Towards Instability-Free Welded Moment Connections

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Abstract In capacity-designed steel moment resisting frames (MRFs), inelastic deformations are mostly concentrated in the steel beams and the column fixed ends of the first story, while the participation of the panel zones in energy dissipation is limited. In such a design context, flexural strength degradation due to local instabilities forming in the steel beam ends is likely at modest lateral drift demands. Consequently, structural repairs are expected in the aftermath of low probability of occurrence earthquake events. One of the primary reasons for this design principle is the increased fracture potential of beam-to-column connections during the 1994 Northridge earthquake. Recent experimental findings on welded moment connections that are compliant with the current welding specifications and quality control indicate a stable hysteretic response up until 5-6% lateral drift demands and limited damage on the beams when panel zones are designed to achieve shear distortions higher than $10\gamma_{\nu}$ (where γ_{ν} is the panel zone shear distortion at yield). This paper contrasts the finite element simulation results of welded beam-to-column connections that are designed with elastic and highly inelastic panel zones. The simulation results reveal that by allowing for inelastic deformations within the panel zone, the beam-to-column connection achieves an instability-free inelastic response up until high lateral drift demands.

Keywords Steel moment-resisting frames, Inelastic panel zones; Welded beamto-column connections; Continuum finite element modeling.

1 Introduction

Steel moment-resisting frames (MRFs) are commonly used structural systems for seismic applications. In steel MRFs, increased ductility demands can be sustained if rigorous connection detailing is employed along with the application of capacity design principles. In capacity-designed steel MRFs, the dissipative zones are mostly

concentrated in the steel beam ends. The current design practice limits the participation of the beam-to-column web panel zones in the energy dissipation. This design concept is motivated by findings from the 1994 Northridge earthquake. On the resistance side, the welding toughness specifications and practice were inadequate, and the quality control was poor at that time [1]. On the demand side, the notion of promoting inelastic distortions within the panel zones was attributable to the increased strain demands in the beam-to-column connection welds due to panel zone kinking [1].

The development of the current prequalified welded beam-to-column connections in the US [2], namely welded unreinforced flange-welded web (WUF-W) connections, was based on both experimental research [3–8] as well as corroborating finite element analyses [9–12]. Figure 1 illustrates the typical WUF-W connection detailing. Compared to the typical welded moment connection prior to the 1994 Northridge earthquake, this connection features: (a) improved fracture toughness requirements for the weld metal of the complete joint penetration (CJP) welds, (b) an optimized access hole geometry that minimizes the stress concentration nearby that region, (c) removal of the bottom beam flange backing bar after the completion of the CJP weld, and (d) utilization of doubler plates in most of the beam-to-column connection design cases to ensure limited deformations within the panel zone due to the high shear demand in this region.



Fig. 1 Schematic of a typical welded unreinforced flange-welded web connection detailing

Experimental evidence [3–8] on WUF-W connections designed according to the current AISC provisions [2,13,14] highlights that this connection typology generally meets the prequalification criteria [13]. With regards to the anticipated seismic performance, the elastic panel zone design concept triggers local buckling in the steel beam ends at modest lateral drift demands (i.e., 2% rads, on average). This has been highlighted in prior work [15,16]. Local buckling triggers cyclic deterioration of flexural strength of steel beams, thereby increasing the earthquake-induced collapse risk of steel MRFs [17]. Moreover, once local instabilities occur at the steel beam ends, the adjacent column is subjected to increased twisting

demands [18,19]. Subsequently its lateral load resistance is compromised [20,21]. From a repairability point of view, beam local buckling is challenging to address after an earthquake due to the required repairs and functionality recovery. Hwang and Lignos [22] showed that, in the advent of design-basis events, the structural repairs due to flexural yielding and/or local buckling at the beam ends are in the order of 20% of the total building replacement cost. Therefore, to limit functional recovery, research has focused on limiting damage in structural components [23].

Experimental research in the early 1970s highlighted the beneficial aspects of panel zone shear yielding in providing a stable beam-to-column connection hysteretic response [24–27]. An example of a stable connection hysteretic response up to 6% rad lateral drift demands when panel zones experience appreciable shear yielding is depicted in Fig. 2a. However, it should be noted that this test featured small-scale cross sections that are associated with decreased strain demands in the CJP welds at the beam flange-to-column flange joint. In larger-scale connections with inelastic panel zones, the fracture potential due to kinking (see Fig. 2b) may compromise the overall connection performance in steel MRF buildings during the 1994 Northridge earthquake [28,29].



Fig. 2 (a) Beam-to-column connection test results with inelastic panel zone [30]; and (b) kinematics of the panel zone joint [image adopted from [30]]

Motivated by the above discussion, this paper revisits the current state of welded beam-to-column connection design. First, available experimental data on beam-to-column connections designed with the post-Northridge design provisions but with inelastic panel zones are reviewed thoroughly. Second, continuum finite element simulations on welded moment connections are conducted. It is concluded that by allowing for $10\gamma_y$ shear distortions within the panel zone, an instability-free response is achieved in the beam-to-column connection with a minimal fracture potential.

2 Review of Beam-to-Column Connection Experimental Data after the 1994 Northridge Earthquake

Experimental research over the last years and after the 1994 Northridge earthquake [8,31–33] focused on the influence of inelastic pane zone design on the overall ductility of welded moment connections [2,13,14]. These studies, that are collectively described in [34], feature panel zones that attained peak shear distortions, γ_{max} , ranging from $2\gamma_y$ to $30\gamma_y$, as depicted in the histogram of Fig. 3a. The experimental results of these studies demonstrate that connections with panel zone design shear distortions up to at least $10\gamma_y$, meet the beam-to-column connection prequalification criteria reaching 5-6% lateral drift demands without significant strength and stiffness deterioration. This connection performance can be directly contrasted with the current design paradigm, where beams are expected to experience local buckling, therefore leading to degradation of the connection moment carrying capacity.



Fig. 3 Statistical analyses of post-Northridge beam-to-column connection experimental data: (a) Histogram of peak normalized panel zone shear distortions; and (b) probability of connection fracture given panel zone shear distortions and story drift ratio

To give a quantitative sense on the above, the joint cumulative distribution function of the connection fracture probability given γ/γ_y and story drift ratio (SDR) was calculated based on the available experimental data, as shown in Fig. 3b. For a maximum considered earthquake event (i.e., 2475-year return period), where the expected lateral drift demands in steel MRFs range from 3 to 4% rad, the probability of connection fracture does not practically change for panel zone design shear distortions ranging from $4\gamma_y$ to $10\gamma_y$. Therefore, given the beneficial aspects of highly inelastic panel zone designs, targeted panel zone shear distortions of $10\gamma_y$ could be an interesting alternative to the current design paradigm. Moreover, a recent system-level study on steel MRF buildings demonstrated a superior performance in terms of collapse risk and structural repairs when the panel zones are designed to attain $10\gamma_y$ [35].

3 Instability-Free Beam-to-Column Connections through Finite Element Analyses

To demonstrate the beneficial aspect of panel zone shear yielding in a beam-tocolumn connection response, an elastic and highly inelastic panel zone design approach are compared through continuum finite element (CFE) modeling. An interior beam-loaded subassembly is analyzed for this purpose, with the boundary conditions illustrated in Fig. 4. The utilized finite element software is ABAQUS v6.19 [36]. The interior subassembly features a stocky column with a 498x432x45x70 cross section and deep beams with a 650x300x16x25 one, which are equivalent to W14x398 and W24x131 cross sections, respectively, in North America. To satisfy an elastic panel zone response, a 45 mm thick doubler plate is utilized in the former model ($\gamma_d = \gamma_y$), contrary to the latter one where doubler plates are not imperative by targeting $\gamma_d = 15\gamma_y$ in the panel zone. The nominal yield stress of the beams and the column is 325 MPa.



Fig. 4 Schematic of the continuum finite element model and modeling assumptions

With regards to the finite element model, 8-node linear brick elements with reduced integration (C3D8R) are utilized as concluded from a mesh sensitivity analysis. For the beams and the column, two and four elements per web thickness and three and five elements per flange thickness are employed. Local imperfections in the steel beams are assumed as per [37] and residual stresses according to [38]. The multiaxial constitutive law proposed by [39] is employed to capture the steel material nonlinearity. The CFE model was thoroughly validated, but it is not discussed in this paper due to brevity. More details on the CFE assumptions and validation can be found in [34,40].

The simulation results are depicted in Fig. 5 for both the global subassembly, as well as the panel zone responses. Referring to the elastic panel zone design, beam local buckling is evident at 3-4% rad lateral drift demands. Therefore, at 6% rad, the connection loses more than 30% of its load carrying capacity. Contrary to that, the beam-to-column connection with the highly inelastic panel zone achieves an

instability-free response until 6% rad. It should be noted that the maximum story shear resistance in both cases is almost matching. This stable hysteretic response is attributable to the panel zone shear yielding mechanism (see Fig. 5b). The above results demonstrate the beneficial aspect of inelastic panel zone shear distortions from a structural repairability standpoint. This is also an important consideration from a collapse safety standpoint, especially for mainshock/aftershock earthquake events.



Fig. 5 Response comparisons of elastic and highly inelastic panel zone design approaches: (a) global response; and (b) panel zone response

4 Summary and Conclusions

This paper revisits the current seismic provisions for prequalified beam-to-column connections in steel MRFs. Experimental evidence on welded beam-to-column connections with panel zones attaining shear distortions up to $10\gamma_y$ demonstrates that fracture is not a concern for post-Northridge welded moment connections for lateral drift demands up to 5% rad. By allowing the panel zone to participate in the energy dissipation during an earthquake, the respective welded connections achieve an instability-free hysteretic response up until lateral drift demands that exceed those expected in steel MRFs subject to a maximum considered earthquake (i.e., 2% probability of exceedance over 50 years). Contrary to this design concept, connection designs that follow the current seismic provisions experience beam end local buckling, which is likely to increase the time for functionality recovery of the respective steel building in the aftermath of earthquakes. The authors further explore the above concepts through full-scale experiments, which are currently underway.

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