned with strong beams and columns so that the weakest components are the joint components themselves (end plate, bolts, panel zone and column flange) rather than the beam end, which conflicts with the “strong connection, weak member” principle of the current Chinese Standard [35] and is intentionally adopted to explore the joint yielding and energy dissipation that the current Chinese code framework excludes. This is also reflected in Table 2, where for each specimen, the demand-to-capacity ratios per the Chinese code [35, 37], as well as the governing plastic mechanism (end plate in bending “epb”, column flange in bending “cfb”, or column web panel in shear “wp”, with the numbers in parentheses denoting bolt row locations (individual or group) and the equivalent T-stub failure mode in square brackets) and the joint plastic resistance M_j (kN·m) computed with nominal material properties and without partial safety factors per the Eurocode 3 Part 1-8 component method [23], are summarized.

The welding details in Chinese Technical Specification [37] were followed, including complete-joint-penetration (CJP) groove welds between beam flanges and end plates, and supplementary fillet welds (6 mm leg size) at weld roots and beam web, both using gas metal arc welding (GMAW) with Q355-matched electrodes of type E501T-1, whose specified minimum yield and tensile strength are 375 and 490 MPa, respectively. Continuity plates matching beam flange thickness were used, connected via CJP and fillet welds to the column flanges and web,

Table 2. Design evaluation per the Chinese code and Eurocode 3 Part 1-8

Specimen label	Chinese code: D/C ratios ¹				Eurocode 3 Part 1-8: mechanism ³	
	End plate ²	Bolt in tension	Bolt in shear	Panel zone	Yielding sequence	M_j (kN·m)
B355-C690-EP16	0.64	0.51	0.71	0.80	epb(1)[Mode 2] → epb/cfb(2)[Mode 3] → wp	247
B355-C690-EP12	1.14	0.70	0.71	0.80	epb(1)[Mode 1] → epb(2)[Mode 2] → wp	220
B355-C690-EP8	2.57	0.83	0.71	0.80	epb(1)[Mode 1] → epb(2+3+4)[Mode 1]	145
B355-C690-EP16-WPZ	0.64	0.51	0.71	1.21	epb(1)[Mode 2] → wp	188
B355-C690-EP16-WCF	0.64	0.51	0.71	0.79	epb(1)[Mode 2] → cfb(2)[Mode 2] → wp	240

¹ Per the Chinese code, the demands of the end plate, the bolts in tension and the bolts in shear (slip-critical) are computed from the prototype-frame internal forces ($M = 97.2$ kN·m, $N = 0$, $V = 194.4$ kN); D/C denotes the demand-to-capacity ratio.

² Per the Chinese code, the end-plate, bolt-in-tension, bolt-in-shear and panel-zone demand-to-capacity ratios follow Eqs. (6), (8), (4) and (2), respectively; the end-plate demand-to-capacity ratio is expressed as $(t_{e,req}/t_e)^2$ with $t_{e,req} = 12.8$ mm.

³ Per Eurocode 3 Part 1-8, the evaluation uses nominal material properties without partial safety factors; the “mechanism” column reports the governing plastic yielding sequence and the joint plastic resistance.

respectively.

2.2 Material properties

Table 3 summarizes the mechanical properties of the Q355 and Q690 steel plates used in this experimental investigation. The comprehensive material characterization includes: modulus of elasticity (E), yield strength (f_y) or 0.2% offset strength where applicable, strain at yield plateau termination/strain hardening initiation (ϵ_{st}), ultimate tensile strength (f_u), strain at ultimate strength (ϵ_u), yield-to-tensile strength ratio (f_y/f_u), percentage elongation at fracture (δ). Notably, two distinct batches of 12 mm-thick Q690 steel plates were utilized in specimen fabrication: i) Batch I, used for Specimens B355-C690-EP16 and B355-C690-EP12, and ii) Batch II, employed for Specimens B355-C690-EP8 and B355-C690-EP16-WCF. The complete engineering stress-strain relationships for these Q690 steel coupons have been previously reported by the authors [33, 34].

Additionally, tensile tests were performed on the Class 10.9 M20 high-strength bolts, with

key mechanical properties including elastic modulus (E), ultimate tensile strength (f_u), and strain at ultimate strength (ϵ_u). These bolt properties are similarly presented in Table 3 for complete material documentation.

Table 3. Material properties

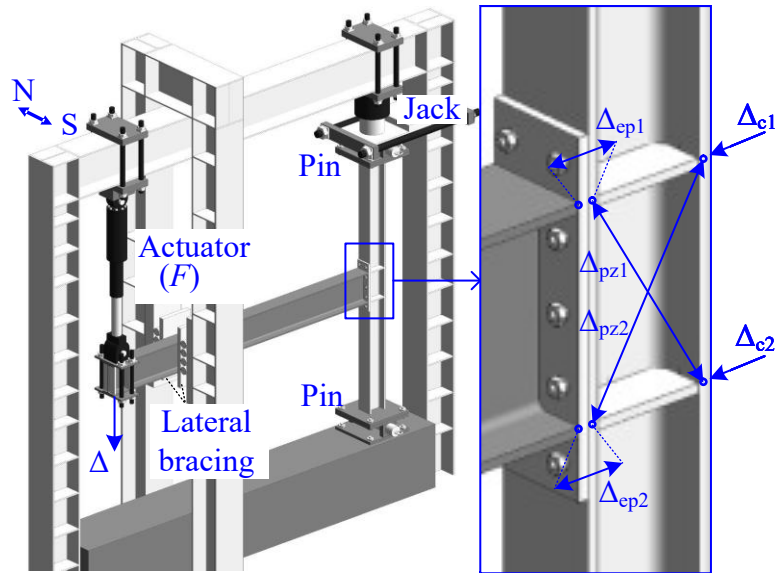
Steel grade	Plate thickness (mm)	E (GPa)	f_y (MPa)	ϵ_{st}	f_u (MPa)	ϵ_u	f_y/f_u	δ
Q355	8	205.0	406	0.014	539	0.153	0.75	25%
	14	203.5	368	0.020	522	0.200	0.70	29%
Q690	8	208.3	723	—	822	0.100	0.88	20%
	10	189.4	794	—	902	0.106	0.88	21%
	12(I)	205.8	775	0.018	816	0.060	0.95	16%
	12(II)	203.7	660	—	749	0.079	0.88	17%
	16	219.8	811	0.022	840	0.055	0.97	17%
10.9s	M20	206.0	—	—	1135	0.110	—	—

2.3 Test setup

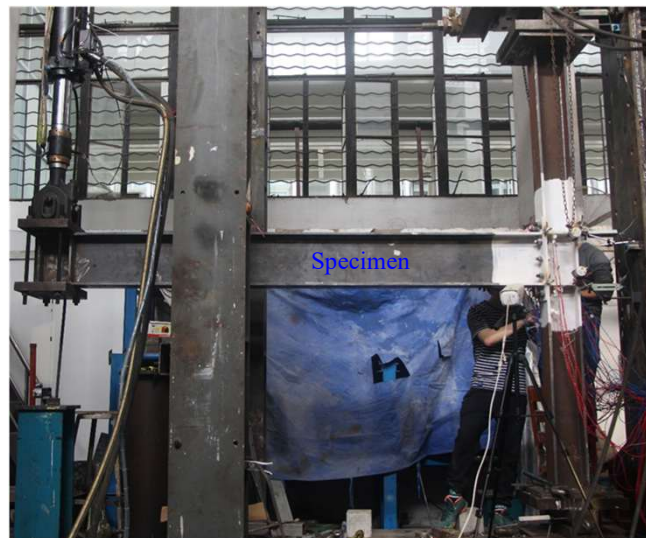
The experimental configuration, illustrated in Figure 2, features a beam-to-column connection specimen mounted within a high-capacity reaction frame. The test setup was designed to accurately simulate boundary conditions of the prototype structure through unidirectional hinge supports at both column ends positioned at mid-story height (1500 mm from the beam centerline). A 300-ton capacity hydraulic jack was installed at column top applying constant axial compression (30% of the axial yield load, P_y). A 30-ton capacity servo-hydraulic actuator with a stroke range of ± 250 mm was mounted at beam end, 3000 mm away from the column centerline representing a half-span distance. A lateral restraint system with roller bearings was employed to prevent out-of-plane instability while minimizing frictional effects.

2.4 Loading protocol

The loading protocol consisted of two distinct phases: i) preloading phase for instrumentation calibration and zeroing, and ii) formal testing phase to apply displacement-controlled cyclic



(a) Schematic



(b) With installed specimen prior to testing

Figure 2. Overview of the test setup

loading per AISC Seismic Provisions [38], with continuous monitoring until fracture of critical joint components (e.g., end plate fracture, bolt rupture) occurred.

2.5 Instrumentation

As shown in Figure 2(a), the actuator force, F , was measured by a built-in load cell in the actuator, with push force defined as positive and pull force as negative. The measured displacements include beam-end displacement (Δ), diagonal distance variations (Δ_{pz1} and Δ_{pz2}) at the joint panel zone corners, horizontal displacements (Δ_{c1} and Δ_{c2}) at the column flanges aligned with continuity plate centerlines, horizontal gap openings (Δ_{ep1} and Δ_{ep2}) at the end-plate locations aligned with beam flange centerlines. All displacements follow the positive direction indicated by arrows in Figure 2(a).

The story drift angle (SDA), φ , which is usually used for loading control and performance evaluation of steel moment frames, indicates the behavior of the whole beam-to-column assembly. It is defined as:

$$\varphi = \frac{\Delta}{L/2} \quad (9)$$

where L is the beam span (6000 mm). The SDA consists of deformations of the beam (θ_b), column, and rotation of the end-plate joint, which includes shear distortion (γ) of the panel zone and connection rotation (θ_{ep}) due to opening between the end plate and column flange [39, 40]:

$$\gamma = \frac{\Delta_{pz1} - \Delta_{pz2}}{2} \frac{\sqrt{b_{pz}^2 + h_{pz}^2}}{b_{pz}h_{pz}} \quad (10)$$

$$\theta_{ep} = \frac{\Delta_{ep1} - \Delta_{ep2}}{h_b - t_{fb}} \quad (11)$$

$$\theta_b = \varphi - \theta_{ep} \frac{L - h_c}{L} - \frac{\Delta_{c1} - \Delta_{c2}}{h_b - t_{fb}} \quad (12)$$

Note that for each specimen, strain gauges were glued to the beam flange and web surfaces near the face of the column to detect yielding, as well as a strain rosette to the center of the panel

zone to monitor its shear yielding.

3 Test results

3.1 Test observations and failure modes

The specimens exhibited distinct failure mechanisms depending on the design plastic failure sequence of joint components (see Table 1), including bolt rupture, end-plate cracking and fracture, or combined failure modes, as illustrated in Figure 3. In the following test observations, the north and south sides are indicated in Figure 2(a) when describing bolt locations. It should be noted that nothing happened to columns outside panel zones in all specimens throughout testing, due to the SCWB design adopted to ensure elastic column behavior. Local buckling was never observed as the beam cross section was rather compact.

For Specimen B355-C690-EP16, the beam flanges near the end plate developed yielding at the SDA amplitude of 3%, at which the end plate bending was also noticeable, followed by the beam web and panel zone at the SDA amplitude of 4%. No other notable observations were made until the first cycle to -6% SDA, when two outer bolts at the bottom beam flange fractured at -5% SDA, followed by fracture of two inner bolts at -5.8% SDA. In the subsequent second cycle, two outer bolts at the top beam flange fractured at 4.7% SDA, immediately followed by fracture of two inner bolts at 5.4% SDA. After completing this cycle, testing was terminated with observed cracking at the weld toe between the top beam flange and extended end plate, though without complete fracture of the end plate, as shown in Figure 3(a).

For Specimen B355-C690-EP12, both the beam and the panel zone basically remained elastic throughout testing. Only bending deformations were obviously noted in the end plate starting from the SDA amplitude of 3%. Unlike Specimen B355-C690-EP16 which failed by bolt fracture before apparent cracking in the end plate, Specimen B355-C690-EP12 exhibited initial cracking at the flange-to-end-plate weld toes after 5% SDA amplitude. During the first cycle of 6% SDA, two inner bolts at the top beam flange fractured nearly simultaneously at 5% SDA, while the south

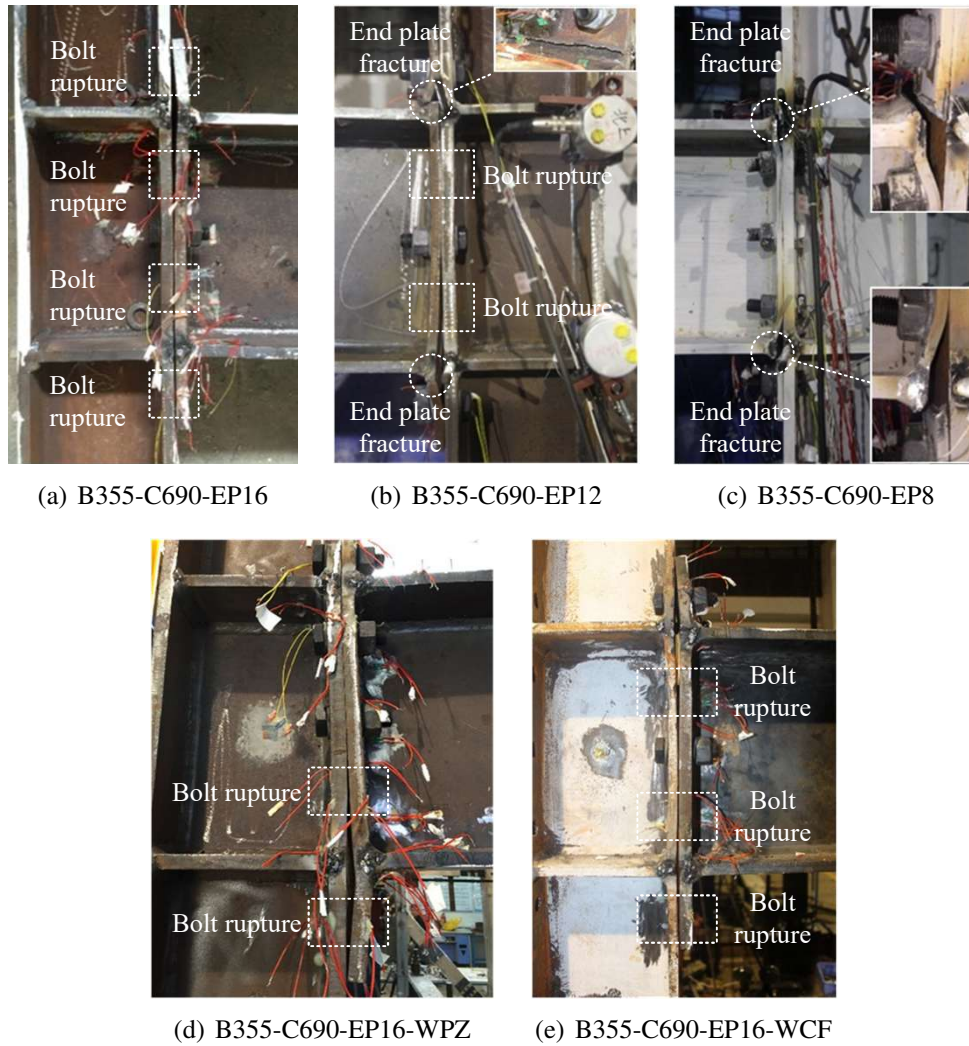


Figure 3. Failure modes of the test specimens

inner bolt at the bottom beam flange fractured at -4.5% SDA, followed by the north inner one at -5% SDA. In the second cycle of 6% SDA, complete penetration of cracks occurred at both the top end-plate weld toe (6% SDA) and bottom end-plate weld toe (-6% SDA). Final failure showed complete separation of both upper and lower extended end plates from the beam flanges, as illustrated in Figure 3(b).

For Specimen B355-C690-EP8, no yielding was detected in the beam or panel zone. Notable behavior of Specimen B355-C690-EP8 initiated with cracking at the bottom end-plate weld toe

219 during the first cycle to -3% SDA, followed by cracking at the top end-plate weld toe in the
 220 second cycle to 3% SDA. These cracks progressively propagated, with partial penetration of the
 221 end plate occurring after 4% SDA, resulting in significant strength degradation during the second
 222 cycle of 4% SDA. Complete fractures of both top and bottom extended end plates occurred during
 223 the first cycle of 5% SDA, as shown in Figure 3(c). Subsequent loading in the second cycle of 5%
 224 SDA revealed full-penetration cracking at the north end-plate weld toe inside the top beam flange
 225 at 4.3% SDA (see Figure 4(a)), accompanied by a 30mm crack at the south end-plate weld toe. At
 226 -5% SDA of this cycle, a 20mm crack developed at the north end-plate weld toe inside the bottom
 227 beam flange. Further loading produced a 40mm crack at the south end-plate weld toe inside the
 228 bottom beam flange during the first cycle to -6% SDA (see Figure 4(b)). Testing concluded with
 229 only the web-to-end-plate welds remaining intact and all bolts surviving without fracture.

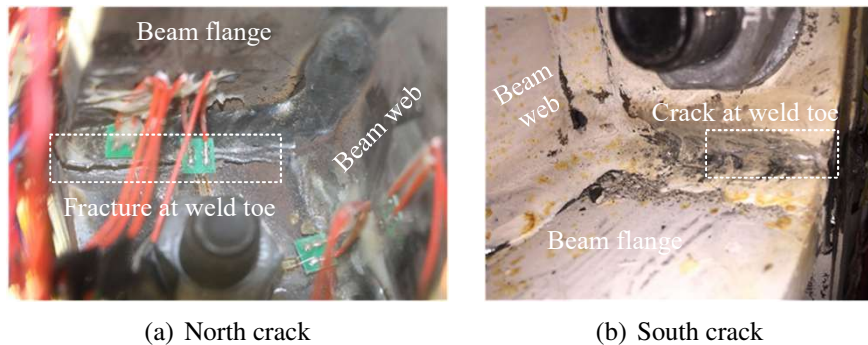


Figure 4. Cracks in the end plate inside beam flanges of Specimen B355-C690-EP8

230 For Specimen B355-C690-EP16-WPZ, the beam remained elastic as expected. The specimen
 231 experienced panel zone yielding starting from the first cycle of 2% SDA, and demonstrated
 232 significant panel zone shear deformation initiating at cycles of 3% SDA, evidenced by spalling of
 233 the whitewash on the specimen surface. During the first cycle of 8% SDA, the south outer bolt at
 234 the bottom beam flange fractured at -6.9% SDA, followed by fracture of both the north outer and
 235 inner bolts at the bottom flange at -7.3% SDA, as depicted in Figure 3(d).

236 For specimen B355-C690-EP16-WCF, observable end plate bending deformations were noted

since the SDA amplitude of 3%, and slight yielding was noticed in the beam flange and panel zone since the SDA amplitude of 5%. The north inner bolt at the bottom beam flange fractured at -4.6% SDA during the second cycle to -5% SDA. Subsequent loading produced fracture of the north outer bolt at the bottom flange at -4.3% SDA and the south outer bolt at -5.5% SDA during the first cycle to -6% SDA. The north inner bolt at the top beam flange ultimately fractured at 5.9% SDA in the second cycle to 6% SDA, as shown in Figure 3(e), marking the termination of testing after completing the positive excursion of this cycle.

It is noteworthy that the experimentally observed failure modes are in good agreement with the plastic failure sequences predicted by the component method of Eurocode 3 Part 1-8 (Table 2). In the component method, the equivalent T-stub of a bolt row may fail in one of three modes: Mode 1, complete yielding of the flange, so that the end plate fractures with the bolts essentially intact; Mode 2, bolt failure with yielding of the flange; and Mode 3, bolt failure. Mode 2 and Mode 3 both culminate in bolt rupture. Since the component method adopts an ideal elastic-perfectly-plastic assumption, and since the strain-hardening capacity of the end-plate steel is markedly higher than that of the high-strength bolt material, a theoretical Mode 2 or Mode 3 mechanism is expected to ultimately develop into bolt rupture accompanied by substantial residual bending of the end plate under cyclic loading, exactly as observed in the tests (Figure 3). Although the component method was originally developed for monotonic static design, the present results demonstrate that it nevertheless provides a valuable reference for predicting the failure modes of Q690 high-strength steel end-plate connections under cyclic (seismic) loading.

3.2 Hysteresis and backbone curves

Moment-SDA ($M-\varphi$) hysteresis curves for all specimens are shown in Figure 5, where the moment is that at the column face, calculated by multiplying the beam tip reaction force by the lever arm (3000 mm minus half the column section height). The beam's first yield moment ($M_{y,b}$) and full plastic moment ($M_{p,b}$), determined from measured material properties, are included for reference. Moment-panel zone shear distortion ($M-\gamma$) curves, moment-connection rotation

($M-\theta_{ep}$) curves, and moment-beam rotation ($M-\theta_b$) curves are shown in Figures 6, 7 and 8, respectively, with $M_{y,pz}$ indicating the column face moment at panel zone yielding based on measured material strengths [33, 34].

The results demonstrate that only Specimens B355-C690-EP16 and B355-C690-EP16-WCF reached $M_{y,b}$ at 4% SDA, though their ultimate (or peak) moments remained below $M_{p,b}$. The other four specimens maintained essentially elastic beam behavior throughout testing, consistent with the design intent. Some shifts even at the initial elastic loading stage in the hysteresis curves of the beams are apparently observed in Figures 8(c) and 8(e), owing to measurement errors associated with small end-plate rotations since the beam rotation is a derived quantity. Specimen B355-C690-EP16-WPZ exhibited exceptionally stable and full hysteresis loops (see Figure 6(d)), reflecting the effective energy dissipation capacity of its intentionally weakened panel zone design. In contrast, the remaining five specimens showed pronounced pinching effects in their $M-\theta_{ep}$ hysteresis curves, characteristic of bolted end-plate connections. Notably, the pinching effect was less pronounced in Specimen B355-C690-EP16-WPZ compared to the benchmark Specimen B355-C690-EP16, as the significant panel zone shear deformation in the former specimen altered the end-plate connection's deformation pattern and reduced bolt deformations.

Constructed from the $M-\varphi$ hysteresis curves by connecting each data point corresponding to the moment of the first cycle at each target story drift angle, $M-\varphi$ backbone curves are shown in Figure 9, revealing that Specimen B355-C690-EP16-WPZ achieved its peak moment at 7% SDA, demonstrating the superior deformation capacity of its panel zone. The other specimens generally reached their peak moments between 4–5% SDA. Specimens B355-C690-EP16 and B355-C690-EP16-WCF showed comparable moment strength, while the specimens with thinner end plates (B355-C690-EP12 and B355-C690-EP8) exhibited progressively reduced strength.

3.3 Mechanical indices

Following the ECCS recommended method [41], the yield and ultimate (peak) moment resistances (M_y and M_u) were determined from the $M-\varphi$ backbone curves, as shown in Table 4,

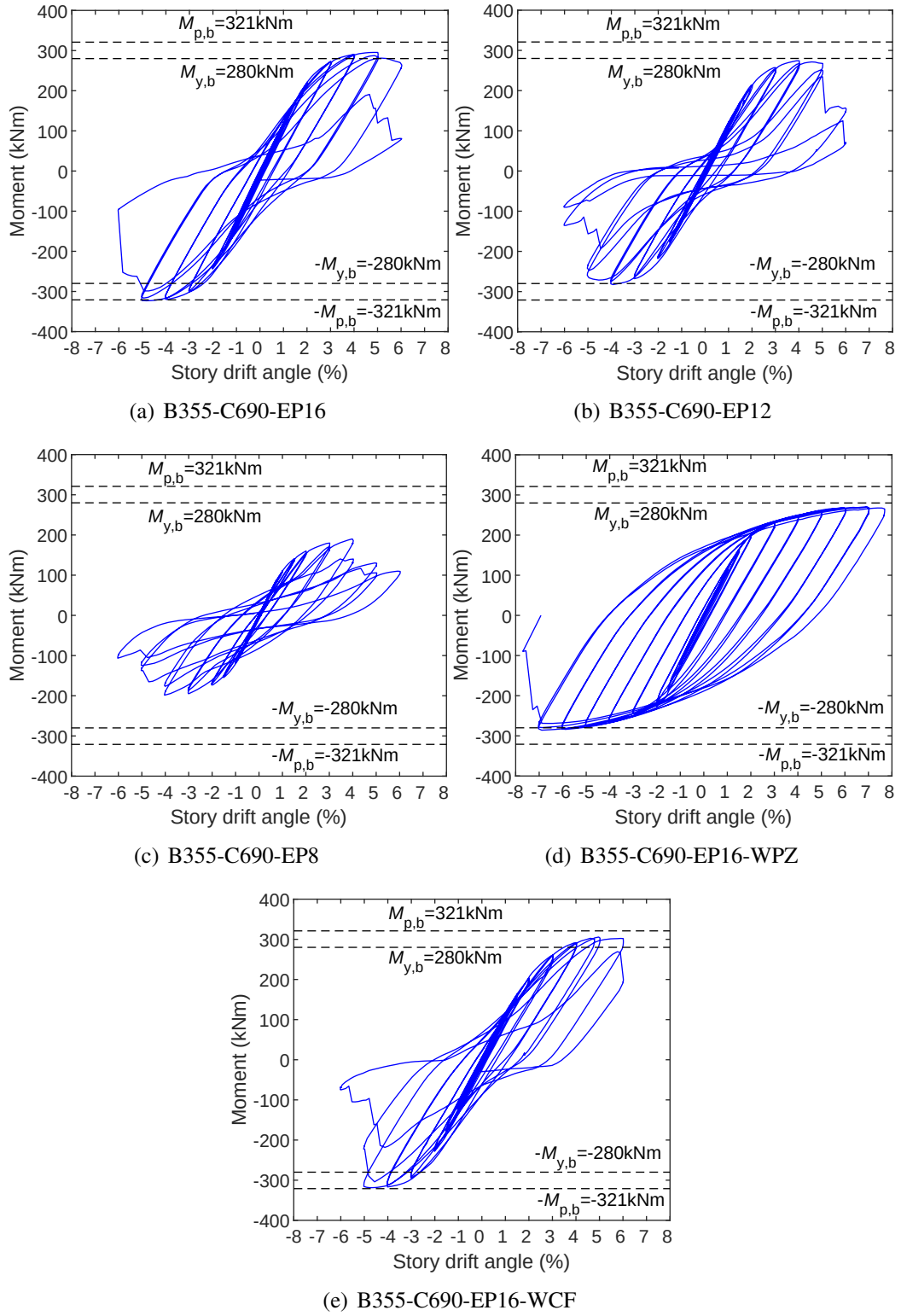


Figure 5. Moment–SDA (M – φ) hysteresis curves of the test specimens

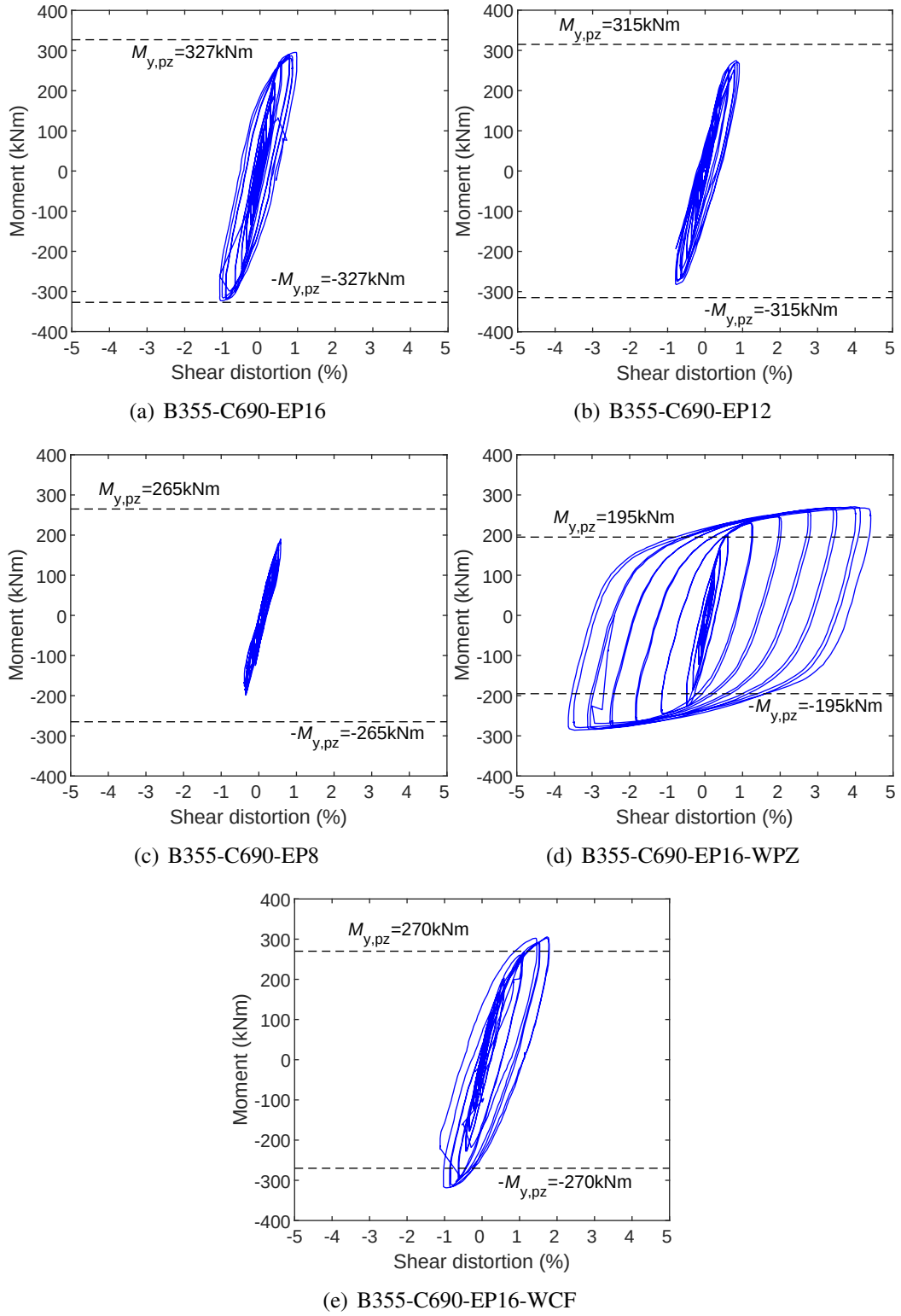


Figure 6. Moment–shear distortion (M – γ) hysteresis curves of the test specimens

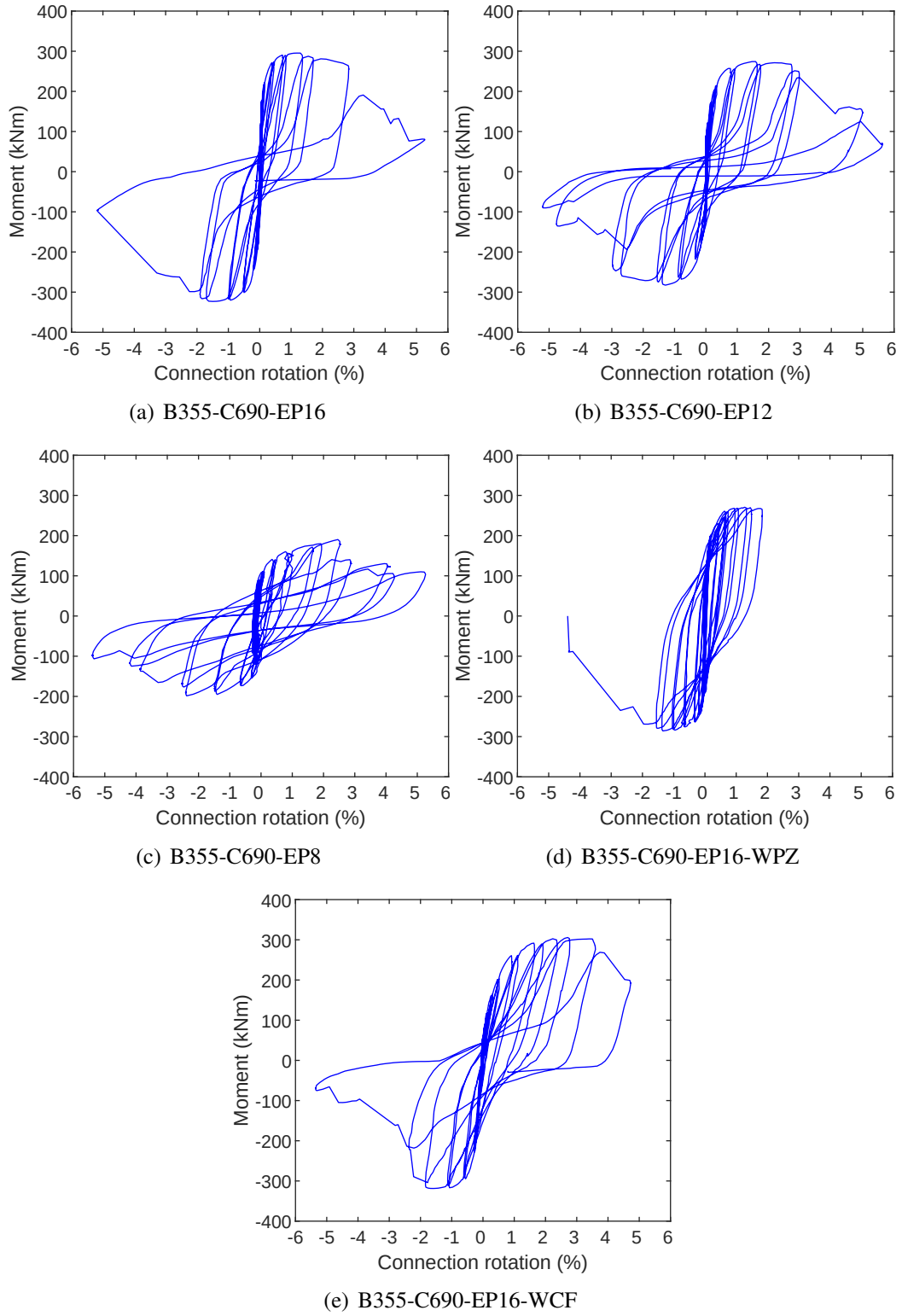


Figure 7. Moment–connection rotation ($M-\theta_{ep}$) hysteresis curves of the test specimens

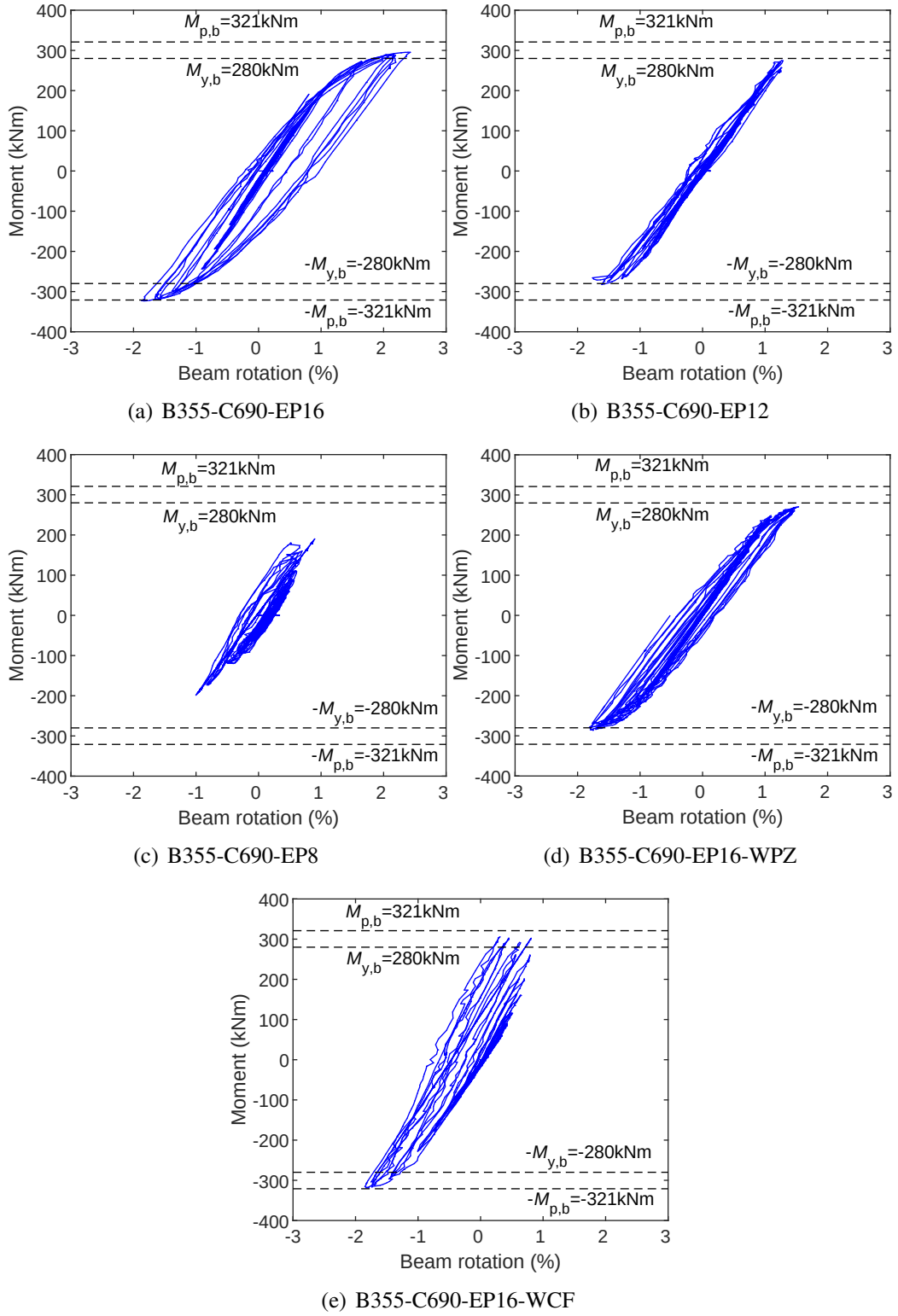


Figure 8. Moment–beam rotation ($M-\theta_b$) hysteresis curves of the test specimens

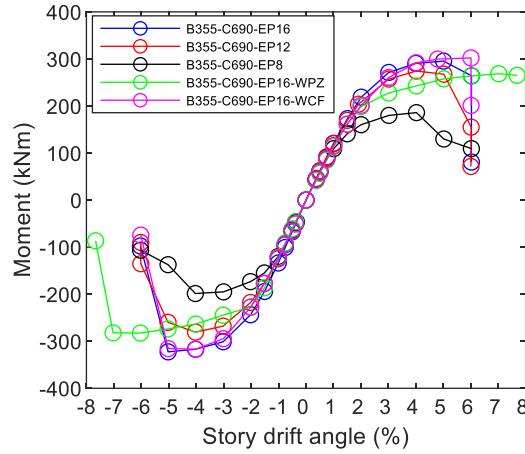


Figure 9. Moment–SDA (M – φ) backbone curves of the test specimens

where both M_y and M_u represent the average values of positive and negative loading directions. The design moment resistance ($M_{j,Rd}$) calculated using the component method in Eurocode 3 Part 1-8, based on the measured material yield strengths (Table 3) and without partial safety factors, is presented in Table 4 as well. The ratio, $M_{j,Rd}/M_y$, ranges between 0.85–0.98, confirming code applicability (slightly conservative) for high-strength steel extended end-plate connections and also validating the authors' previous findings from static performance studies [32]. Regarding joint stiffness, prior static tests have demonstrated that the component method tends to underestimate the initial rotational stiffness due to bolt pretension effects [32], therefore this aspect is not further discussed herein. Table 4 also shows that all specimens exhibited moment overstrength factors (M_u/M_y) within the range of 1.1–1.2.

Table 4. Results of moment resistance

Specimen label	M_y (kNm)	M_u (kNm)	$M_{j,Rd}$ (kNm)	$\frac{M_{j,Rd}}{M_y}$	$\frac{M_u}{M_y}$
B355-C690-EP16	281.2	309.0	275.5	0.98	1.10
B355-C690-EP12	253.5	277.5	245.6	0.97	1.09
B355-C690-EP8	169.2	192.2	152.5	0.90	1.14
B355-C690-EP16-WPZ	230.9	275.6	201.1	0.87	1.19
B355-C690-EP16-WCF	284.4	309.4	242.8	0.85	1.09

The ultimate SDA (φ_u) corresponding to the last amplitude to complete at least one full cycle before the first occurrence of either bolt rupture or complete end-plate fracture [42], with failure defined when either condition was observed, is summarized in Table 5. The plastic SDA (φ_p) and its components - the plastic rotations in the bolted end-plate connection ($\theta_{ep,p}$) and panel zone (γ_p) - are also included. The residual plastic rotation ($\theta_{b,p} = \varphi_p - \theta_{ep,p} - \gamma_p$) represents the contribution from beam yielding, as the columns remained elastic throughout all tests. Cumulative plastic SDA ($\Sigma\varphi_p$), connection rotation ($\Sigma\theta_{ep,p}$), shear distortion ($\Sigma\gamma_p$), and beam rotation ($\Sigma\theta_{b,p}$) before the failure criterion defined above reached were computed per ATC Guidelines [43], and are set out in Table 5 as well. Additionally, the cumulative energy dissipation capacity was quantified through four parameters [43]: total energy dissipation (A), connection energy dissipation (A_{ep}), panel zone energy dissipation (A_{pz}), and the residual energy ($A_b = A - A_{ep} - A_{pz}$) which similarly reflects the energy dissipated through beam yielding, as shown in Table 5. The results in Table 5 are also illustrated by Figure 10, showing contributions of the end-plate connection, panel zone, and beam to the overall maximum plastic SDA (φ_p), cumulative plastic SDA ($\Sigma\varphi_p$), and energy dissipation (A), respectively.

Table 5. Results of rotation capacity and energy dissipation

Specimen label	φ_u	φ_p	$\theta_{ep,p}$	γ_p	$\theta_{b,p}$	$\Sigma\varphi_p$	$\Sigma\theta_{ep,p}$	$\Sigma\gamma_p$	$\Sigma\theta_{b,p}$	A	A_{ep}	A_{pz}	A_b
		(mrad)	(mrad)	(mrad)	(mrad)	(mrad)	(mrad)	(mrad)	(mrad)	(kJ)	(kJ)	(kJ)	(kJ)
B355-C690-EP16	5%	26	17	4	5	304	116	82	106	67.5	22.1	20.0	25.4
B355-C690-EP12	5%	28	25	2	1	291	224	60	7	43.7	29.4	12.3	2
B355-C690-EP8	4%	24	23	1	0	219	199	20	0	26.3	23.1	3.2	0
B355-C690-EP16-WPZ	7%	46	8	35	3	1044	203	772	69	225.6	37.3	172.7	15.6
B355-C690-EP16-WCF	5%	24	15	5	4	228	114	111	3	48.8	23.2	24.1	1.5

As evidenced in Table 5 and Figure 10, the cumulative damage effect under cyclic loading

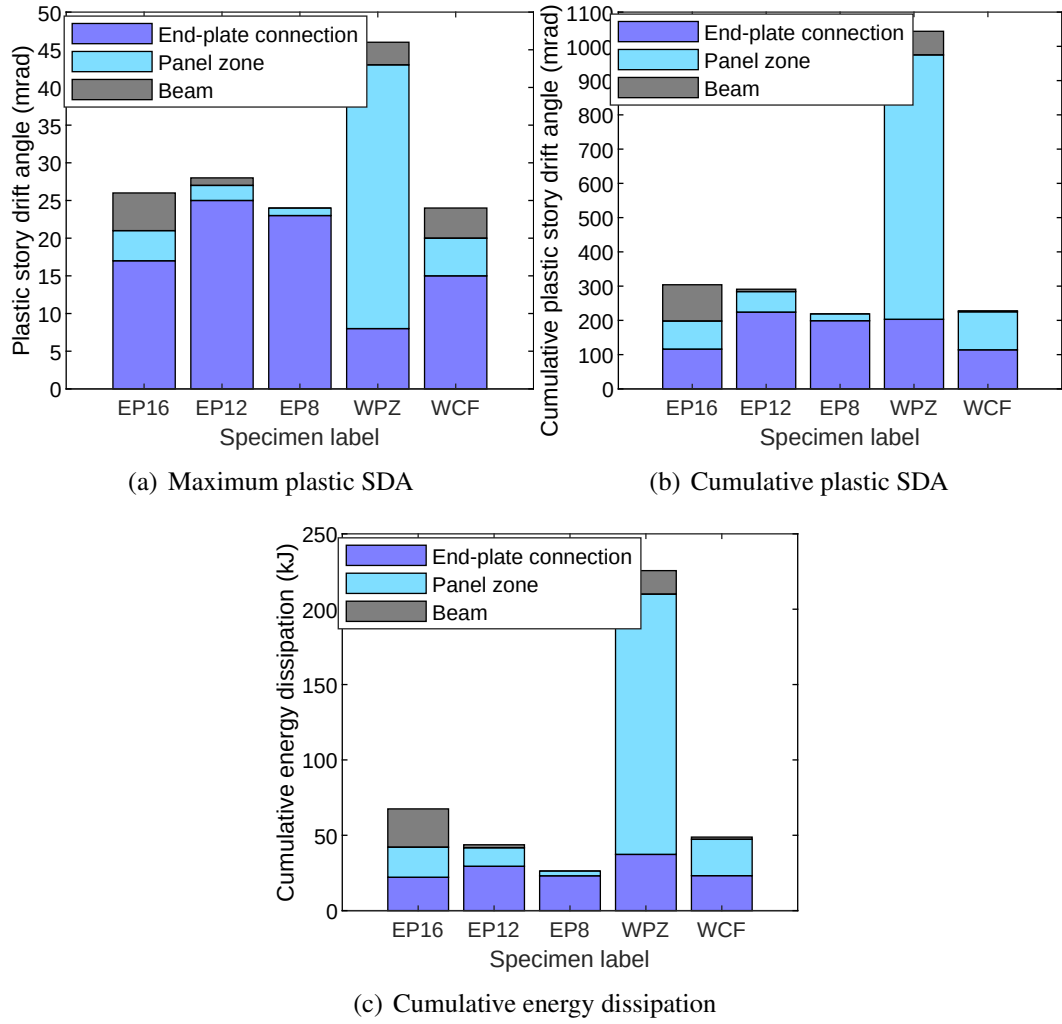


Figure 10. Contributions of joint components to plastic deformation and energy dissipation capacities

leads to reduced plastic rotation capacity for connections with thinner end plates. Specimen B355-C690-EP8 exhibited the lowest values of both φ_p and $\Sigma\varphi_p$ among all specimens, demonstrating behavior distinctly different from its monotonic loading performance reported in the authors' previous study [32]. While specimen B355-C690-EP16 showed the largest $\Sigma\varphi_p$ among the three specimens with varying end plate thicknesses, this should not be interpreted as indicating that thicker end plates universally enhance cumulative plastic deformation capacity. The parameter $\Sigma\varphi_p$ represents the total cumulative plastic rotation of the entire beam-joint

system, incorporating contributions from all components. When examining specifically the connection contribution ($\Sigma\theta_{ep,p}$), the relationship with end plate thickness is not monotonic - the maximum value (224 mrad) occurs at the intermediate thickness (12 mm), suggesting that both excessively thick and thin end plates adversely affect the hysteretic energy dissipation capacity. The relatively high $\Sigma\varphi_p$ of the benchmark Specimen B355-C690-EP16 primarily resulted from beam-end yielding (see Figure 8(a)), with its $\Sigma\theta_{b,p}$ contribution ($= \Sigma\varphi_p - \Sigma\theta_{ep,p} - \Sigma\gamma_p = 106$ mrad) being significant. These observations are further corroborated by the cumulative energy dissipation analysis.

The weak panel zone specimen (B355-C690-EP16-WPZ) demonstrated exceptional performance, with $\Sigma\varphi_p$ exceeding 1 rad ($A > 220$ kJ), far surpassing other specimens. This superior behavior stems primarily from its remarkable panel zone cumulative plastic deformation ($\Sigma\gamma_p = 772$ mrad, $A_{pz} = 173$ kJ), with no observed panel zone failure, confirming the outstanding cumulative deformation and energy dissipation capacity of Q690 high-strength steel panel zones [33, 34]. Interestingly, this specimen also exhibited significantly higher $\Sigma\theta_{ep,p}$ (203 mrad) compared to the benchmark specimen (116 mrad), indicating that panel zone plastic deformation can actually enhance the rotation capacity of end-plate connections - a finding that contrasts sharply with conventional wisdom regarding welded moment connections, where excessive panel zone deformation typically compromises connection performance. This phenomenon likely results from deformation compatibility between the end plate and panel zone, which alters the boundary conditions and force distribution of the end plate in bending when idealized as equivalent T-stubs.

The weak column flange specimen (B355-C690-EP16-WCF) achieved $\Sigma\varphi_p = 228$ mrad ($A = 49$ kJ), still lower than the benchmark specimen (304 mrad and 68 kJ). This reduction primarily resulted from earlier failure of the former specimen, with its $\Sigma\theta_{b,p}$ contribution (nearly zero) being substantially less than the latter specimen (106 mrad), despite comparable connection contributions ($\Sigma\theta_{ep,p} \approx 115$ mrad, $A_{ep} = 22\text{--}23$ kJ for both). These results confirm that column

flanges thinner than the end plate do adversely affect the cumulative plastic deformation and energy dissipation capacity of the joint system.

4 Discussion

4.1 Rotation capacity of bolted end-plate connection

According to Eurocode 3 Part 1-8 [23], a joint with bolted end-plate connection may be assumed to have sufficient rotation capacity (typically ≥ 35 mrad) for plastic analysis, provided that both of the following conditions are satisfied: i) the moment resistance of the joint is governed by the resistance of either the column flange or the end plate in bending, and ii) the thickness, t , of either the column flange (t_{fc}) or end plate (t_e) – not necessarily the same component as i) – satisfies:

$$t \leq t_{EC3} = 0.36d_b \sqrt{\frac{f_{u,b}}{f_y}} \quad (13)$$

where $f_{u,b}$ and d_b are the tensile strength and diameter of the bolt, respectively; f_y is the yield strength of the relevant component in bending (i.e., the column flange, or end plate). Prior studies [20, 21, 31, 32] have evidenced the applicability of Eq. (13) for high-strength steel end-plate connections, despite it being too strict in some cases. Hence, the authors proposed the use of the dimensionless parameter β ($= M_{pf} / (Bm)$), where M_{pf} is the plastic moment resistance of the equivalent T-stub flange, B is the tension resistance of a single bolt, and m is the distance between the bolt axis and the plastic hinge location at the flange-to-web connection in the T-stub flange) corresponding to the equivalent T-stub representing the first tension bolt row to meet $\beta \leq \beta_{12}$ (β_{12} is the boundary between Mode 1 and Mode 2 failure [11]) for sufficient plastic rotation [32]. To evaluate the above rotation capacity requirements, the ratio, t_{EC3}/t , according to Eq. (13), as well as β and β_{12} , are summarized in Table 6.

It should be noted that either Eq. (13) or the authors' proposal of the use of β can only ensure the rotation capacity under monotonic loading, but does not work well for connections under

Table 6. Assessment of code specifications

Specimen label	t_{EC3}/t	β	β_{12}	$h_{wc}/t_{wc} \leq 69\epsilon?$	$V_{wp,Ed}/V_{wp,Rd}$	γ_p/φ_p	$\Sigma\gamma_p/\Sigma\varphi_p$
B355-C690-EP16	0.53	0.83	0.53	Yes	1.07	15%	27%
B355-C690-EP12	0.73	0.44	0.53	Yes	1.07	7%	21%
B355-C690-EP8	1.13	0.18	0.53	Yes	0.90	4%	9%
B355-C690-EP16-WPZ	0.53	0.83	0.53	Yes	1.63	76%	74%
B355-C690-EP16-WCF	0.79	0.83	0.53	Yes	1.23	21%	49%

Note: t_{EC3}/t , β and β_{12} evaluate the Eurocode 3 Part 1-8 rotation-capacity criterion (Eq. (13)) and the authors' proposed criterion, respectively; $h_{wc}/t_{wc} \leq 69\epsilon$ and $V_{wp,Ed}/V_{wp,Rd}$ are the Eurocode 3 Part 1-8 panel-zone stability check and the Eurocode 8 Part 1 panel-zone resistance ratio, respectively; γ_p/φ_p and $\Sigma\gamma_p/\Sigma\varphi_p$ are the panel-zone contributions to the maximum and cumulative plastic SDA.

cyclic loading. This is evident, since all specimens could not survive up to a plastic connection rotation of 35 mrad (see $\theta_{ep,p}$ in Table 5), even including Specimen B355-C690-EP8 which is well qualified as satisfying Eq. (13) or the limit of β_{12} (see Table 6). The cause is cumulative damage under cyclic loading, resulting in early cracks followed by complete fracture of the end plate. The best end-plate connection rotation capacity ($\theta_{ep,p} = 25$ mrad and $\Sigma\theta_{ep,p} = 224$ mrad by Specimen B355-C690-EP-12, as shown in Table 5) corresponds to the specimen with the closest β value to the limit (β_{12} , see Table 6), potentially indicating an optimal seismic design.

As per Eurocode 8 Part 1 [24], when beam-to-column joints in seismic-resistant steel moment frames are designed as energy-dissipating zones, the rotation capacity of the plastic hinge region is not less than 35 mrad for structures of DCH and 25 mrad of DCM, and this rotation capacity should be ensured under cyclic loading without strength and stiffness degradation exceeding 20%. While the plastic SDAs (φ_p) of all specimens shown in Table 5 nearly satisfy the DCM requirement (25 mrad), the bolted end-plate connections, when considered individually, generally fail to meet this criterion (see $\theta_{ep,p}$ in Table 5). Considering the inherent conservatism required for seismic design, it is recommended to adopt the design concept of DCL (Ductility Class Low) for high-strength steel bolted end-plate connections.

4.2 Contribution of panel zone

It is generally recognized that panel zone (or column web panel) yielding can develop substantial plastic rotation in joints. According to Eurocode 3 Part 1-8 [23], a joint whose design moment resistance is governed by the design resistance of the panel zone in shear may be assumed to have adequate rotation capacity for plastic analysis, provided that $h_{wc}/t_{wc} \leq 69\epsilon$, where h_{wc}/t_{wc} is the slenderness of the column web panel and $\epsilon = \sqrt{235/f_y}$. It is easy to verify that all specimens in Table 6 meet such criterion. Furthermore, as for seismic design, Eurocode 8 Part 1 [24] specifies that the column web panel shear resistance should conform to $V_{wp,Ed}/V_{wp,Rd} \leq 1.0$ and the column web panel shear deformation should not contribute for more than 30% of the plastic rotation capability, where $V_{wp,Ed}$ is the design shear force in the web panel due to development of the plastic resistance of the adjacent dissipative zones in beams or connections, and $V_{wp,Rd}$ is the shear resistance of the web panel. Results of the ratio, $V_{wp,Ed}/V_{wp,Rd}$, are set out in Table 6, as well as γ_p/φ_p and $\Sigma\gamma_p/\Sigma\varphi_p$ demonstrating web panel contributions to the total plastic SDA in terms of maximum and cumulative quantities, respectively.

Clearly, only Specimen B355-C690-EP8 satisfies the above rotation requirements by Eurocode 8 Part 1, while Specimens B355-C690-EP16, B355-C690-EP12 and B355-C690-EP16-WCF all exhibit panel resistance ratios ($V_{wp,Ed}/V_{wp,Rd}$) exceeding 1.0, but their web panel contributions to joint plastic rotation (γ_p/φ_p) remain within 30%. However, the weak web panel specimen, B355-C690-EP16-WPZ, which highly outperformed other specimens in terms of plastic rotation and energy dissipation, is characterized with both much higher panel contribution (76% and 74% in maximum and cumulative quantities, respectively) and resistance ratio (1.63) than the code limits. This specimen is the only one that conforms to the ductility requirement (> 35 mrad) for structures of DCH. The results are consistent with prior studies that recommended extending this contribution to 50% [44] or found experimental contributions even more than 70% [22]. Therefore, the current code requirement of $V_{wp,Ed}/V_{wp,Rd} \leq 1.0$ is too conservative, and the code limit of 30%

413 contribution of the web panel is too strict as well and it can be relaxed at least to 70%. It should be
 414 borne in mind, however, that a large panel-zone shear deformation amplifies the second-order (P-
 415 Δ) effects on the frame and reduces the post-earthquake reparability of the structure. Accordingly,
 416 the above relaxation is recommended only on the condition that the panel-zone shear deformation
 417 is explicitly considered in the structural analysis; when the panel-zone deformation is neglected in
 418 a simplified analysis, the current code provisions should continue to apply.

419 In light of the superior inelastic deformation capacity of high-strength steel panel zones to be
 420 exploited in seismic design, a mechanical model for hysteretic analysis is required to accurately
 421 predict their shear stiffness, strength, and deformation capacity. Krawinkler [45] proposed a
 422 trilinear model (hereafter referred to the Krawinkler model) characterized by the elastic shear
 423 stiffness, K_e , up to the yield shear strength, V_y , followed by a post-yield hardening branch with
 424 the stiffness, K_p , up to a shear strength, V_p , at a shear distortion $\gamma_p = 4\gamma_y = 4V_y/K_e$,

$$K_e = 0.95h_c t_{pz} G \quad (14)$$

$$V_y = \frac{f_{yc}}{\sqrt{3}} 0.95h_c t_{pz} \quad (15)$$

$$K_p = \frac{1.15b_c t_{fc}^2}{h_b h_c t_{pz}} \quad (16)$$

$$V_p = V_y \left(1 + 3 \frac{K_p}{K_e} \right) \quad (17)$$

425 where G is the steel shear modulus of the panel zone. The post- γ_p stiffness is suggested as E_s/E
 426 times the elastic stiffness, K_e , in which E is the elastic modulus and E_s is the strain-hardening
 427 modulus of the steel [46]. For convenience, E_s can be taken as the slope between the end of yield
 428 plateau and the tensile strength point [19]. Skiadopoulos et al. [47] proposed another panel zone
 429 model to explicitly account for the effects of the panel zone aspect ratio, column flange width, and
 430 column flange thickness. Their model is also trilinear featuring three characteristic points at γ_y ,

431 $\gamma_p = 4\gamma_y$, and $6\gamma_y$. The elastic shear stiffness, K_e , and shear strength, V_y , evaluated at γ_y , are,

$$K_e = \frac{K_s K_b}{K_s + K_b} \quad (18)$$

$$K_s = (h_c - t_{fc}) t_{pz} G \quad (19)$$

$$K_b = \frac{12EI_c}{h_b^2} \quad (20)$$

$$V_y = \frac{f_{yc}}{\sqrt{3}} a_y (h_c - t_{fc}) t_{pz} \quad (21)$$

432 where I_c denotes the moment of inertia of the column cross section; a_y equals 0.9 and 1.0 for
 433 slender ($K_f/K_e < 0.02$) and stocky ($K_f/K_e > 0.07$) panel zones, respectively. Note that K_f denotes
 434 the column flange stiffness, evaluated by,

$$K_f = \frac{K_{sf} K_{bf}}{K_{sf} + K_{bf}} \quad (22)$$

$$K_{sf} = 2t_{fc} b_c G \quad (23)$$

$$K_{bf} = \frac{2Eb_c t_{fc}^3}{h_b^2} \quad (24)$$

435 For moderate panel zones with K_f/K_e ranging from 0.02–0.07, a_y is evaluated by,

$$a_y = \frac{0.58 (K_f/K_e + 0.88)}{1 - K_f/K_e} \quad (25)$$

436 The shear strength, V_{pz} , at both $\gamma_p (= 4\gamma_y)$ and $6\gamma_y$ is evaluated by a unified expression,

$$V_{pz} = \frac{f_y}{\sqrt{3}} \left[a_{w,eff} (h_c - t_{fc}) t_{pz} + a_{f,eff} (b_c - t_{pz}) 2t_{fc} \right] \quad (26)$$

437 where $a_{w,eff}$ equals 1.1 and 1.15 at γ_p and $6\gamma_y$, respectively; $a_{f,eff}$ equals 0.02, 0.1, and
 438 $0.93 (K_f/K_e) + 0.015$ at γ_p for slender panel zones, stocky panel zones, and other general cases,
 439 respectively, while it equals 0.03, 0.1, and $1.05 (K_f/K_e) + 0.02$ at $6\gamma_y$ for slender panel zones,

stocky panel zones, and other general cases, respectively. It should be noted that, the effect of column axial loads (P) on the shear strength of the panel zone can be included by multiplying the shear strength with a reduction factor, ρ , defined by von Mises yield criterion as,

$$\rho = \sqrt{1 - \left(\frac{P}{P_y}\right)^2} \quad (27)$$

The aforementioned models by Krawinkler [45] and Skiadopoulos et al. [47] are evaluated by three connection specimens with weak panel zones, i.e., B355-C690-EP16-WPZ in this study along with specimens B355-C690-PZ8b [33] and B690-C690-PZ12c [34] in prior tests by the authors. The shear force versus shear distortion ($V_{pz}-\gamma$) curves corresponding to the panel zones in the three specimens are shown in Figure 11, in which the test hysteretic response is compared with the predicted envelope curves based on both models. Interestingly, the comparisons highlight that, despite the more complexity in the model by Skiadopoulos et al. [47] to account for panel zone slenderness, the Krawinkler model generally outperforms Skiadopoulos et al. [47]' model. In particular, the model by Skiadopoulos et al. [47] tends to underestimate shear strength, while the elastic stiffnesses by two models are quite similar and both accurate. Note that, the panel zones are slender (K_f/K_e equals 0.025, 0.009, and 0.019 for the three specimens, respectively) according to the classification by Skiadopoulos et al. [47]. It is thus recommended that the Krawinkler model be used for inelastic seismic analysis of moment frames with high-strength steel panel zones, unless further evidence can be provided for improvement. Besides, a maximum plastic distortion capacity of 0.04 rad is suggested for the deformation limit, considering that Specimen B355-C690-PZ8b is the only specimen that underwent panel zone fracture after this plastic distortion [33]. It is further noted that the tested connections were subjected to a column axial load ratio of 0.3, which reduces the panel-zone shear resistance through the von Mises reduction factor ρ (Eq. (27)); the fact that the panel zone still developed its full, stable deformation and energy dissipation capacity under this axial load further supports the recommendation of exploiting the high-strength steel panel

zone in seismic design.

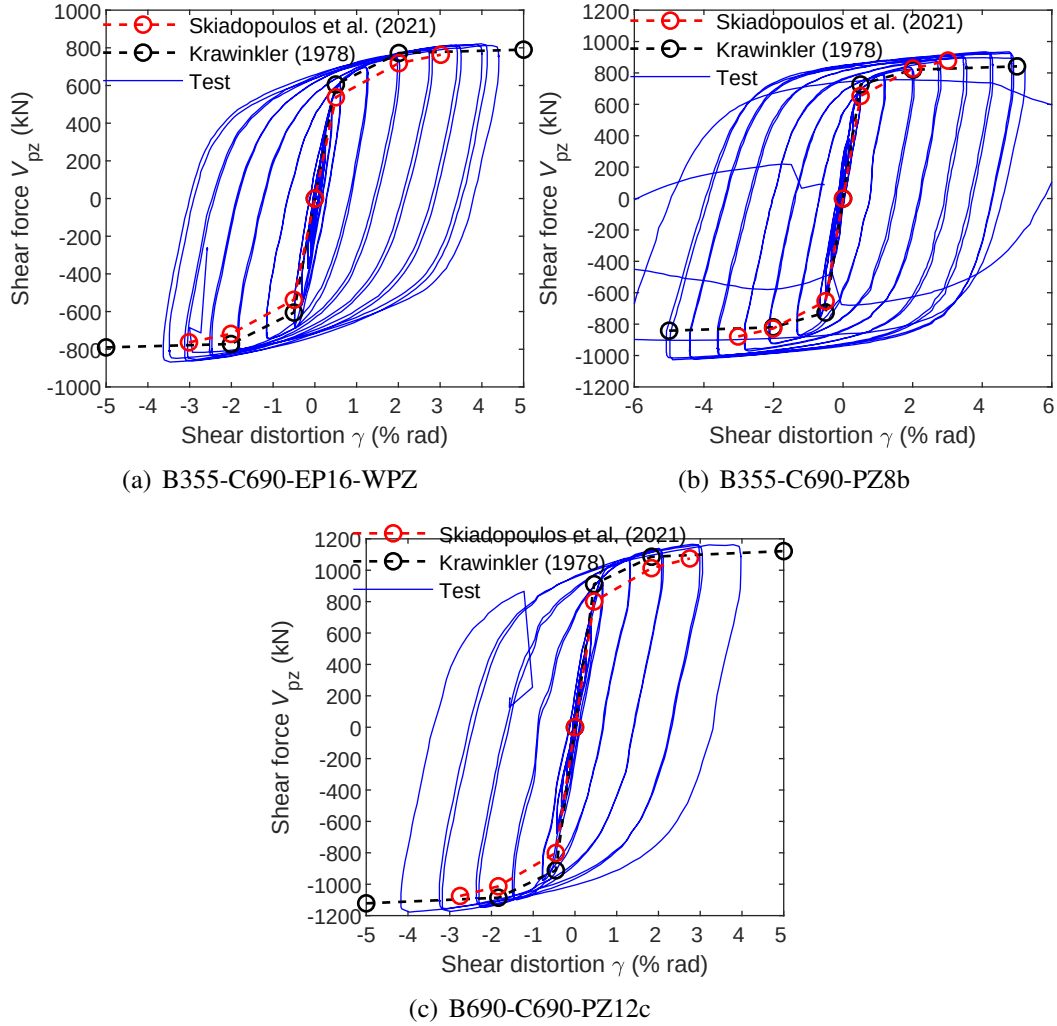


Figure 11. Assessment of panel zone models

5 Conclusions

This paper reports on an experimental investigation into the cyclic response of grade Q690 high-strength steel extended end-plate beam-to-column connections. A set of five one-sided joint specimens was tested, whose experimental results were extracted and analyzed comprehensively to verify current code provisions on rotation capacity. The study yields the following conclusions:

- 1) Q690 high-strength steel extended end-plate connections exhibit limited plastic rotation and energy dissipation capacity under cyclic loading, with maximum plastic rotations between 15–25 mrad. Thus, DCL (Ductility Class Low) may have to be enforced in seismic design by Eurocode 8 in cases that bolted end plates are energy-dissipating components. The results also suggest the existence of an optimal bolted end-plate design (for the equivalent T-stub representing the first tension bolt row) that maximizes cumulative plastic rotation and energy dissipation, which, if expressed by the dimensionless parameter $\beta = 2M_{pf}/(Bm)$, lies close to the boundary between Mode 1 (end-plate yielding followed by fracture) and Mode 2 (bolt rupture) failure; this optimum, however, requires confirmation through a broader experimental and numerical parametric study.
- 2) Q690 high-strength steel panel zones demonstrate stable and superior plastic deformation capacity, achieving maximum plastic shear distortions up to 35 mrad under cyclic loading. This indicates that DCH (Ductility Class High) can be employed in seismic design, if panel zones are designed as energy-dissipating components. The contribution of panel zone deformation to the total joint plastic rotation can be safely increased to 70%, exceeding the conventional 30% limit specified in Eurocode 8. This relaxation, which is supported by the present test together with prior evidence [33, 34], is proposed on the condition that the panel-zone shear deformation is explicitly considered in the structural analysis and that the associated P- Δ effects and the reduced post-earthquake reparability are duly accounted for; a broader parametric confirmation is still required.
- 3) Thinner column flanges relative to the end-plate thickness may lead to premature connection failure and adversely affect the cumulative plastic rotation and energy dissipation capacity. For connections using matching steel grades and equipped with continuity plates, it is recommended to maintain the Chinese design requirement stipulating that the column flange thickness should not be less than the end-plate thickness.
- 4) The design moment resistance of Q690 high-strength steel extended end-plate connections

under cyclic loading can still be reliably determined using the component method formulas in Eurocode 3 Part 1-8. Experimental evidence indicates that the ultimate moment capacity typically ranges between 1.1–1.2 times the yield moment resistance, providing consistent overstrength factors for seismic design applications. The Krawinkler model is recommended for inelastic seismic analysis of Q690 high-strength steel panel zones.

Finally, the limitations of the present study should be acknowledged. The conclusions are drawn from five cyclic tests on a single connection configuration and therefore demonstrate the behavior of the tested specimens and identify clear, physically-based trends, but they do not yet constitute broadly applicable design rules. In particular, the proposed relaxation of the panel-zone contribution limit and the suggested optimal end-plate design require confirmation through a larger experimental database and through a validated finite-element model that can reproduce the fracture of the end plate and bolts, together with a comprehensive parametric study. The development of such a fracture-capable model is the subject of ongoing and future work.

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