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COMPOSITE BEAM EFFECTS AND IMPLICATIONS TO SEISMIC DESIGN PROVISIONS

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ABSTRACT

Previous studies have investigated the slab effects on the cyclic behavior of composite beams in moment-resisting frames. Of interest in this paper, is the slab contribution to the flexural strength and effective stiffness of the beam. The former influences the strong column/weak beam ratio while the latter affects the fulfillment of serviceability limits. For this purpose, experimental data of composite beams from prior subassembly and frame system tests are assembled in a consistent database format. This database facilitates the evaluation of code-based approaches that are adopted in the seismic design of composite beams in steel moment-resisting frames. In particular, the developed database is used to assess the plastic flexural strength and effective stiffness of the beams according to the current Eurocode, US and Japanese seismic provisions. The database is also used to quantify the plastic rotation capacity of composite beams that is particularly important for nonlinear seismic assessment. In general, all three design codes estimate the sagging plastic moment resistance of the beams reasonably well. On the other hand, the estimated effective stiffness shows greater variability. The proposed composite beam effective stiffness according to the Japanese seismic provisions is found to be the most accurate between the three design provisions. Results also suggest that the plastic deformation capacity of composite beams is higher under sagging bending than hogging bending due to the slab restraint. Although inconclusive, the framing action seems to enhance the plastic deformation capacity of beams intersecting interior joints.

Keywords: Steel moment resisting frames; Plastic moment resistance; Composite action; Plastic deformation capacity; Seismic assessment

1. INTRODUCTION

Steel moment-resisting frames (MRFs) are commonly used in highly seismic regions to provide lateral load resistance. In an attempt to mitigate the earthquake-induced collapse risk of steel MRFs, seismic design codes have adopted capacity design principles that permit structural damage in controlled dissipative fuses. The strong-column-weak-beam (SCWB) ratio imposed in seismic provisions aims at preventing the development of plastic hinges in the columns, and hence the potential collapse due to soft story mechanisms. There is a considerable variability when it comes to the SCWB ratio, with European, US and Japanese provisions adopting values of 1.3, 1.1 and 1.5 respectively (AIJ 2010; AISC 2016a; CEN 2004a; b). All three codes ignore the composite action in the formulation of the SCWB ratio. Additionally, in order to fulfill story drift limits for MRFs, design codes impose serviceability requirements. These include both lateral drift and beam deflection checks.

Several tests have been conducted in the past 30 years to assess the effect of composite action on steel MRFs subjected to seismic loading. Uang et al. (2000) tested deep 912 mm beams with reduced beam sections (RBS) and showed that the slab increases the beam flexural strength under sagging bending by around 10%. Subassembly tests conducted by Jones et al. (2002) showed that the slab amplifies the beam flexural strength by up to 17% under sagging bending. Civjan et al. (2001) tested shallower beams

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with a depth of 753 mm. Composite action was shown to increase the beam flexural strength under sagging bending by 10%-30%. Elkady and Lignos (2014) assessed composite beams with depths varying between 533 mm and 911 mm that are commonly used in the North American steel construction practice. The slab was found to increase the beam sagging flexural strength by up to 35%, on average. This increase in beam flexural strength due to composite action is typically not considered in the SCWB ratio, which might lead to a shift to the undesirable strong-beam-weak-column mechanism. Of course, this depends on the degree of composite action provided by the shear studs.

In Japan and Europe, on the other hand, steel MRF buildings are more redundant. Consequently, shallow beams (400 mm to 600 mm) are more common even for tall buildings (Mele 2002; Nakashima et al. 2000). For shallow beams (<500 mm), the effect of composite action on the sagging flexural strength is more pronounced. A full-frame test conducted by Nakashima et al. (2007) showed that the flexural strength amplification can reach up to 50% for 400 mm beams when compared to non-composite steel beams. Similar findings hold true based on subassembly tests conducted by (Bursi et al. 2009). This issue deserves for attention for capacity design considerations.

The slab was also shown to substantially increase the beam effective stiffness as deduced from the moment-rotation relations. Nakashima et al. (2007) tests showed a 100% increase in the beam stiffness when compared to the non-composite beam. Similarly, Nam and Kasai (2012) showed that the composite beam stiffness is 2-3 times that of the non-composite beam. A side issue is the shear demand increase on the beam-to-column web panel zone (Leon et al. 1998).

Despite the mentioned detrimental effects of composite action, the slab provides lateral restrain on the top flange of the beam, which increases the plastic deformation capacity of the beam itself. The assessment of the plastic deformation capacity of composite beams is vital for modeling the component behavior under cyclic loading. Elkady and Lignos (2014) showed that the pre- and post-capping plastic rotations in composite beams with RBS sections increase by 80% and 35% respectively compared to those of the non-composite steel beam. However, the studied beams were primarily deep (>500 mm) and therefore do not cover a wide range of beam sizes that are more common in other seismic regions.

In this paper, a composite beam database is assembled. The database is used to (i) assess the flexural strength, (ii) the effective stiffness (iii) and the plastic deformation capacity of composite beams in fully restrained beam-to-column connections. The first two are vital for collapse and serviceability requirements, respectively. The third one is associated with nonlinear assessment of steel MRFs. Three design provisions are used in the assessment of the flexural strength and stiffness of the composite beams: the American specifications for structural steel buildings and their seismic provisions, Eurocodes 4 and 8 for composite and seismic design, and the Japanese recommendations for limit state design of steel structures. The current assessment provisions do not include specific provisions for the plastic deformation capacity of composite beams. Additionally, most of the available nonlinear modeling recommendations (e.g. ASCE/SEI 41-13) refer to non-composite steel beams (ASCE 2014).

2. DESCRIPTION OF COMPOSITE BEAMS DATABASE

The assembled database includes 39 beams with fully restrained beam-to-column connections (Bursi et al. 2009; Chen and Chao 2001; Cheng et al. 2007; Cheng and Chen 2005; Del Carpio et al. 2014; Engelhardt et al. 2000; Kim et al. 2004; Nakashima et al. 2006, 2007; Ricles et al. 2002; Yu and Uang 2000; Zhang et al. 2004). Connection types include reduced beam section (RBS), welded unreinforced flange bolted web connections (WUF-B), Japanese welded unreinforced flange welded web connections, and steel beam to concrete column connections. The database is composed of five beams that are part of full-frame (system-level) tests. The rest of the beams are part of subassembly tests. Configurations include beams framing both interior and exterior joints, beams with symmetric and asymmetric slabs (i.e., beams in perimeter steel MRFs). Most specimens have a steel deck with ribs oriented parallel to the beam. Two specimens have a steel deck with ribs oriented perpendicular and one specimen has a solid slab with no deck.

Steel beam depths vary between 353 mm and 912 mm to account for American, Japanese and European sizes. Shear span-to-depth ratios, L_o/h, vary between 3.5 to 6.3. Additionally, steel grades of the beams include American steel A572 and A992 Gr.50 and A36 steels, Japanese SN400B steel, and European S355 steel. The measured compressive strength of the concrete slabs varied between 20 to 40 MPa.

3. DEFINITIONS

The hysteretic moment-rotation relation of the composite steel beam is used to obtain a number of key parameters associated with composite effects. Shown in Figure 1 is the plastic moment resistance, $M^{\pm}_{pl}$, and plastic deformation capacity, $\theta^{+}_{p,80\%}$, of the composite beam under sagging bending. The plastic deformation capacity is defined as the plastic rotation corresponding to an ultimate flexural strength drop of 20% (i.e., $80\% M^{+}_{u}$). Values under hogging bending were obtained in a similar fashion. Also shown in Figure 1 is the effective stiffness under hogging bending derived from the unloading stiffness of the first inelastic cycle. Because the stiffness deduced from the moment-rotation relations considers both flexural and shear deformations, the term effective stiffness is adopted hereafter.

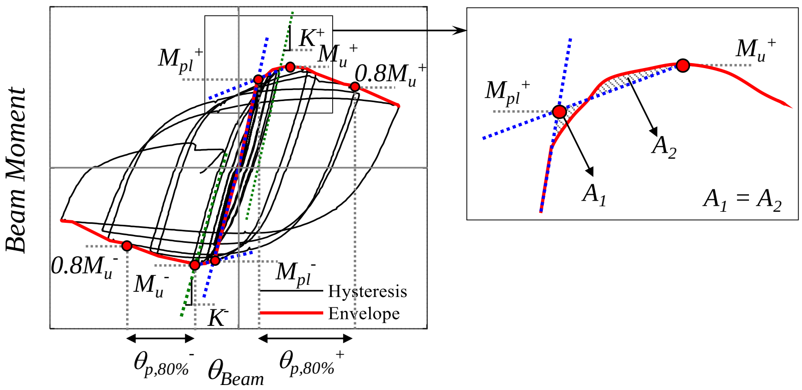


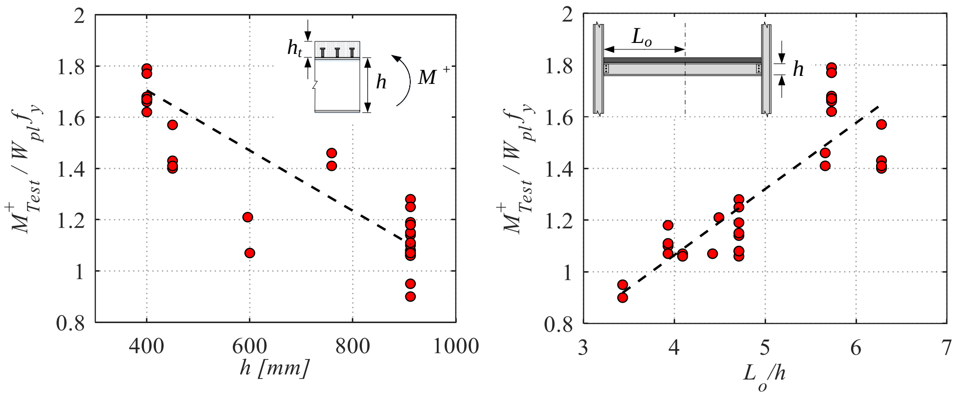
Figure 1. Derivation of the plastic moment resistance, effective stiffness and plastic deformation capacity from the hysteretic envelope

4. PLASTIC MOMENT RESISTANCE OF COMPOSITE BEAMS

4.1 Sagging Plastic Moment Resistance

The ratio of the plastic moment resistance obtained from the tests to the non-composite beam plastic moment resistance, $M^{+}/W_{pl}f_y$, was calculated for each specimen. For sagging (positive) moment resistance, this ratio depends on the beam depth, h, the span-to-depth ratio, L_o/h and the degree of composite action, η. On the other hand, for hogging (negative) moment resistance, this ratio depends on the slab reinforcement and L_o/h.

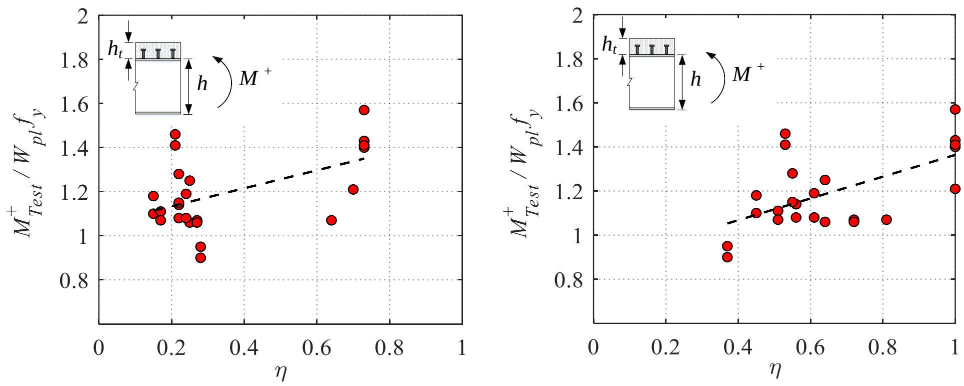
Figure 2a shows an increasing ratio with decreasing beam depth. Shallow beams of 350 mm depth attained around 80% higher flexural strength than the corresponding non-composite beams due to the presence of the slab. Referring to Figure 2b, composite beams with low span-to-depth, L_o/h, ratios attained lower resistances due to their larger depths and higher moment-shear interaction. In fact, for a span-to-depth ratio of 3.4, the attained ratio, $M^{+}/W_{pl}f_y$, is less than 1 (0.9-0.95). None of the specimens abide by the span-to-depth ratio specified by ANSI/AISC 358-16 for prequalified connections. The specified limit for steel MRFs is 7.0 or 9.0 depending on the type of the fully-restrained beam-to-column connection (AISC 2016b).



(a) (b)

Figure 2. Dependence of attained sagging moment resistance on (a) beam depth; and (b) span-to-depth ratio

The majority of the assembled beams are partially composite. Eurocode 8 recommends a value of 0.8 for the minimum degree of composite action (CEN 2004b). However, the available locations on deck module commonly falls short of required number of headed studs to achieve full interaction. Therefore, it is important to consider the partial composite action in the codes assessment. Figure 3 shows an increasing trend for higher degrees of composite action based on the three code provisions assessed herein. Due to brevity, only the US and European provisions are shown. Note the variability in the calculated degree of composite action between the codes. This is due to the difference in the shear stud strength and the recommended effective width per code provision as discussed later on.



(a) AISC provisions (b) Eurocode provisions

Figure 3. Dependence of attained sagging moment resistance on degree of composite action

The sagging moment resistance of the composite beams is then calculated based on the measured material properties. Therefore, Eurocode material partial factors γ, as well as the flexural strength reduction factor ϕ in US and Japanese provisions are not included in the formulas. Rigid plastic analysis is adopted in all three design codes to calculate the plastic moment resistance. The plastic moment resistance of composite beams is dependent on the effective width of the slab (for fully-composite beams) and the shear strength of the studs (for partially-composite beams).

European and US provisions consider the minimum of the stud and concrete resistance in the calculation of the stud shear strength (AISC 2016c; CEN 2004a). On the other hand, Japanese provisions only consider the concrete resistance while imposing an upper limit on its value (AIJ 2010).

All three codes impose limitations on the nominal stud height to shank diameter ratio, h_{sc}/d : 3.0, 4.0 and 5.0 for European, Japanese and US provisions respectively (AIJ 2010; AISC 2016c; CEN 2004a). While Eurocode 4 allowable h_{sc}/d ratio is less stringent, it imposes a reduction factor, α, for $h_{sc}/d <$ 4 (CEN 2004a). Additionally, ANSI/AISC 360-16 section D3 limits the stud diameter to 19 mm (AISC 2016a).

In case a steel deck is present, all provisions impose limitations on the deck dimensions. If the deck ribs are parallel to the beam, US and European provisions propose reduction factors on the stud shear strength, while Japanese provisions do not. If the ribs are transverse to the beam, all three provisions reduce the shear strength of the studs. The reduction factor depends on the average rib width to rib height ratio as well as the stud height to rib height ratio. The reduction factor also accounts for the number of studs per rib if the steel deck is transverse to the beam (AIJ 2010; AISC 2016c; CEN 2004a). European provisions are particularly stringent when the steel deck ribs are transverse to the beam. Eurocode 4 limits the allowable number of studs per rib to two while US and Japanese provisions allow up to three studs per rib. Moreover, Eurocode 4 clause 6.6.4.2 imposes an upper limit on the strength reduction factor, $k_{t,max}$, and Eurocode 8 clause 7.6.2 introduces an additional rib shape efficiency factor k_r (CEN 2004a; b). As per section D3 of the US seismic provisions for structural streel buildings, ANSI/AISC 341-16, an additional 25% reduction is applied on the shear strength for studs that are part of seismic force resisting systems (AISC 2016a). Eurocode 8 requires a 25% reduction only for studs in the dissipative zone (CEN 2004b). For this reason, US provisions provide the most conservative stud shear strength estimate of the three design codes while Japanese provisions provide the least conservative estimate. European provisions provide the lowest estimate of the stud shear strengths for composite beams with transverse deck ribs. Note that only two of the assessed beams have a steel deck with transverse ribs. Therefore, the code assessment presented hereafter is for composite beams with parallel deck ribs.

The effective width of the slab is compared for the three design codes. European provisions distinguish between effective widths under gravity and seismic loading, while US provisions do not (AISC 2016c; CEN 2004b; a). For beams framing exterior columns, Eurocode 8 proposes three different values for the effective width. The effective width in the US and European provisions depends on the beam span (AISC 2016c). On the other hand, the effective width in the Japanese provisions is equal to the width of the column flange (AIJ 2010). Pilot studies by Du Plessis and Daniels (1972) showed that the effective concrete slab width is equal to the width of the column face. Furthermore, it is important to note that for deck ribs oriented parallel to the beam, US provisions permit the inclusion of the concrete below the top of the deck for determining the composite beam resistance and properties (AISC 2016c).

Eurocode 4 and ANSI/AISC 360-16 assume a slab compressive stress of $0.85f'_c$ where f'_c is the concrete design compressive strength. Japanese recommendations for limit state design assume a slab compressive stress of $2f'_c$. The compressive stress acting along the width of the column face was found to be $1.3f'_c$ by Du Plessis and Daniels (1972), and $1.3f'_c$ by Tagawa et al. (1989) . The tested specimens were fully composite as per AISC 360-16 (AISC, 2016c).

The ratio of the attained to calculated sagging plastic moment resistance of the composite beam was plotted against h, L_o/h and η for all three design codes. Figure 4a shows that the moment resistance of beams with larger depths was overestimated in all three codes. These beams tend to have a lower L_o/h and therefore a reduced moment resistance due to moment-shear interaction as evident in Figure 4.

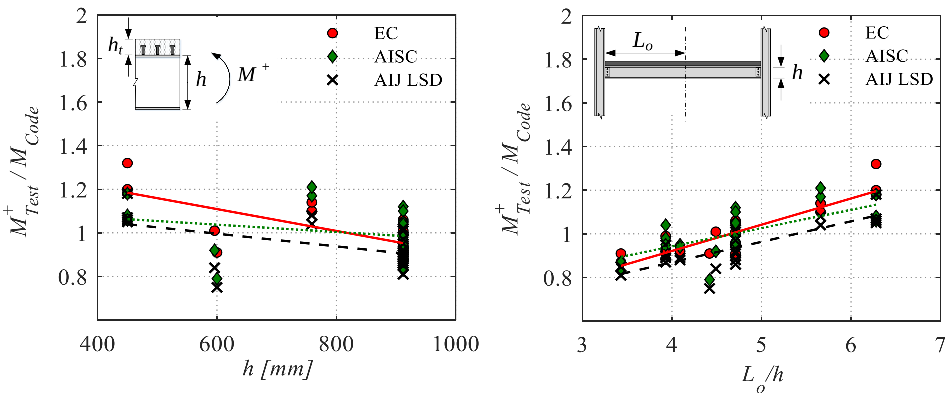


Figure 4. Comparison of attained to code-based sagging moment resistance with respect to (a) beam depth; and (b) span-to-depth ratio

Figure 5 shows the attained moment resistance for all beams as well as beams with lower moment-shear interaction (L_o/h > 5). Referring to Figure 5b, the Eurocode (EC) underestimates the moment resistance for fully composite beams. The tests achieved a plastic moment resistance between 20% to 30% higher than predicted by the code. This shows that the effective width is on the conservative side and a larger effective width than predicted is actually mobilized. Although the effective width proposed by the Japanese limit state design recommendations (AIJ LSD) is generally smaller than the Eurocode effective width, the assumed compressive stress in the concrete is around 2.35 times higher than assumed in Eurocode 4 (AIJ 2010; CEN 2004a).

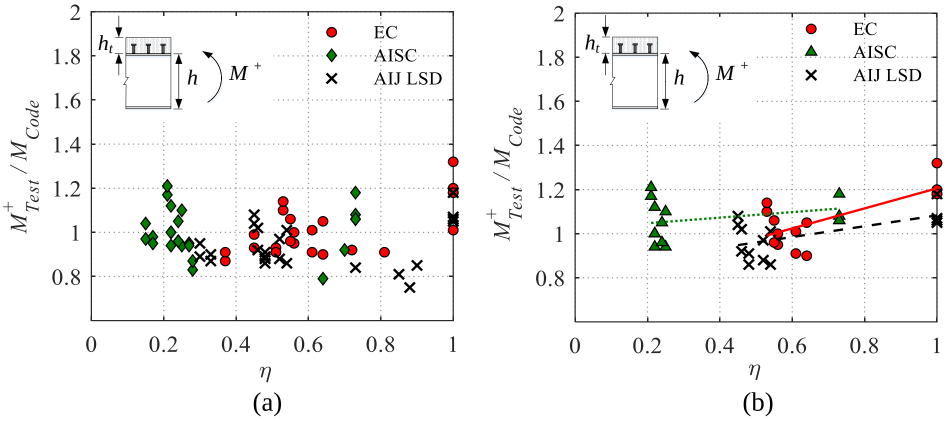
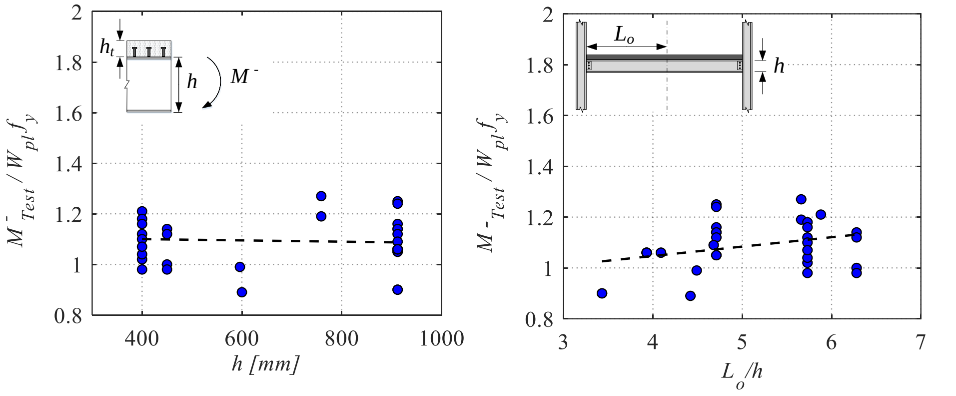


Figure 5. Comparison of attained to code-based sagging moment resistance with respect to degree of composite action; (a) all beams; (b) beams with L_o/h > 5

On average, all three codes estimate the sagging plastic moment resistance reasonably well particularly for beams with L_o/h > 5. The sagging plastic moment resistance is not very sensitive to the discrepancies in the values of the effective width and strength of headed studs. European provisions show the highest COV (0.13) due to the aforementioned reason. The US provisions are slightly more conservative than the Japanese ones for partially composite beams due to the lower stud shear strength estimation. As mentioned earlier, the moment resistance was calculated using the measured material properties for each test. In order to account for material overstrength in the design, Eurocode 8 adopts an overstrength factor, γ_{ov}, of 1.25 (CEN 2004b). AISC 341-16 adopt R_y factors defined as the ratio of the expected to the specified minimum yield stress (AISC 2016a). R_y factors vary depending on the material and structural shape. For this reason, larger variability is expected in the current Eurocode formulation if design material properties are used for the calculation of the plastic moment resistance. In both codes, the non-composite beam plastic moment is increased by 10% to account for cyclic strain hardening. However, this increase is not included in the assessment of the beam plastic moment resistance.

4.2 Hogging Plastic Moment Resistance

The hogging moment resistance is, on average, around 10% higher than the corresponding non-composite beam moment resistance regardless of the respective beam depth (see Figure 6a). Figure 6b shows that beams with low L_o/h ratios attained lower moment resistances due to moment-shear interaction. Note that all the beams were lightly reinforced with a wire mesh in order to control creep and shrinkage cracking. Thus, slab reinforcement does not significantly contribute to the hogging moment resistance of the beams.



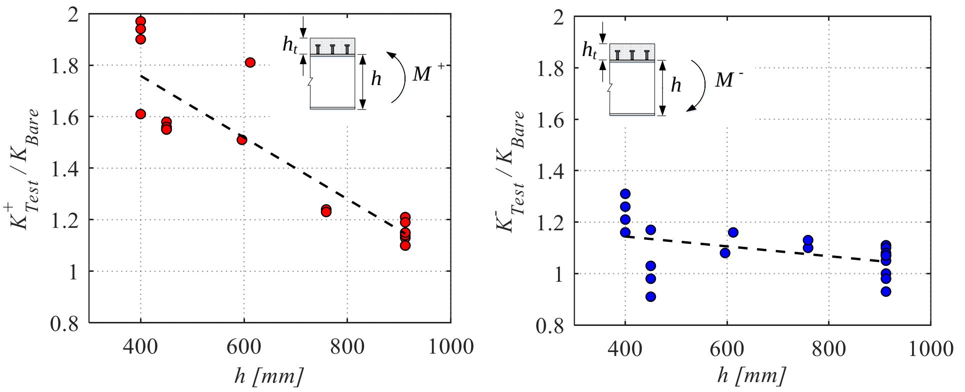
(a) (b)

Figure 6. Dependence of attained hogging moment resistance on (a): beam depth; and (b): span-to-depth ratio

5. EFFECTIVE STIFFNESS OF COMPOSITE BEAMS

A total of 21 beams were assessed for their effective stiffness. The ratio of the effective stiffness obtained from the tests to the non-composite beam stiffness was calculated for each specimen. The calculated stiffness of the non-composite beam consists of a flexural and a shear component, K_{es}. It is assumed that the beam web contributes to the shear component, K_{ef}, and the shear coefficient is taken as $(5+5\nu)/(6+5\nu)$, where ν is the Poisson's ratio, as discussed in Hutchinson (2000).

Figure 7a show that sagging elastic effective stiffness of the composite beams is higher than the corresponding non-composite beams. Composite action tends to increase the stiffness from 15% for deep beams of 900 mm depth to nearly 200% for shallow beam of 400 mm depth. This agrees with previous tests, which showed that the stiffness increased by around two times for 400 mm beams (Nakashima et al. 2007) and 2 to 3 times for 350 mm beams (Nam and Kasai 2012). The effective stiffness also depends on the degree of composite action as will be discussed later on. Referring to Figure 7b, hogging elastic rotational stiffness ratio shows no clear variation with respect to the beam depths. The stiffness is around 10% percent higher, on average, due to the slab reinforcement.



(a) sagging effective stiffness (b) hogging effective stiffness

Figure 7. Dependence of attained sagging and hogging effective stiffness on beam depth

According to Castro et al. (2007), different effective widths should be used for strength (plastic moment resistance) and serviceability (stiffness and deflection check) calculations. This is because shear lag effects occur in the elastic range while stress redistribution occurs in the inelastic range due to material nonlinearity. Hence, the effective width used for the calculation of the effective stiffness should be lower than that for the calculation of the plastic moment resistance. The US provisions propose the same effective width for plastic moment resistance and serviceability checks, while both European and Japanese provisions propose different values for the same purpose. In Eurocode 8, the effective width depends on the column configuration (exterior or interior) as well as the presence of transverse beams and anchored rebar in the slab (CEN 2004b).

Both Eurocode 8 and Japanese recommendations for limit state design (AIJ LSD) specify a fixed value for the modular ratio (*n* = 7 and *n* = 15, respectively) (AIJ 2010; CEN 2004b). It is worth noting that the modular ratio value adopted in the Japanese provisions is more than twice as high than that of the US and European provisions. Elastic transformed section analysis is adopted by the design codes to calculate the effective stiffness of the composite beam. The Eurocode approach has no formulation for obtaining the stiffness of partially composite beams. The US and Japanese provisions consider the degree of composite action in the calculation of composite beam stiffness through the following approximation (AIJ 2010; AISC 2016c):

$$I_{equiv} = I_{non-comp} + \sqrt{\eta} \times (I_{comp} - I_{non-comp}) \tag{1}$$

in which, I_{equiv}, I_{comp} and $I_{non-comp}$ are the moments of inertia of the partially composite, fully composite and non-composite beam, respectively.

Leon (1990) and Leon and Alsamsam (1993) recommended a 25% reduction in the composite beam stiffness for realistic deflection calculations. This recommendation is adopted in AISC 360-16 (AISC 2016c). Japanese limit state design recommendations account for this reduction by using a larger modular ratio, *n* = 15 (AIJ 2010).

The effective stiffness under sagging bending was calculated from the composite beam stiffness. Both the flexural and shear components of the stiffness were considered. It was assumed that the beam web contributes to the shear stiffness as mentioned earlier. The ratio of the attained to calculated effective stiffness under sagging bending was plotted against the beam depth and degree of composite action for all three design codes. Figure 8 shows that on average, the Japanese code (AIJ LSD) estimate of the composite beam stiffness is closest to the attained stiffness. The AISC estimate is around 20% lower than the Japanese one. While the 25% reduction in the stiffness is required for realistic deflection calculations, it seems that this reduction results in an overly conservative estimate of the effective stiffness of the beam for new construction. However, long term effects may come into play over the life cycle of a building. The Japanese recommendation of using a large modular ratio of 15 produces better estimates of the effective stiffness of composite steel beams. The collected tests were conducted soon after the 28th day of concrete casting; therefore, long term effects cannot be assessed herein.

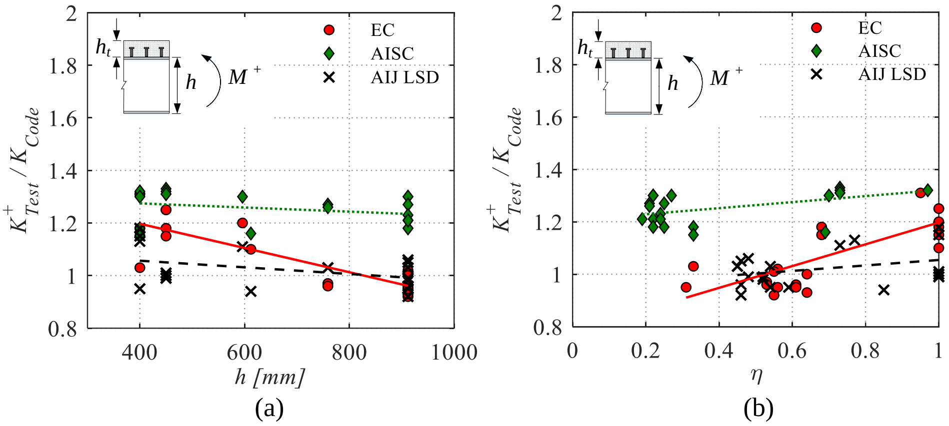


Figure 8. Comparison of attained to code-based sagging effective stiffnes with respect to (a) beam depth; (b) degree of composite action

Both AISC and AIJ LSD data points have a low COV (0.07). EC data points have a nearly double COV value (0.13). This is because the European provisions do not distinguish between full and partially composite beam for stiffness calculations. The Eurocode recommended effective width is low when compared to that of the US and Japanese codes. For this reason, the estimated effective stiffness of fully composite beams is around 17% lower than the attained one. The estimated effective stiffness of partially composite beams with η < 0.8 is around 8% higher than the attained. As previously mentioned, Eurocode 8 adopts a minimum value of η = 0.8. Hence, for deflection checks of composite beams with $\eta \geq 0.8$, the stiffness estimate is on the conservative side as in AISC 360-16. For the purpose of calculating the effective stiffness of partially composite beams with η < 0.8, Eurocode yields unconservative estimates.

6. PLASTIC DEFORMATION CAPACITY OF COMPOSITE BEAMS

The assembled database was used to obtain the plastic deformation capacity of the beams, $\theta^+_{p,80\%}$, under sagging and hogging moments. Thirty beams are part of subassembly specimens that were subjected to cyclic loading and the remaining five beams are part of full frame tests. Beams in which early fracture occurred were excluded. Moreover, some of the tests were terminated before a strength drop of 20% occurred. Lower bound values were provided for the corresponding data points.

According to Lignos and Krawinkler (2011), the web slenderness is the main contributor to the pre-capping plastic rotation especially in deep and slender beams, which are typical in the North American steel construction practice. This is because web local buckling is triggered first in most cases followed by flange local buckling. The $\theta^+_{p,80\%}$, was plotted against the beam web slenderness c/t_w where c is the fillet-to-fillet web height, and t_w is the web thickness.

Figure 9 shows that the plastic deformation capacity increases with decreasing web slenderness values. Note that Engelhardt et al. (2000) beams attained higher plastic deformation capacities than other beams with the same web slenderness. This is because in Engelhardt et al. (2000) the beam webs were heavily braced laterally to prevent the out-of-plane movement of the web. This explains the higher $\theta^+_{p,80\%}$ values attained in this case. The trend line shown in Figure 9 does not pass throught the superscripted data points. An increasing trend is shown on the figure with plastic deformation capacity values varying between 0.026 rad for c/t_w = 54.4, to 0.046 rad for c/t_w = 43.4. Web local buckling is triggered earlier in beams with higher web slenderness values, which explains their lower plastic deformation capacity. Also shown in Figure 9, are the plastic deformation capacities of the corresponding non-composite steel beams. The predicted values are based on the regression equations proposed by Hartloper and Lignos (2016) to obtain the first-cycle envelope of steel beams subjected to symmetric-cyclic loading histories. The positive plastic deformation capacity of the composite beams is 30%-60% higher than that of the corresponding non-composite beams. The slab in composite beams provides lateral restrain for the top flange and top portion of the web (Cordova and Deierlein 2005).

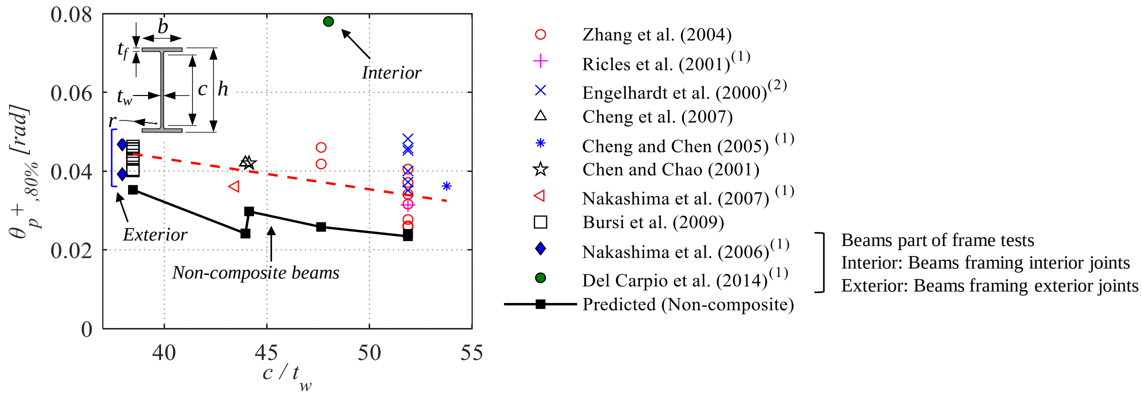


Figure 9. Dependence of positive plastic deformation capacity on beam web slenderness. (1): lower bound values; (2): heavily braced specimens

One of the data points achieved a significantly high "lower bound" plastic deformation capacity of 0.078 rad. This data point corresponds to an interior beam part of a frame test conducted by Del Carpio et al. (2014). This behavior was attributed by Cordova and Deierlein (2005) to the beam and slab continuity at interior joints, which restrains the joint rotation and reduces the rotation demand in the plastic hinge. In contrast to subassembly specimens, which allow axial shortening of the beam, frames provide restrain against axial shortening. Although inconclusive, this framing action increases the plastic deformation capacity of interior beams. The authors are currently investigating this issue.

Figure 10 shows a similar trend for $\theta^{-}_{p,80\%}$ as $\theta^{+}_{p,80\%}$ with values varying between 0.018 rad for c/t_w = 54.4, to 0.031 rad for c/t_w = 43.4. In general, $\theta^{+}_{p,80\%}$ values are around 40% - 50% higher than $\theta^{-}_{p,80\%}$. The predicted $\theta^{-}_{p,80\%}$ values for the non-composite beams are comparable to that of the beam under hogging bending. The maximum difference is around 15%. Under hogging bending, the slab does not provide restrain for the flange and portion of the web under compression. Also note that Del Carpio et al. (2014) interior beam attained a high lower bound plastic deformation capacity of 0.055 rad due to the framing action.

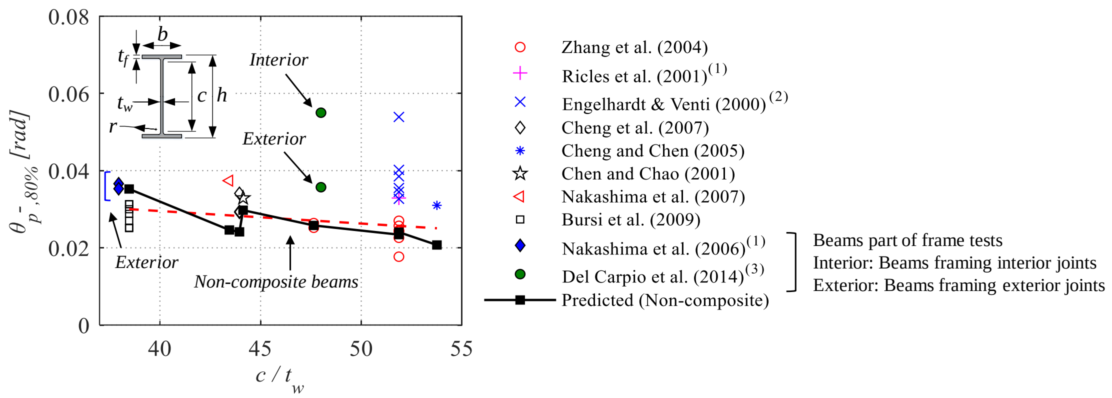


Figure 10. Dependence of positive plastic deformation capacity on beam web slenderness. (1): Lower bound values, (2): Heavily braced specimens, (3): Lower bound value for the interior joint beam only

7. CONCLUSIONS

This paper is concerned with the properties of composite beams in steel moment resisting frames designed in seismic regions. Of particular interest are the sagging moment resistance, effective stiffness and plastic deformation capacity. For this purpose, a database of 39 composite beams was assembled. The American, European and Japanese provisions for calculating the sagging moment resistance and effective stiffness were assessed through a comparison with the test data. The plastic deformation capacity of the composite beams was also assessed and compared with predicted values for non-composite steel beams. The main conclusions are listed below:

- *Sagging plastic moment resistance:* European provisions underestimate the moment resistance for fully composite beams. Tests achieved a plastic moment resistance between 20% to 30% higher than predicted by the Eurocode. US provisions are slightly more conservative than Japanese provisions for partially composite beams due to their lower estimation of the stud shear strength. All three codes estimate the sagging plastic moment resistance reasonably well particularly for beams with high span-to-depth ratios. The resistance is not very sensitive to the discrepancies in the values of the effective width and strength of headed studs.
- *Effective stiffness:* Japanese provisions provide the best estimate of the composite beam effective stiffness under sagging bending. The US provisions estimate is around 20% lower than the Japanese one. Eurocode estimates showed the largest discrepancy due to the lack of a formulation for estimating the stiffness of partially composite beams. The stiffness of fully composite beams, as per Eurocode, is around 17% lower than the attained in the collected tests.
- *Plastic deformation capacity:* Composite beams showed a decreasing trend for $\theta^{+}_{p,80\%}$ and $\theta^{-}_{p,80\%}$ for increasing web slenderness. In general, $\theta^{+}_{p,80\%}$ for composite beams values are around 30% - 60% higher than those corresponding to non-composite beams while $\theta^{-}_{p,80\%}$ are comparable to those of the non-composite steel beams. Framing action seems to increase the plastic deformation capacity of interior joint beams. However, this issue deserves more attention in future studies.

8. ACKNOWLEDGMENTS

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