ce/papers
The online collection for conference papers in civil engineering
Ernst & Sohn A Wiley Brand

ORIGINAL ARTICLE

eurosteel SHEFFIELD 2021
Ernst & Sohn A Wiley Brand

Hysteretic Behaviour of Shear Stud Connectors in Composite Steel Moment-Resisting Frames

Hammad El Jisr, Dimitrios G. Lignos[1]

Correspondence

Hammad El Jisr
École Polytechnique Fédérale de Lausanne
ENAC IIC RESSLab
Station 18
CH-1015 Lausanne
Email: hammad.eljisr@epfl.ch

Affiliations

[1]École Polytechnique Fédérale de Lausanne, Lausanne, Switzerland

Abstract

In steel moment resisting frames (MRFs) with composite floor slabs, seismic loads are transmitted from the slab into the beam through shear studs. Shear strength degradation of the studs due to cyclic loading results into the loss of the load transfer mechanism and composite action. To date, several low-cyclic, high-amplitude push-out tests have been conducted to characterize the shear stud hysteretic behaviour. These isolated tests showed considerable degradation in the shear stud resistance; nevertheless, they do not simulate the actual hysteretic behaviour of shear studs within a composite steel beam. To elaborate, conventional cyclic push-out tests do not replicate the actual stress state in the concrete slab, nor do they account for the force redistribution between the shear studs. This paper investigates the hysteretic behaviour of shear studs in steel MRFs with composite floor slabs. The load-slip hysteretic behaviour of the shear studs is examined through continuum finite element (CFE) models of 2-bay sub-systems and their corresponding cruciform subassemblies. Moreover, shear force and interlayer slip envelopes are obtained along the composite steel beams. Results of the CFE models are used to propose new design recommendations for shear studs in composite-steel MRFs.

Keywords

Shear studs, Shear strength degradation, Composite floors, Composite-steel moment resisting frames

1 Introduction

Steel moment resisting frames (MRFs) are widely used for seismic design of framed buildings. Often, a composite floor slab connected to the beams via shear studs, is present [1]. The floor slab acts as a rigid diaphragm and transmits seismic loads to the moment frame beams through friction and shearing of the studs.

Past studies have shown that the reliability of shear studs subjected to cyclic loading is contentious [2]. As a precaution against severe shear stud strength degradation, seismic provisions provide recommendations that diminish the consequences associated with the uncertainty in shear stud hysteretic behaviour. For instance, Eurocode 8-1 [3] limits the degree of composite action to a minimum of 80%. The rationale behind this lower limit is that shear studs in composite steel beams with low degree of composite action act as the weak link. As such, they may experience large inelastic deformations, and eventually fracture. Moreover, both Eurocode 8-1 [3] and ANSI/AISC 341-16 [4] propose a 25% reduction in the design shear resistance of the studs. This reduction is based on low-cyclic high-amplitude push-out tests conducted on headed shear studs [5–8]. In these tests, the shear studs experienced severe strength degradation compared to those subjected to monotonic loading. The loss of composite action due to stud failure leads to the loss of the load transfer mechanism between the slab and the steel beams [2]. As a result, extensive spalling may occur as seismic loads are transferred to the moment frame through bearing of the slab on the column flanges. In that regard, it may seem justifiable to reduce the ultimate shear strength of the studs under cyclic loading to ensure that they maintain their load carrying capacity at design level lateral drift demands (2%) and control the slab damage. Nevertheless, only a few subassembly tests with composite beams showed early fracture of the shear studs [6,9,10]. This was attributed to poor weld quality and/or additional out-of-plane forces acting on the studs. In more recent tests [11] of shallow composite beams (310mm depth and a reported degree of composite action of 48%) the studs remained intact and experienced little or no deformation. Similar findings were reported in [2]. It is understood that the shear stud hysteretic performance is better than anticipated particularly in shallow beams (depth less than 400mm).

Although informative, conventional cyclic push-out tests do not replicate the actual hysteretic behaviour of shear studs in composite steel beams. Particularly, the slab stress state alternates between tension and compression depending on the sign of a loading excursion (sagging or hogging). This is not the case in conventional cyclic push-out tests in which the concrete remains under compression regardless of the sign of the loading excursion. Recently conducted cyclic push-out tests [12] account for this limitation by applying fully-reversed cyclic stress to the shear studs and concrete slab. Another

limitation of cyclic push-out tests is that they do not take into consideration the force redistribution between the studs [12–14] nor the effect of the slip restraint near the beam-column joints [15]. From here stems the necessity of assessing the hysteretic behaviour of shear studs in composite steel beams through simulation to further comprehend their hysteretic behaviour.

Several modelling approaches have been proposed to model the behaviour of shear studs in composite steel beams. In [16] it was shown that the local load-slip behaviour of shear studs may be accurately captured by using 15-node quadratic prisms and 20-node brick elements for the shear studs, and 15-node quadratic prisms for the concrete surrounding the shear studs. Adopting these higher order elements to model the global cyclic behaviour of composite steel subassemblies and their members is not computationally efficient. Accordingly, researchers have attempted to increase the efficiency of the models through simplifications. [17] modelled the shear studs using linear Timoshenko beam elements with a modified cross-sectional area to represent the shear strength of the studs. [18] adopted a similar approach and added a spot weld failure criteria to simulate the stud-to-metal deck connection. [19] used pin jointed truss members with an effective stiffness to model shear studs. Elastic perfectly-plastic spring elements with modified properties were implemented by [20] and [21]. These modelling procedures replicate the monotonic behaviour of the shear studs fairly well. However, they cannot be extended to cyclic loading as they do not capture the pinched cyclic behaviour of the shear studs. Others [22–25] attempted to model the interface slip using a non-linear load-slip relation. While these models account for the cyclic degradation of the studs, they do not consider the stress state in the slab under fully reversed cyclic loading, nor the force redistribution between the studs.

In this paper, a continuum finite element model that accounts for the interaction between the composite slab and the beams is proposed and validated with experimental data. The modelling approach is then used to compare the hysteretic behaviour of shear studs in MRFs with beams of various depths. For this purpose, two-bay subsystems and subassemblies are modelled. The differences between the load-slip behaviour of studs in sub-systems, where axial restraint is present, and subassemblies are highlighted. Moreover, the shear stud force and interlayer slip envelopes along the beams are evaluated in the sub-systems. In light of the findings, aspects associated with the shear stud design in composite-steel MRFs in seismic provisions are discussed.

2 Continuum finite element model

2.1 Description

Modelling the slab-beam interaction in steel MRFs necessitates a comprehensive CFE model that is able to capture contact between the slab and the top beam flanges, as well as the non-linear cyclic behaviour of the shear studs. For this purpose, a high-fidelity model, is developed in Abaqus 6.14 [26]. The model, shown in Figure 1, is validated with data from a full-scale subassembly test [20]. Details of the model specifics, the element types and the material constitutive laws are found in [27]. The developed CFE model can be found in https://doi.org/10.5281/zenodo.3678262.

Synergy between the slab and the steel beam is accounted for through a hard contact interaction. Additionally, the contact interface is governed by Coulomb's friction model with a steel-to-concrete friction coefficient, μ = 0.2 [28]. Referring to Figure 1(b), the shear studs connecting the slab to the main steel beams are simulated through idealized non-linear zero-length springs.

Past cyclic push-out tests have shown that the behaviour of shear studs in composite steel beams is characterized by a degrading pinched hysteretic response. This response is captured using a non-linear load-slip behaviour. In particular, the modified Ibarra-Medina-Krawinkler (IMK) deterioration model with pinched hysteretic behaviour can explicitly simulate the effects of stiffness, strength, post-capping strength and accelerated reloading stiffness deterioration. Furthermore, the model has the benefit of being loading-history independent as it assumes a reference inherent energy dissipation capacity regardless of the loading protocol. The IMK model is implemented in the non-linear zero-length springs through a user-defined element (VUEL), publicly available from https://github.com/RESSLab-Team/IMK_Pinching_VUEL. The shear stud model parameters are obtained through calibration with recently conducted cyclic push-out tests [12]. Unlike conventional cyclic push-out tests available in literature [5,6,8], these tests did not demonstrate severe strength degradation in shear studs with 19mm diameter. They replicate the mechanical behaviour of the shear studs under fully reversed cyclic loading in that they account for the stress state in the slab under sagging and hogging excursions. Referring to Figure 2, the ultimate shear strength when the slab is under tension (i.e. hogging bending) is around 40% that when the slab under compression (i.e. sagging bending). Figure 2 shows the parameters obtained though the calibration of a cluster of four 19mm shear studs. Note that the ultimate shear strengths Q_u^+ and Q_u^- are adjusted based on the properties of the composite floor slab of SPEC3.

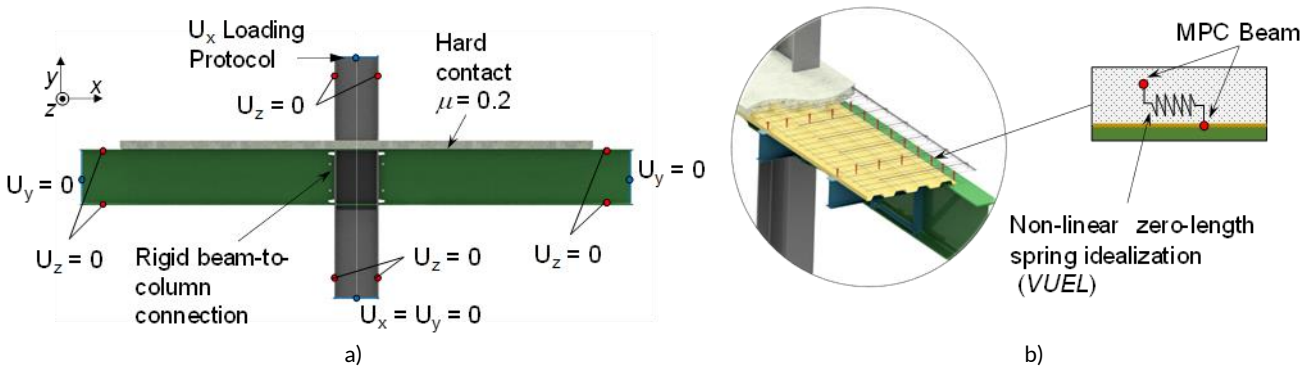


Figure 1 a) Continuum finite element model boundary conditions and beam-slab interaction; b) shear stud detail

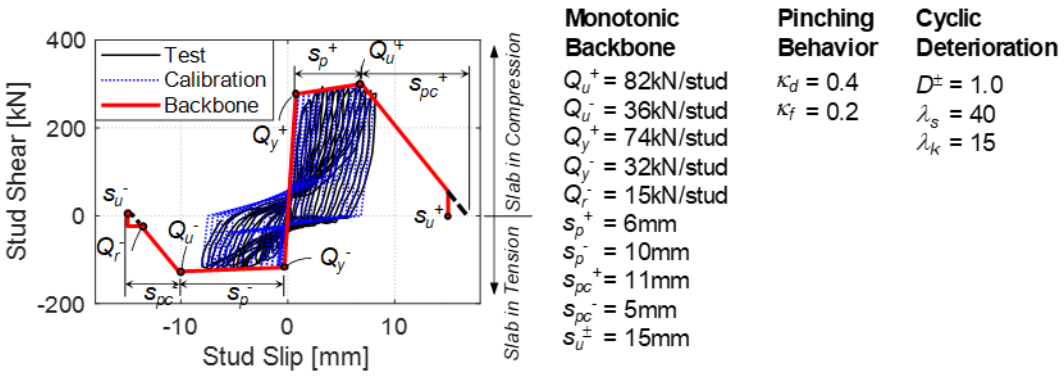


Figure 2 Calibration of a cluster of four 19mm shear studs [12]

2.2 Validation

The proposed model is validated with an available physical test [20] subjected to a cyclic symmetric loading history [29] at the column tip. This specimen was chosen since most of the energy dissipation occurred in the beams and lateral torsional buckling in the beams was prevented by sufficient lateral bracing. The specimen dimensions and composite floor slab details are found in [20]. An explicit dynamic procedure is implemented in the model. Static equilibrium and energy balance are used to verify that the behaviour is quasi-static. Details regarding this important matter can be found in [27].

Referring to Figure 3, the simulated hysteretic response of the beam (at the column face) shows a fairly good agreement with the test data. In particular, the CFE model predicts the peak flexural strength within a margin of error of 10%. Furthermore, flexural strength degradation due to local buckling and shear stud degradation is captured fairly well. Note that the simulated behaviour slightly deviates from the test at 6% SDR. This deviation is due to the occurrence of ductile tearing in the bottom flange of the beam during the test.

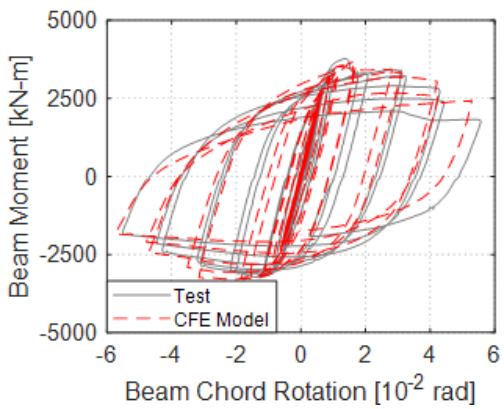


Figure 3 Comparison between the simulated and experimental beam hysteretic behaviour

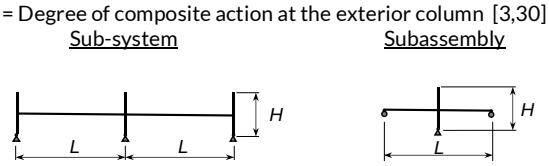
2.3 Parametric study

The validated CFE modelling approach is used to assess the shear stud hysteretic behaviour in composite-steel MRFs comprising deep and shallow beams. Two two-bay sub-systems with different beam depths are modelled along with the corresponding cruciform subassemblies. Table 1 shows the dimensions of the modelled sub-systems. The degree of composite action, defined according to EN 1994-1-1 [30] and EN 1998-1 [3], is the same at the interior column and differs only slightly at the exterior column. It does not comply with EN 1998-1 [3] requirements (η <80%). Supplementary details regarding the parametric study are found in [27].

Table 1 Properties of the modelled sub-systems

	Beam				Column	
	Section	L [mm]	d_b [mm]	η [%]	Section	H [mm]
$S_D^{a,b}$	W36x150	8992	911	32	W27x194	3962
S_S^{a}	W16x45	8992	409	32	W14x132	3962

a = Cyclic symmetric loading history up to 6% story drift [29]
b = Collapse consistent loading protocol with two phases [31]
d_b = Beam depth
η_i = Degree of composite action at the interior column [3,30]
η_e = Degree of composite action at the exterior column [3,30]



3 Behaviour of shear studs

3.1 Local hysteretic behaviour

The hysteretic response of the west beam shear studs nearest to the interior joint is shown in Figure 4. The moment-rotation hysteretic response of the west beam (at the column face) is superimposed in the same figure. The aim is to evaluate the influence of the shear stud degradation on the behaviour of the beams at large lateral drift demands. The hysteretic response of the shear studs in sub-systems with deep and shallow beams is compared to that in typical cruciform subassemblies with simplified boundary conditions.

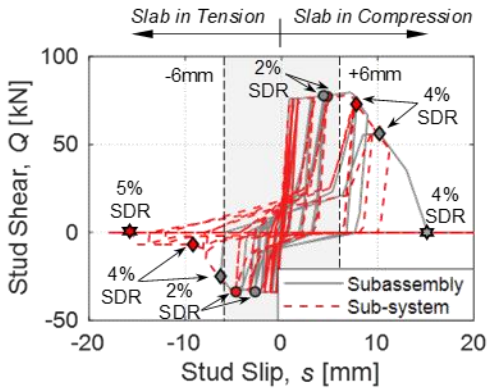
Referring to Figure 4(a), the shear studs in both subassembly and sub-system S_D experience severe strength degradation. At lateral drifts associated with design-basis earthquakes (i.e., 2% SDR), the studs are subjected to large inelastic slip demands (> 5mm). At 4% lateral drift, the shear studs degrade to less than 50% of their ultimate shear capacity. There is a notable difference between the shear stud response in the subassembly and sub-system. First, loss of shear stud capacity in the subassembly occurs at 4% SDR compared to 5% in the sub-system. Second, in the subassembly, the shear stud degrades more under sagging bending and less under hogging bending when compared to that in the sub-system. The reasons behind the difference in shear stud demand in subassemblies and sub-systems are elaborated later in this section. Referring to Figure 4(b), the beam hysteretic behaviour differs between the subassembly and sub-system. This difference arises from the presence of the axial restraint that delays local buckling in sub-systems as discussed in [27]. Additionally, the shear stud behaviour also influences the rate of strength degradation of the composite steel beam. Unlike in the subassembly, the stud integrity, and therefore composite action, is maintained in the sub-system at 4% SDR. Accordingly, local buckling in the top flange of the beam is delayed. On the other hand,

under hogging bending, earlier stud shear strength degradation in the sub-system results in a lower hogging flexural resistance in comparison with the subassembly. The difference is not pronounced since the slab contribution to the flexural strength is low under hogging bending. However, it is reflected by a slightly lower peak flexural strength in the sub-system beam (~10%).

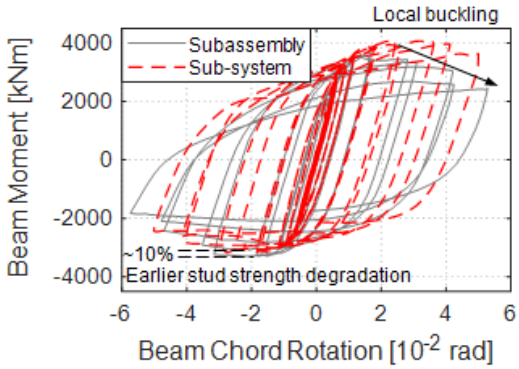
The shear studs in both subassembly and sub-system S_S do not experience large inelastic slip demands (< 1mm) at 2% lateral drift and show no shear strength degradation even at 4% lateral drift (see Figure 4(c)). Eventually, the shear stud in the subassembly loses its load-carrying capacity at 6% SDR while that in the sub-system maintains its integrity. It is evident that the non-linear load-slip behaviour in S_S (with shallow beams) deviates from that in S_D (with deep beams) despite the fact that they have the same slab configuration, number of studs and degree of composite action. The lower shear demand in the shallow beam results in much less strength degradation in the shear studs connecting the beam to the slab. Referring to Figure 4(d), the peak hogging flexural resistance attained in the beam is the same in the sub-system as in the subassembly. In both cases, the shear studs do not degrade under hogging bending. In contrast, the subassembly beam exhibits flexural strength degradation during the last sagging bending cycle (6% SDR). This is mainly due to shear stud degradation and not due to local buckling in the beam as revealed by the CFE model. The ductility of composite beams with partial interaction is governed by the ultimate slip capacity of the shear studs [32].

It is important to acknowledge the uncertainty in the value of the ultimate slip capacities, $s_u^{\pm}$ (see Figure 2). Lower values of $s_u^{\pm}$ may result in the shear studs losing their integrity at modest lateral drift demands than those shown in Figure 4. In that respect, the shear studs can be assessed for a lower bound limit for $s_u^{\pm}$. EN 1994-1-1 [30] define ductile shear studs as those having a characteristic deformation capacity, δ_{uk}= 6mm, at 10% drop in ultimate shear resistance [33]. At 4% SDR, the maximum stud slip demand is 11mm for sub-system S_D and 2mm for sub-system S_S. In the latter, no loss in stud shear resistance is observed at the specified lateral drift demand. On the other hand, the slip demand at 2% SDR associated with a design-basis earthquake (10% probability of exceedance over 50 years) is within 4mm and 2mm for sub-systems S_D and S_S, respectively and no shear strength degradation is observed. Based on the above, the shear stud integrity is maintained in the sub-system S_S at 2% and 4% SDR, and in the sub-system S_D at 2% SDR. Moreover, for higher degrees of composite action, the shear demand on the studs is expected to be lower. Further investigation of the shear stud behaviour along the beam span is conducted in section 3.2.

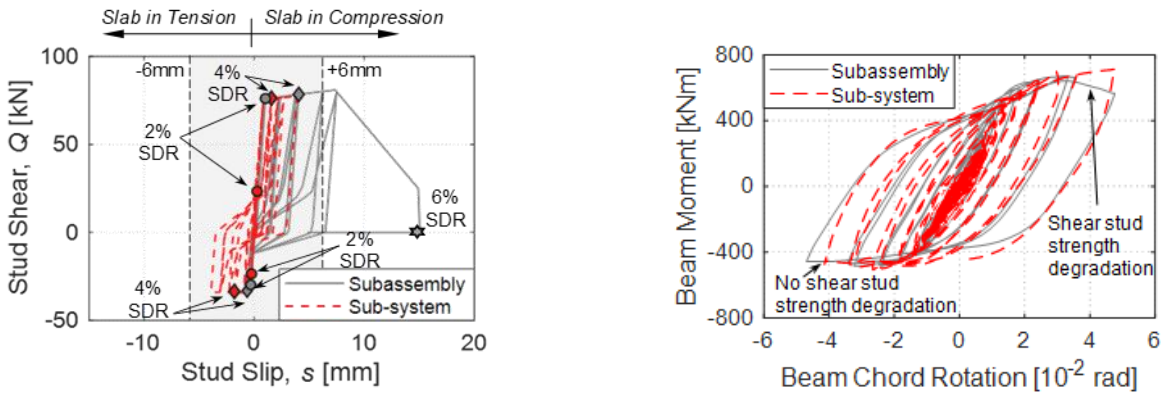
The difference in the slip demand between the shear studs in subassemblies and sub-systems arises from the difference in boundary conditions. In subassemblies, the inflection points are enforced at the beam ends (see Figure 1(a)). As a result, the shear span is constant regardless of the sign of the excursion. Conversely, the shear span in sub-systems depends on the magnitude of the sagging and hogging moments at the beam ends. It is larger than that in subassemblies under sagging bending and smaller under hogging bending. Based on the above discussion, the following two phenomena are used to explain the difference in the shear stud hysteretic behaviour between subassemblies and sub-systems. First, in sub-systems, the number of shear studs in the shear span is lower than that in the subassemblies under hogging bending excursions and higher under sagging bending excursions. Accordingly, the shear demand on the studs in sub-systems is higher than that in subassemblies under hogging bending and lower under sagging bending. Second, in subassemblies, beams are free to shorten axially at their ends as local buckling progresses. Axial shortening in the subassembly beams pries the slab from the beams. Referring to Figure 5(a), the slip demand on the shear studs under sagging bending, s_o^+ is increased by s_{prying} and that under hogging bending, s_o^- is decreased by s_{prying}. In contrast, sub-system beam axial shortening is limited as shown in Figure 5(b). Note that at SDRs less than 2%, axial shortening is limited and the hysteretic behaviour of the shear studs coincide as shown in Figure 4(a,d). Once beam axial shortening becomes substantial in the subassemblies, a discrepancy in the behaviour of the shear studs is observed. Sub-systems better replicate the boundary conditions in real buildings than subassemblies. Therefore, the shear stud behaviour in sub-systems is deemed representative of those in composite-steel MRFs.



a) S_D

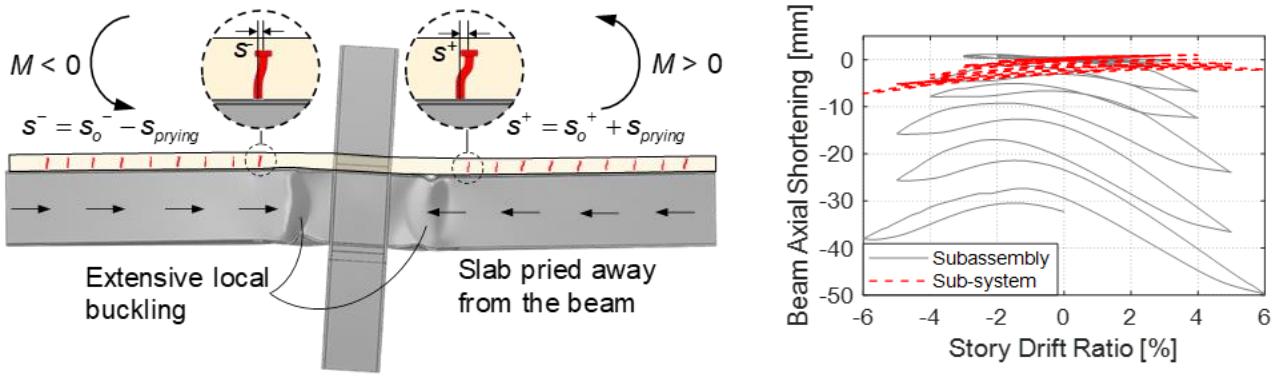


b) S_D



c) S_S d) S_S

Figure 4 Comparison between sub-system and sub-assembly response: a, c) load-slip hysteretic response of the west beam shear stud nearest to the interior joint; b, d) flexural hysteretic response of the west beam at the interior joint



a) b)

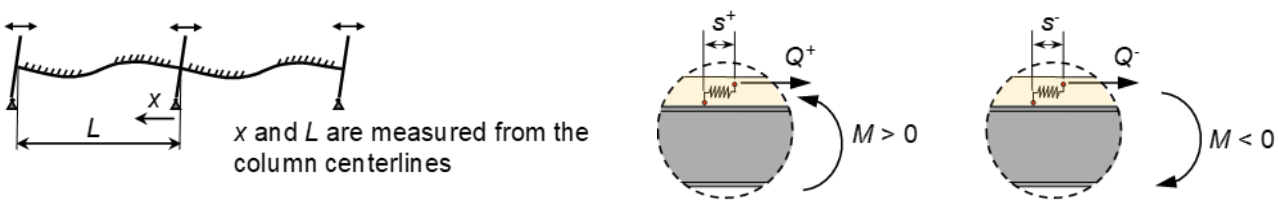
Figure 5 a) Effect of beam axial shortening on the slip demand of shear studs in a subassembly; b) comparison of west beam axial shortening in sub-system and subassembly S_D

3.2 Response envelopes

In the previous section, the hysteretic behaviour of the studs near the interior joint is assessed for sub-systems with deep and shallow beams. However, the demand at these locations may be low due to the presence of a slip restraint at the beam-column joints [15]. It is therefore necessary to investigate the maximum slip demand and shear strength of the studs along the composite-steel beams.

Figure 6 shows the slip, s, and normalized shear strength, $Q/Q_u^{\pm}$ envelopes of the west beam studs at targeted lateral drift demands. Referring to Figure 6(b,d), the slip demand is lower (up to 3mm) in the studs near the joints than those further away from the joints. Contrary to the shear stud near the joints (see Figure 4(a)), in sub-system S_D, the shear studs within 0.2x/L and 0.6x/L experience a slip demand larger than 6mm at 2% SDR. Figure 6(c) shows that these studs experience severe strength degradation at 2% SDR and attain a zero shear strength at 4% SDR. On the other hand, the shear studs in sub-system S_S do not experience slip demands exceeding 6mm even at 4% SDR. Shear stud strength degradation in sub-system S_S initiates at 6% SDR (Figure 7(e)).

Vis-à-vis the above discussion, and considering the uncertainty associated with the moment frame behaviour due to loss of composite action [2], the following design recommendations are proposed: (i) For composite-steel MRFs with deep beams ($d_b \geq 700$mm), the 25% reduction in stud shear strength is required as recommended by design provisions [3,4]. Of course, this depends on the degree of composite action as well. In EN 1998-1 [3], where the minimum degree of composite action is 80%, the shear strength reduction may not be critical. However, as a precaution against early stud failure, the strength reduction is required; (ii) for composite-steel MRFs with shallow beams ($d_b \leq 500$mm), the behaviour of the shear studs is within that of ductile shear studs according to EN 1994-1-1. No reduction in the shear stud strength is recommended even for partially composite beams (η~30%).



a) Definition

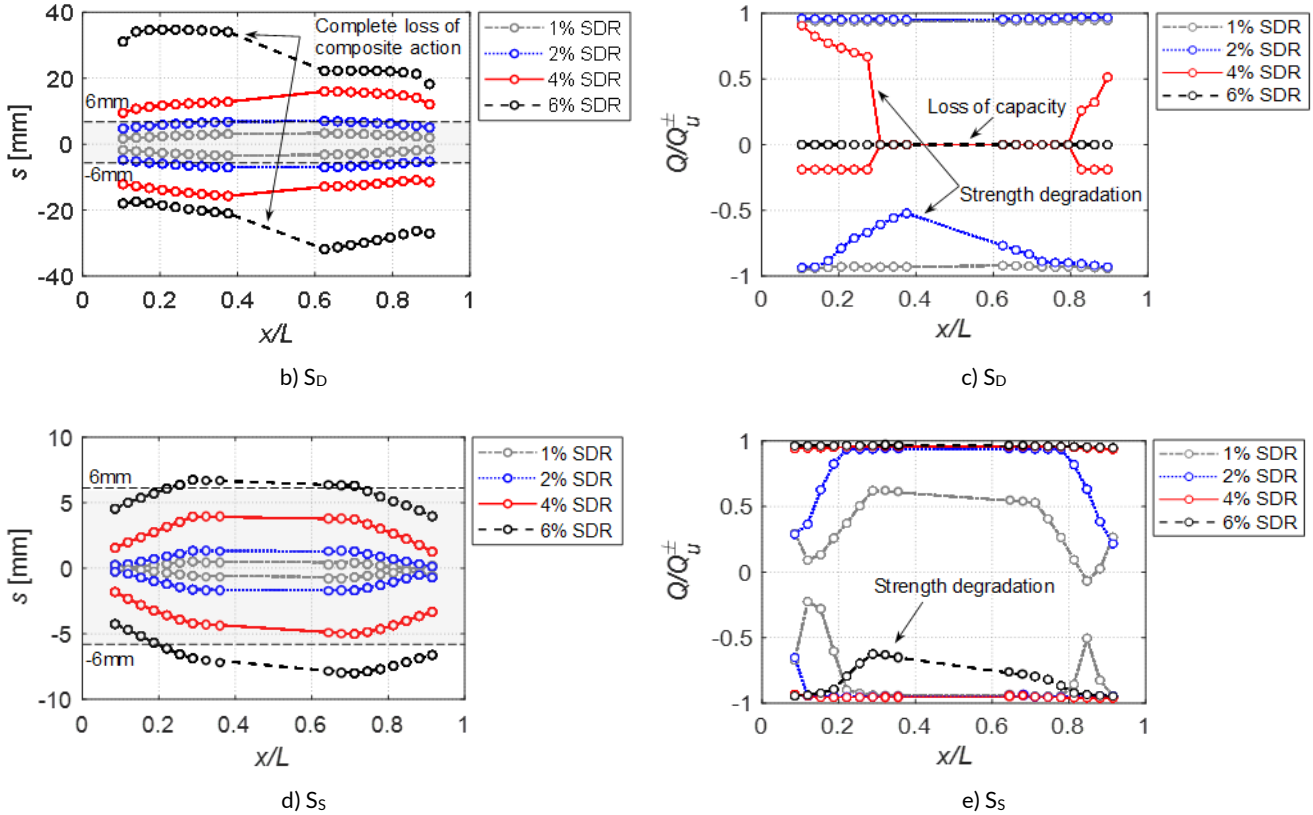


Figure 6 Response envelopes along the west beam at various lateral drift demands; b, d) stud slip envelope; c, e) stud shear force envelope

3.3 Influence of loading protocol

The hysteretic behaviour of shear studs was examined under a symmetric cyclic lateral loading protocol. Although informative, this protocol overestimates the seismic demands in MRFs. Collapse-consistent protocols [31,34,35] represent a more realistic alternative for estimating the seismic demands at large deformations associated with structural collapse. For this reason, sub-system S_D is subjected to a collapse-consistent loading protocol developed by [31]. The protocol consists of an asymmetric drift demand that represents the "ratcheting" behaviour observed in MRFs prior to earthquake-induced collapse [36,37]. The hysteretic response of the west beam shear stud near the interior joint is compared between both protocols.

Referring to Figure 7, under both loading protocols, the shear studs lose their capacity at 5% SDR. For similar drift values, the slip demand is similar. Moreover, the rate of shear strength degradation is nearly the same. The loading protocol independency may be explained by the fact that cyclic degradation in the 19mm studs is minimal as shown in Figure 2. Having said that, if studs with larger diameters were used (e.g. 22mm), shear stud strength cyclic degradation becomes significant [12] as does the effect of the loading protocol on the stud hysteretic behaviour. However, stud diameters exceeding 19mm are not commonly used in European composite-steel MRFs in which shallow beams are employed. Furthermore, ANSI/AISC 341-16 [4] limits the stud diameter in composite-steel MRFs to 19mm.

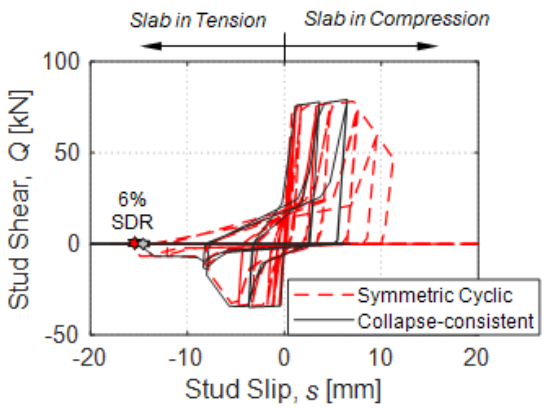


Figure 7 Load-slip hysteretic response of the west beam shear stud nearest to the interior joint – comparison between symmetric cyclic and collapse consistent loading protocols

4 Limitations

It is worth noting that although the model is capable of capturing the non-linear load slip behaviour at the beam-slab interface, it is not void of limitations. The model cannot capture fracture of the stud-to-beam welds. Furthemore, it does account for uplift or out-of-plane forces acting on the studs. With regard to the shear stud demand, the effects of gravity load are ignored. Nevertheless, they may not be significant since the additional loading of shear studs in one half is offset by a concurrent unloading in the other half [38].

5 Conclusions

This paper investigates the hysteretic load-slip behaviour of shear studs in composite-steel MRFs. A detailed CFE model is proposed and validated for this purpose. The hysteretic behaviour of the shear

studs, as well as the interface shear/slip demands along the composite-steel beam, is compared between subassemblies and sub-systems with two different beam depths. The main conclusions are:

1. In composite-steel MRFs with shallow and partially composite-steel beams, shear studs maintain their integrity at 4% SDR. On the other hand, in deep beams, with a similar degree of composite action, shear studs experience higher slip demands and shear strength degradation even at 2% SDR. This is due to the higher seismic shear in sub-systems with deep beams.
2. The behaviour of shear studs in sub-systems is more representative than that in subassemblies. In the latter, axial shortening and the constant length of the shear span result in higher shear demand under sagging bending, and lower demand under hogging bending.
3. The slip-restraint near the beam-to-column joints may lead to lower (up to 3mm) slip demand in the shear studs close to the column than those further away from the column.
4. For composite-steel MRFs comprising shallow beams ($d_b \leq$ 500mm), the 25% reduction in shear strength capacity proposed by seismic provisions [3,4] may be waived as long as ductile shear connectors are used.
5. The hysteretic behaviour of shear studs commonly employed in composite-steel MRFs (≤19mm) does not seem to be influenced by the imposed loading history.

6 Acknowledgments

This study is based on work supported by the Swiss National Science Foundation (Project No. 200021_169248). The financial support is gratefully acknowledged. Any opinions expressed in the paper are those of the authors and do not necessarily reflect the views of sponsors.

References

[1] Nakashima M.; Matsumiya T.; Suita K.; Zhou F. (2007) *Full-Scale test of composite frame under large cyclic loading*. Journal of Structural Engineering **133**(2): 297–304

[2] Cordova PP.; Deierlein G. (2005) *Validation of the seismic performance of composite RCS frames: Full-scale testing, analytical modeling, and seismic design*. Stanford, CA: The John A. Blume Earthquake Engineering Center, Stanford University; 2005

[3] CEN. (2004) *EN 1998-1: Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*. Brussels, Belgium: European Committee for Standardization; 2004

[4] AISC. (2016) *Seismic provisions for structural steel buildings, ANSI/AISC 341-16*. Chicago, IL: American Institute for Steel Construction; 2016

[5] Bursi OS.; Gramola G. (1999) *Behaviour of headed stud shear connectors under lowcycle high amplitude displacements*. Materials and Structures **32**(4): 290–297

[6] Civjan SA.; Singh P. (2003) *Behavior of shear studs subjected to fully reversed cyclic loading*. Journal of Structural Engineering **129**(11): 1466–1474

[7] Hawkins NM.; Mitchell D. (1984) *Seismic response of composite shear connections*. Journal of Structural Engineering **110**(9): 2120–2136

[8] Zandonini R.; Bursi OS. (2002) *Cyclic behavior of headed stud shear connectors*. Composite Construction in Steel and Concrete IV

[9] Cheng CT.; Chen CC. (2005) *Seismic behavior of steel beam and reinforced concrete column connections*. Journal of Constructional Steel Research **61**(5): 587–606

[10] Leon RT.; Hajjar JF.; Gustafson MA. (1998) *Seismic response of composite moment-resisting connections. I: Performance*. Journal of Structural Engineering **124**(8): 868–876

[11] Chaudhari T.; MacRae G.; Bull D.; Clifton C.; Hicks S. (2019) *Experimental behaviour of steel beam-column subassemblies with different slab configurations*. Journal of Constructional Steel Research **162**: 105699

[12] Suzuki A.; Kimura Y. (2019) *Cyclic behavior of component model of composite beam subjected to fully reversed cyclic loading*. Journal of Structural Engineering **145**(4): 04019015

[13] Schafer B.; Easterling WS.; Sabelli R.; Eatherton MR.; Hajjar JF. (2019) *Steel Diaphragm Innovation Initiative Workshop Report*. The Cold-Formed Steel Research Consortium (CFSRC); 2019

[14] Sjaarda M.; Walbridge S.; West JS. (2018) *Assessment of shear connection through composite beam modeling*. Transportation Research Record: 0361198118781685

[15] Zona A.; Barbato M.; Conte JP. (2008) *Nonlinear seismic response analysis of steel–concrete composite frames*. Journal of Structural Engineering **134**(6): 986–997

[16] Lam D.; El-Lobody E. (2005) *Behavior of Headed Stud Shear Connectors in Composite Beam*. Journal of Structural Engineering **131**(1): 96–107

[17] Baskar K.; Shanmugam NE.; Thevendran V. (2002) *Finite-Element Analysis of Steel–Concrete Composite Plate Girder*. Journal of Structural Engineering **128**(9): 1158–1168

[18] Alashker Y.; El-Tawil S.; Sadek F. (2010) *Progressive collapse resistance of steel-concrete composite floors*. Journal of Structural Engineering **136**(10): 1187–1196

[19] Liang QQ.; Uy B.; Bradford MA.; Ronagh HR. (2005) *Strength Analysis of Steel–Concrete Composite Beams in Combined Bending and Shear*. Journal of Structural Engineering **131**(10): 1593–1600

[20] Zhang X.; Ricles J.; Lu LW.; Fisher J. (2004) *Development of seismic guidelines for deep-column steel moment connections*. Lehigh University; 2004

[21] Zhou F.; Mosalam KM.; Nakashima M. (2007) *Finite-element analysis of a composite frame under large lateral cyclic loading*. Journal of Structural Engineering **133**(7): 1018–1026

[22] Ayoub A.; Filippou FC. (2000) *Mixed Formulation of Nonlinear Steel-Concrete Composite Beam Element*. Journal of Structural Engineering **126**(3): 371–381

[23] Bursi OS.; Sun FF.; Postal S. (2005) *Non-linear analysis of steel–concrete composite frames with full and partial shear connection subjected to seismic loads*. Journal of Constructional Steel Research **61**(1): 67–92

[24] Gattesco N. (1999) *Analytical modeling of nonlinear behavior of composite beams with deformable connection*. Journal of Constructional Steel Research **52**(2): 195–218

[25] Salari MR.; Spacone E. (2001) *Analysis of steel-concrete composite frames with bond-slip*. Journal of Structural Engineering **127**(11): 1243–1250

[26] Abaqus. (2014) *Abaqus 6.14 Documentation*. RI, USA: Dassault Systemès Simulia Corp.; 2014

[27] El Jisr H.; Lignos DG.; Elkady A. (2020) *Hysteretic Behavior of Moment-Resisting Frames Considering Slab Restraint and Framing Action*. Journal of Structural Engineering **146**(8)

[28] Johansson M.; Gylltoft K. (2002) *Mechanical behavior of circular steel–concrete composite stub columns*. Journal of Structural Engineering **128**(8): 1073–1081

[29] SAC Joint Venture. (1997) *Protocol for fabrication, inspection, testing, and documentation of beam-column connection tests and other experimental specimens*. Rep No SAC/BD-97 **2**

[30] CEN. (2004) *EN 1994-1-1: Eurocode 4: Design of composite steel and concrete structures – Part 1-1: General rules and rules for buildings*. Brussels, Belgium: European Committee for Standardization; 2004

[31] Suzuki Y.; Lignos DG. (2019) *Development of collapse-consistent loading protocols for experimental testing of steel columns*. Earthquake Engineering & Structural Dynamics **49**(2): 114–131

[32] Ayoub A. (2005) *A force-based model for composite steel–concrete beams with partial interaction*. Journal of Constructional Steel Research **61**(3): 387–414

[33] Bärtschi R. (2005) *Load Bearing Behaviour of Composite Beams in Low Degrees of Partial Shear Connection*. Zürich: Swiss Federal Institute of Technology Zürich; 2005

[34] Krawinkler H. (2009) *Loading histories for cyclic tests in support of performance assessment of structural components*. *The 3rd International Conference on Advances in Experimental Structural Engineering*, International Conference on Advances in Experimental Structural Engineering. San Francisco: 2009

[35] Maison BF.; Speicher MS. (2016) *Loading protocols for ASCE 41 backbone curves*. Earthquake Spectra **32**(4): 2513–2532

[36] Lignos DG.; Hikino T.; Matsuoka Y.; Nakashima M. (2013) *Collapse assessment of steel moment frames based on E-Defense full-scale shake table collapse tests*. Journal of Structural Engineering **139**(1): 120–132

[37] Lignos DG.; Krawinkler H.; Whittaker AS. (2011) *Prediction and validation of sidesway collapse of two scale models of a 4-story steel moment frame*. Earthquake Engineering and Structural Dynamics **40**(7): 807–825

[38] AISC. (2016) *Specification for structural steel buildings, ANSI/AISC 360-16*. Chicago, IL: American Institute for Steel Construction; 2016