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Geometric tolerances of welded connections with inelastic panel zones

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Abstract

Prequalified beam-to-column connections in steel moment resisting frames usually concentrate inelastic deformations in the steel beam ends. As such, nonlinear geometric instabilities (i.e., local buckling) may occur in these regions at modest lateral drift demands, leading to potentially high residual deformations. Previous research has shown that welded connections featuring inelastic panel zones exhibit a stable hysteretic behaviour. In this case, residual deformations in steel beams are often not visually detectable after inelastic cyclic straining. This paper contrasts the residual deformations of steel beams as part of welded connections with elastic and inelastic panel zones through continuum finite element analyses. The employed finite element model is thoroughly validated with available test data. Besides the actual hysteretic response of the connections, the primary focus of the present work is on a number of geometric tolerance indicators, such as the web concavity, flange out-of-square and beam axial shortening. These are strongly correlated to the potential for reuse of structural steel members. The results suggest that connection designs featuring inelastic panel zones delay beam local buckling relative to their strong panel zone counterparts, achieving acceptable residual deformations even at lateral drift demands higher than 4% when considering the geometric tolerances of international standards.

Keywords

steel moment resisting frames, welded connections, inelastic panel zone, geometric tolerance, local buckling, finite element analysis

1 Introduction

Steel moment-resisting frames (MRFs) are commonly used in earthquake-prone areas. In capacity-designed steel MRFs, inelastic deformations usually concentrate near the steel beam ends, as post-Northridge seismic design standards do not promote the participation of column web panel zones in the energy dissipation of a beam-to-column connection. This results in an increased potential for local buckling of the steel beams. Panel zone yielding has been associated with premature connection fractures in inadequately detailed welded connections [1]–[3]. Experiments in post-Northridge welded connections have demonstrated that these can achieve inelastic shear distortions up to $10\gamma_y$ (where γ_y represents the panel zone yield shear distortion angle) without exhibiting premature fracture [4], [5]. Collectively, this is attributed to state-of-the-art welding procedures, toughness-rated weld electrodes and an optimized access hole geometry [6].

While life safety remains a primary objective of earthquake engineering, the financial impacts in the aftermath of earthquakes have gained increased attention. Besides the earthquake-induced collapse risk, minimizing structural damage can reduce financial losses in typical mainshock-aftershock earthquake sequences. Steel MRF connections with elastic panel zones exhibit local buckling in their steel beams even at modest lateral drift demands. This issue is challenging from a repairability standpoint. Moreover, steel recycling/reuse is usually prohibited because the damaged steel beams do not respect the geometric tolerances of current international standards.

Prior studies on welded unreinforced flange-welded web (WUF-W) connections with highly inelastic panel zones have exhibited satisfactory performance without substantial strength and stiffness degradation up until lateral drift demands of 6% [7]–[10]. In a recent study, Skiadopoulos et al. [11] proposed an alternative WUF-W connection with highly inelastic panel zones and simplified weld details that exhibited superior hysteretic response up until 9% rad. The steel beams in this case, did not experience local buckling prior to story drift angle of about 6%. In welded connections featuring inelastic panel zones, the residual deformations in steel beams are considerably less pro-

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nounced compared to their strong panel zone counterparts, thereby offering advantages in terms of post-earthquake damage assessment of welded connections. The potential for reusing structural steel members may also be capitalized.

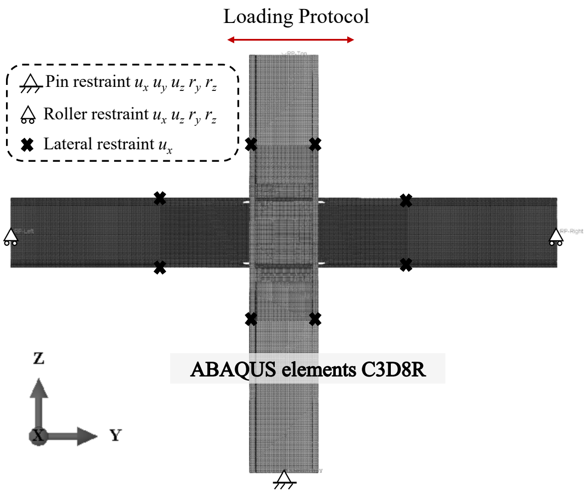
One option to assess, in part, the reusability of reclaimed steel members is to leverage standard geometric tolerances as a quantitative metric. Various international standards, including JIS G 3192 [12], EN10034 [13], ASTM A6/A6M [14], and GB/T 706 [15], provide quantitative recommendations on geometric tolerances of I-shaped (i.e., wide flange) steel beams (e.g., concavity, out-of-square, axial shortening, etc).

Although several studies have demonstrated the benefits of inelastic panel zone connection design approaches, a comprehensive understanding of the impact of the same design concept on the residual deformations of steel beams in welded connections remains scarce. This paper focuses on the geometric tolerance indicators, which are associated with the potential reuse of structural steel members. The residual deformations of steel beams in welded connections featuring elastic and inelastic panel zones are compared through continuum finite element (CFE) analyses. A CFE model is first developed and validated by leveraging available experimental data. The CFE approach is then used to examine the cyclic behaviour of welded connections with both elastic and inelastic panel zones by directly comparing their ability to respect the associated geometric tolerances from various international standards.

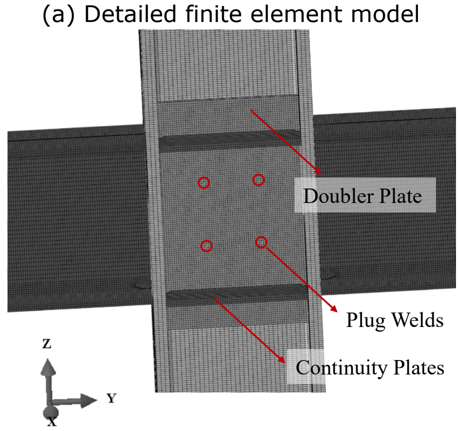
2 Development and validation of CFE model

Figure 1(a) illustrates a global view of a finite element model for a typical WUF-W connection. The CFE model consists of 8-node brick elements (C3D8R) with reduced integration and a maximum element size of 20 mm. Such elements typically avoid hourglassing and shear-locking. A mesh sensitivity analysis is performed to ascertain the ideal element type and mesh size to ensure accuracy in the simulation results at a viable computational cost. Initial geometric imperfections are introduced to the beam members, based on the modelling recommendations by prior research [16]. Parabolic residual stresses, based on Young's research [17], are incorporated into the beam members.

The CFE model accounts for steel material nonlinearity using the UVC model [18]. The input model parameters are based on A992 Gr.50 steel according to [18]. In Figure 1(b), the doubler plate is tied with the column flange. Hard contact is defined in the normal direction between the doubler plate and the column web. Conversely, friction is specified in the tangential direction. The friction coefficient is assumed to be 0.3. Four plug welds are established by using 15-mm fasteners, restraining all six degrees of freedom. The continuity plates are tied with the doubler plate and column flanges.



(a) Detailed finite element model



(b) Doubler plate detailing

Figure 1 Detailed continuum finite element model

The established CFE model was validated with prior experiments available in the literature [10]. Figure 2 depicts a comparison between the experimental data and the results from the CFE model in terms of column tip load displacement versus story drift ratio (SDR). The agreement between the two is noteworthy. Figure 3 provides a visual comparison of the deformed shape of the welded connection at an SDR of 6%, demonstrating that the CFE model is suitable for subsequent analyses.

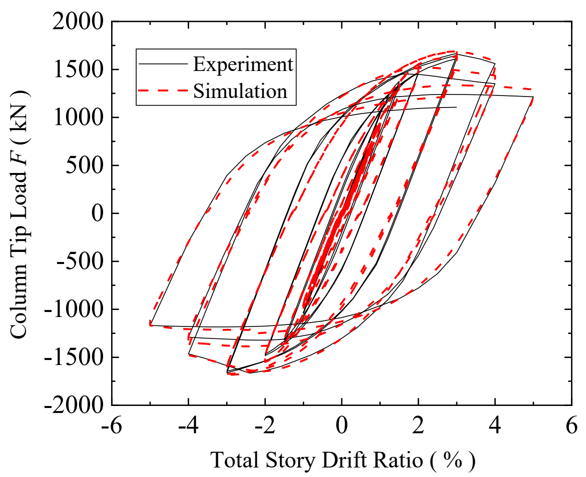
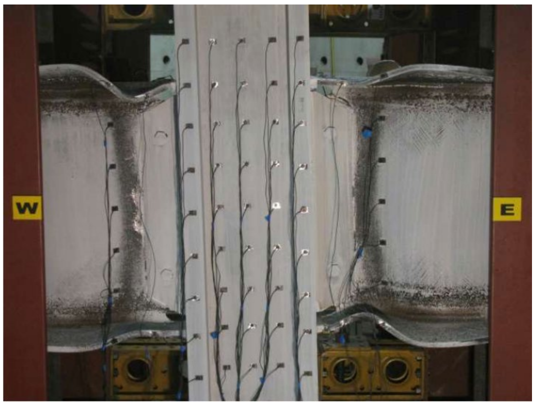
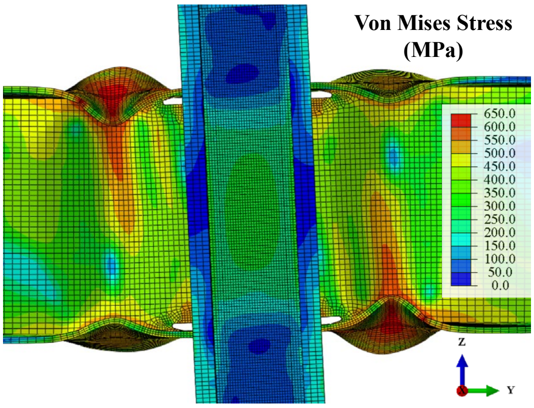


Figure 2 Comparison between experimental and CFE results



(a) Experiment



(b) CFE model

Figure 3 Comparison between experimental and CFE observations at an SDR of 6% (Image in (a) from [10])

3 Results and discussion

3.1 Hysteretic response

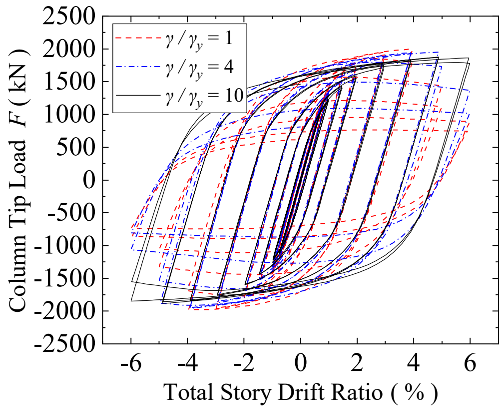
In order to investigate the differences in the performance of welded connections with strong and weak panel zones, three models with nominally identical structural beams, columns, and boundary conditions are established. The loading history involves a symmetric cyclic loading protocol according to AISC 341-16 [19]. The models differ only in the thickness of the doubler plate, which impacts the panel zone strength and demand at maximum considered earthquakes. The connections are designed in accordance with AISC 341-16 [19]. However, the column web panel shear resistance is computed via the Skiadopoulos et al. [20] panel zone model. Table 1 provides geometric and brief design details for the three considered subassemblies.

Table 1 Geometric and design details for the selected subassemblies

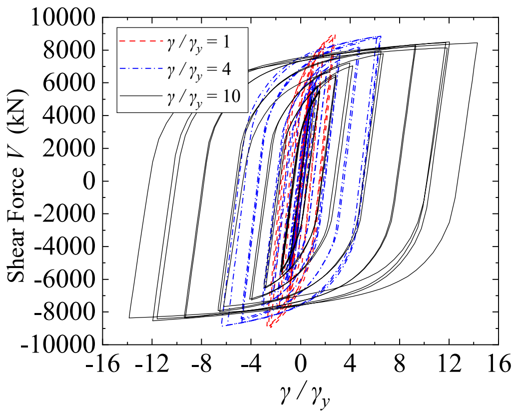
Beam	Column	Doubler plate (mm)	Panel zone demand	Resistance to demand ratio R_u^{pz}/R_n^{pz}
W30X148	W27X336	27.0	$\gamma/\gamma_y = 1$	0.72
W30X148	W27X336	14.3	$\gamma/\gamma_y = 4$	0.89
W30X148	W27X336	7.9	$\gamma/\gamma_y = 10$	1.01

Note: $R_u^{pz}$and R_n^{pz} represent the demand and resistance of the panel zone shear force, respectively, as defined in AISC 360-16 [21] and AISC 358-16 [22].

Figure 4(a) displays a comparison of the global response of the three subassemblies in terms of column tip load versus SDR. The connections featuring highly inelastic panel zones (γ/γ_y=10) demonstrate superior performance compared to those with elastic (γ/γ_y=1) and code-conforming inelastic shear distortions (γ/γ_y=4). Particularly, the welded connection with highly inelastic panel zones does not exhibit in-cycle strength deterioration due to the delay in beam local buckling, i.e., the hysteretic behaviour of the connection is stable up until an SDR of 6%. Conversely, the connections with the elastic and balanced panel zone design experience appreciable strength reductions early on in the loading history.



(a) Global Response



(b) Panel Zone Response

Figure 4 Comparison of the hysteretic response of subassembly models

Figure 4 (b) shows a comparison of the panel zone response between the three considered cases. Noteworthy stating that the welded connection with a highly inelastic panel zone (γ/γ_y=10) did not exhibit panel zone shear buckling.

3.2 Contributions to story drift ratio

The contribution of the beams, columns, and panel zones to the total SDR was quantified for the three examined subassembly models shown in Table 1. Figure 5 depicts these decomposed contributions. The results indicate that, prior to an SDR of 3%, the contribution of each structural component remains relatively stable. The model featuring a panel zone demand of $10\gamma_y$ exhibits the largest contribution from the panel zone as expected. At 4% SDR, the contribution of the beam in the model featuring γ_y panel zone demand is increased due to the formation of local buckling in the beam ends. Similarly, at 5% SDR, the contribution of the model with $4\gamma_y$ panel zone demand is increased.

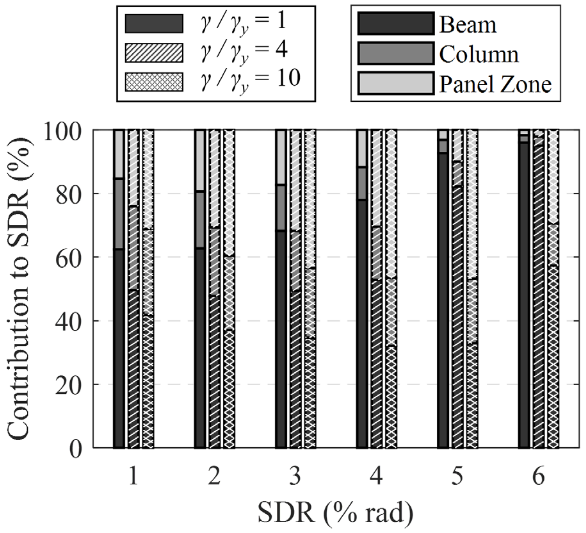


Figure 5 Contribution of the beams, the column, and the panel zone to the total story drift ratio

3.3 Geometric tolerance indicators

To quantify the deformation of beam members, a number of geometric tolerance indicators are examined, as illustrated in Figure 6. These indicators are defined in current standards [12]–[15] and include flange out-of-square *T* and flange folding *F*, which represent the out-of-plane deformation of beam flanges, as well as concavity of web *W*, which denotes the maximum bending deformation of the beam web. Additionally, beam axial shortening, *S* is utilized to represent the decrease in length of the central line of the beam. The geometric tolerance indicators for the subassembly model are calculated within a 1000mm distance from the beam end, i.e., the anticipated dissipative zone. A total of 21 cross sections are selected at intervals of 50mm, with *T*, *F*, and *W* determined as the maximum value among all sections.

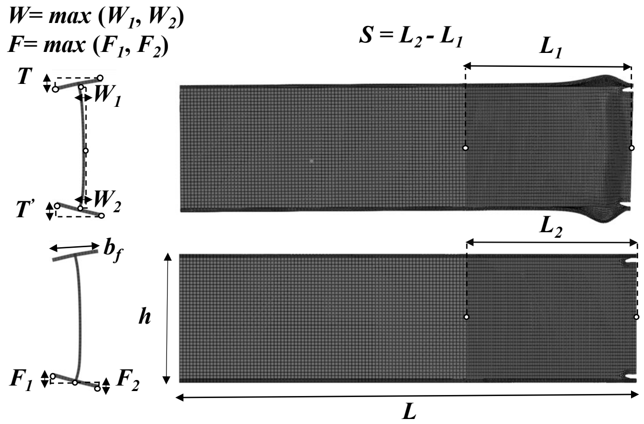


Figure 6 Geometric tolerance indicators

Table 2 summarizes the geometric tolerances for I-shaped steel members in JIS G 3192 [12] and ASTM A6/A6M [14]. While JIS G 3192 specifies values for all four geometric tolerance indicators (*T*, *F*, *W*, and *S*), ASTM A6/A6M only specifies the tolerance values for *T* and *S*. Regarding the out-of-square of the flange (*T*), JIS G 3192 specifies a tolerance of $0.01b_f$ or $0.012b_f$ for each flange (no less than 1.5mm), while ASTM A6/A6M imposes a limit on the sum of both sides (*T+T'*). The limit is determined by the section parameter *h* or b_f, and can be either 6 or 8 mm. Moreover, JIS G 3192 defines the tolerance for flange folding *F* as the minimum value of $0.0075b_f$ and 1.5 mm. Additionally, the web concavity *W* is limited to 2 to 3 mm, depending on *h*. Regarding axial shortening *S*, JIS G 3192 and ASTM A6/A6M prohibit the use of a negative tolerance in a member's length, thus limiting *S* to zero, regardless of the member's length.

Table 2 Geometric tolerance for I-shape steel members

Geometric tolerance indicators	JIS G 3192 [12] (mm)		ASTM A6/A6M [14] (mm)	
Flange out-of-square *T*	h≤300	1.0% b_f min 1.5	h or b_f ≤ 310	6 (for *T+T'*)
	h>300	1.2% b_f min 1.5	h or b_f > 310	8 (for *T+T'*)
Flange folding *F*	0.75% b_f Maximum 1.5mm		/	
Concavity of web *W*	h≤350	2.0	/	
	350≤h<550	2.5		
	h>550	3.0		
Axial shortening in length *S*	0		0	

Note: *h* and b_f represent the height and width of the undeformed I-shape cross-section. *L* refers to the length of steel member.

3.4 Deformation and geometric tolerance

Figure 7 illustrates the characteristic geometric tolerance indicators, i.e., flange out-of-square *T,* flange folding *F,* concavity of web *W* as well as beam axial shortening *S,* and the residual deformation as functions of the SDR. Due to brevity, the deformation of the left beam in the opposite direction of the initial loading direction is displayed. The term "max" in the legend denotes the maximum deformation during the loading cycle, while "residual" refers to the deformation observed when the column tip load was unloaded to zero.

Figure 7(a) and (b) depict *T* and *F* of the beam. The horizontal dashed lines in Figure 7 represent the geometric tolerance defined by JIS G 3192 [12] and ASTM A6/A6M [14]. Notably, there are no limitations on *F* and *W* in ASTM A6/A6M [14]. The simulation results indicate that the limitations imposed by JIS G 3192 are stricter than those of ASTM A6/A6M. Moreover, when γ/γ_y=10, the flange deformation is considerably lower than that observed in the subassemblies with a panel zone demand of γ_y and $4\gamma_y$.

Figure 7(c) and (d) demonstrate a similar trend in which connection designs featuring highly inelastic panel zones delay beam local buckling compared to their strong panel zone counterparts, resulting in acceptable residual *W* and *S* at an SDR demand of 5%.

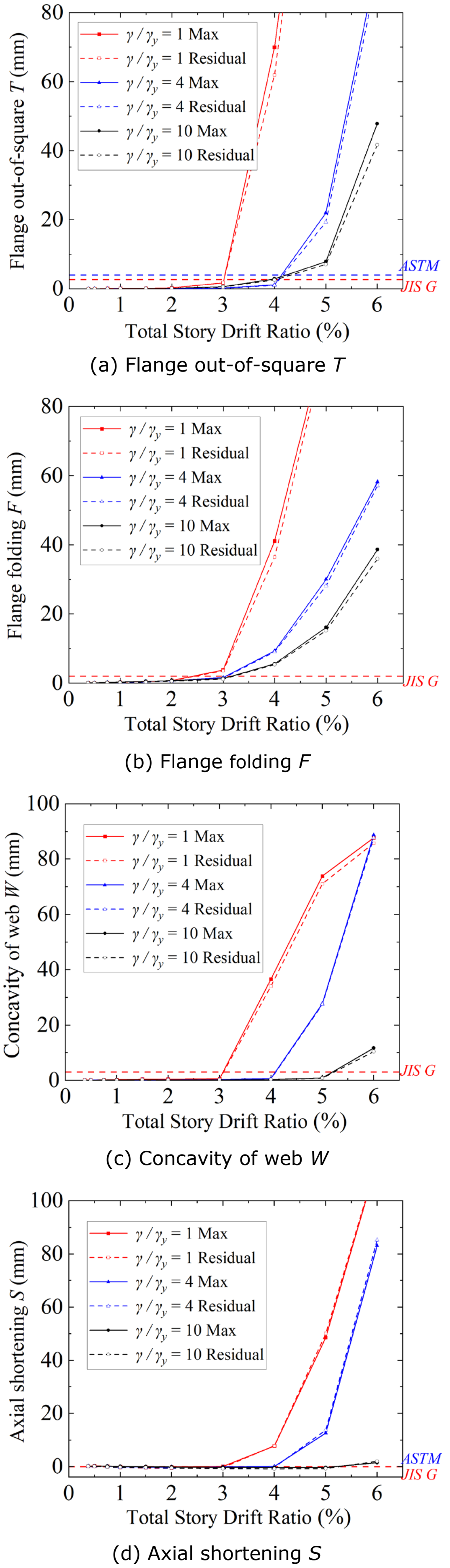


Figure 7 Beam deformation and geometric tolerance

In summary, taking into account the geometric tolerance specified in ASTM A6/A6M, beams in connections with a panel zone demand of $10\gamma_y$ may be reclaimed for SDR demands of up to 4%. Conversely, in welded connections with elastic panel zones, the steel beams exhibit inelastic local buckling early on in the loading history, thereby prohibiting the potential for re-use in the aftermath of earthquakes.

4 Summary and observations

The present paper assesses the impact of panel zone design on the residual deformations and geometric tolerances of steel beams in welded connections through detailed continuum finite element analyses. Three connections were assessed with identical beams and columns, whereas their panel zone was designed to either remain elastic or to exhibit inelastic shear distortions of 4 and 10 γ_y. The comparison was done by employing a symmetric cyclic loading protocol. Therefore some of the findings are loading-history dependent. The following observations hold true:

1. Connections with highly inelastic panel zones ($\gamma/\gamma_y = 10$) demonstrate superior performance in terms of seismic stability and energy dissipation capacity due to the delay of beam local buckling. Conversely, connections with a strong or balanced panel zone experience story shear deterioration early on in the loading history due to the concentration of inelastic deformations in the steel beams.

2. Geometric tolerances as per international standards can be utilized as a metric to assess the reusability of steel beams when undergoing inelastic cyclic straining. Residual deformations of steel beams in welded connections with highly inelastic panel zones are considerably less pronounced compared to those with elastic panel zones. The above possess advantages in terms of post-earthquake damage assessment and the potential for reusing structural steel members in the aftermath of earthquakes.

3. The geometric tolerance limitations imposed by JIS G 3192 are stricter than those of ASTM A6/A6M. Steel beams in connections featuring highly inelastic panel zones ($10\gamma_y$) can be considered reusable up until an SDR of 4% according to ASTM A6/A6M. Conversely, steel beams in connections with strong panel zone designs cannot be reused under similar loading conditions.

The authors acknowledge that the plasticity of the steel material is another aspect to be considered for potential reuse of steel members. However, this is outside the scope of the present study.

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