Proposed nonlinear macro-model for seismic risk assessment of composite-steel moment resisting frames

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Abstract
This paper proposes a macro-model for simulating the hysteretic behavior of composite-steel beams as part of fully restrained beam-to-column connections in composite-steel moment-resisting frames (MRFs). Comparisons with experimental data suggest that the proposed model captures the asymmetric hysteretic response of composite-steel beams including the cyclic deterioration in strength and stiffness. Moreover, the proposed model captures the primary slab-column force transfer mechanisms and predicts the slip demands in beam-slab connections under inelastic cyclic loading. The modeling approach is employed in a system-level study to benchmark the seismic collapse risk of composite-steel MRF buildings across Europe. Moreover, the beam-slab slip demands are quantified through the development of beam-slab slip hazard curves. The simulation studies suggest that the examined composite-steel MRFs exhibit a system overstrength of about 4. This is attributed to the drift requirements in the current European seismic provisions. The annualized probability of collapse of the prototype buildings is well below 1% over a 50-year building life expectancy regardless of the design site and the degree of composite action. Beam-slab connections with a partial degree of composite action experience minimal damage for frequently occurring seismic events (i.e., 50% probability of exceedance over 50 years); and light cracking in the slab for a design basis earthquake. The above are important from a seismic repairability standpoint. Accordingly, it is recommended that the 25% reduction in the shear resistance of stud connectors is not imperative for seismic designs that feature steel beams with depths less than 500 mm.

KEYWORDS
beam-slab connection slip demands, collapse risk assessment, composite-steel moment resisting frames, peak slip hazard curves, slab repairability

1 | INTRODUCTION

Performance-based seismic evaluation of frame buildings has gained attention over the past two decades. While capacity design rules have been benchmarked to establish a tolerable risk for collapse safety of modern steel and...
reinforced concrete (RC) moment-resisting frames (MRFs), there is a lack of similar studies for composite-steel MRFs, that constitute the main focus of this paper. Denavit et al. 16 proposed seismic performance factors for moment frames with steel-concrete composite columns and steel beams by employing the FEMA P695 methodology. However, the role of slab was disregarded in this case.

Prior modeling approaches to consider the role of slab in full- and/or partially-composite steel MRFs disregarded important aspects of the composite beam behavior. Mehanny and Deierlein 17 examined the seismic behavior of composite MRFs. However, the resultant section models ignored the effects of shear stud degradation. Others 18–20 employed fiber-based models to simulate the hysteretic behavior of composite-steel members, with the same limitations as those discussed in Mehanny and Deierlein. 17 Furthermore, these models disregarded the influence of cyclic deterioration due to cross-sectional local buckling. Elkady and Lignos 4 proposed a holistic approach to consider the effects of the composite action on the hysteretic behavior of composite-steel beams exhibiting cyclic deterioration in flexural strength and stiffness. However, they only focused on deep beams (i.e., depths larger than 500 mm), which are typical in the US seismic design practice, with a relatively low degree of composite action (i.e., 10%). Moreover, the deteriorating behavior of the shear stud connectors was disregarded. Other modeling approaches include the component method 21 which is suitable for system-level nonlinear simulations of MRFs with composite floor slabs. However, this method has been mainly used to simulate the behavior of partially-restrained beam-to-column connections. 22–24 In particular, the degrading hysteretic behavior of the shear studs as well as that of steel beams has been neglected.

Bursi et al. 18 and Zona et al. 20 studied the nonlinear dynamic response of composite-steel MRFs with a variable degree of composite action. Composite-steel beams with a partial degree of composite connection (i.e., < 80%) may result in cost savings through the reduction in the number of shear studs and the transverse reinforcement of the slab. Moreover, partially composite beams impose lower demands on the columns and the beam-to-column web panel zone joint than their fully composite counterparts. 18 On the other hand, the loss of composite action due to cyclic deterioration in the shear resistance of beam-slab connectors is undesirable since it leads to the loss of the load transfer mechanism between the beam-slab connections. 25

The current seismic provisions 1,26 propose a 25% reduction in the design shear resistance of the beam-slab connectors in order to decrease the inelastic slip demand on the shear studs and maintain their integrity. According to EN 1998-1, 1 the beneficial aspects of the composite action may only be considered in seismic design when the degree of composite action is at least 80%. Moreover, conventional push-out 27–29 and subassembly tests 30–32 have shown that shear studs experience severe shear degradation under cyclic loading. Nevertheless, Suzuki and Kimura 33 found that in composite-steel MRFs, the shear stud hysteretic performance is far different from that deduced by conventional cyclic push-out tests because of the stress state in the slab (i.e., concrete slab subjected to tensile or compressive stresses upon load reversals). This is acknowledged in recent beam-slab connection tests with more realistic boundary conditions. 33 El Jis et al. 34 showed that the slip demands in beam-slab connections of capacity-designed composite-steel MRFs is not large enough to endanger the integrity of partially composite beams with shallow cross sections (depth less than 500 mm). To the best of our knowledge, the shear connector slip demands have never been benchmarked by means of system-level nonlinear response history analyses. These are essential in establishing failure rate criteria for beam-slab connections in capacity-designed composite-steel MRFs in a consistent manner with performance-based design. Such an endeavor requires a nonlinear model that explicitly simulates the slip demands within beam-slab connections.

In this paper, we propose a nonlinear macro-model for simulating the deteriorating cyclic behavior of fully restrained beam-to-column connections in composite-steel MRFs. The proposed model, which is computationally efficient, captures the deteriorating response of the shear connectors and the composite-steel beam under monotonic and reversed cyclic loading. The proposed macro-model is thoroughly validated with available experimental data from prior testing programs. The model is then incorporated in nonlinear models of prototype composite steel MRF buildings designed according to EN 1998-1 1 at three different European sites. The degree of composite action in the composite beam is varied and its influence on the seismic behavior of the composite-steel MRFs is evaluated by means of nonlinear building simulations. Particularly, we quantify the site-specific collapse risk dependence on the assumed degree of composite action. Finally, we quantify the integrity of the beam-slab connection in the prototype composite-steel MRFs through novel beam-slab connection slip hazard curves, which may constitute a valuable tool for more effective designs of composite-steel MRFs in prospective revisions of current seismic design standards. 1,26

## 2 Behavioral Insights on Composite Beam-to-Column Connections

Experimental studies on subassemblies 31,35–37 and frames 38–40 with a composite slab have demonstrated that the behavior of composite connections differs from their bare steel counterparts. The differences are primarily manifested in (i) the
FIGURE 1 Typical fully-restrained composite beam-to-column connection (A) internal forces in the composite-steel beams; (B) slab-column force transfer mechanisms; (C) comparison between the moment rotation response of a bare and a composite-steel beam

flexural stiffness and strength of composite-steel beams under sagging and hogging bending; (ii) the pre- and post-peak plastic rotation under sagging and hogging; and (iii) the inelastic behavior of the beam-to-column web panel zones.

The presence of the slab augments the flexural stiffness of beams under sagging bending by up to two times. Moreover, a flexural strength enhancement (up to 80%) is particularly evident under sagging bending and is dependent on the beam depth, the slab properties and the degree of composite action. Figure 1(A) illustrates the axial forces \( F_s^\pm, F_b^\pm \) and bending moments \( M_s^\pm, M_b^\pm \) in the floor slab (noted with subscript \( s \)) and steel beam (noted with subscript \( b \)) under sagging and hogging bending, respectively. During an earthquake, the inertia forces are transferred to the steel beams through the shear connectors and friction between the beam-slab interface. Moreover, the inertia forces pass to the column through two primary mechanisms, that is, column face bearing and the strut-and-tie mechanism as shown in Figure 1(B).

The characterization of the hysteretic behavior of the shear connectors is imperative for capturing the flexural resistance of composite-steel beams. Cyclic push-out tests that accounted for the slab stress state demonstrated that the hysteretic behavior of the shear studs becomes asymmetric under reversed cyclic loading. Moreover, shear strength degradation of the studs is not as pronounced as that inferred by conventional cyclic push-out tests.

The presence of the slab restraints the top beam flange against local buckling. As a consequence, the rate of flexural strength deterioration under sagging bending decreases relative to that of the bare steel beam. On the other hand, the plastic rotation capacity under hogging bending decreases relative to that of the bare steel beam. This is caused by the upward shift of the neutral axis of the composite beam cross section due to the presence of reinforcing rebars in the slab. Figure 1(C) shows schematically the influence of the floor slab on the backbone monotonic curve of a composite-steel beam under sagging and hogging bending.

The presence of the slab may influence the behavior of the beam-to-column web panel zone. This is attributed to the increased moment demand at the center of the panel zone, and the increased effective depth of the panel zone under sagging bending. Collectively, the above important findings are confirmed through experimental evidence as well as high-fidelity continuum finite element analyses. The aforementioned behavioral insights should be incorporated in the proposed macro-model of composite-steel beams.

3 PROPOSED MODEL FOR COMPOSITE-STEEL BEAMS

Figure 2 shows schematically the proposed macro-model for simulating the response of composite-steel beams. The model consists of an elastic beam-column element and seven nonlinear elements. These elements simulate both the bearing
and strut-and-tie force transfer mechanisms of the slab-column. Moreover, the slab rebar and beam-slab interaction are considered.

The flexural stiffness of the elastic beam-column element is adjusted by using an equivalent moment of inertia, which may be calculated as follows,

$$I_{eq} = 0.6I^+ + 0.4I^-$$  \hspace{1cm} (1)

$$I^+ = I_{bare} + \sqrt{\eta \cdot (I_{c,f} - I_{bare})}$$  \hspace{1cm} (2)

$$I^- = I_{bare}$$  \hspace{1cm} (3)

$$I_{eq} = I_{bare} + 0.6 \sqrt{\eta \cdot (I_{c,f} - I_{bare})}$$  \hspace{1cm} (4)

in which, $I^+$ and $I^-$ are the moments of inertia of the composite-steel beam under sagging and hogging bending, respectively; they are calculated as per EN 1998-1\(^1\) or ANSI/AISC 360–16\(^45\); $I_{bare}$ is the second moment of area of the bare steel beam; $I_{c,f}$ is the moment of inertia of the uncracked fully composite-steel beam under sagging bending. It may be calculated either by EN 1998-1\(^1\) or by ANSI/AISC 360–16\(^45\); and $\eta$ is the degree of composite action under sagging bending. This is defined as the ratio of the actual number of shear studs to that required to achieve full composite action.

Referring to Figure 2, the three axial nonlinear elements represent the two mechanisms of the slab-to-column force transfer under sagging bending, in addition to the longitudinal rebar under hogging bending. These elements are assumed to act at the mid-depth portion of the slab above the profiled steel deck. Their length is assumed to be equal to 1.5 times the column depth, $h_c$.\(^{24,46}\) The dissipative zones of the bare steel beams are idealized by zero-length rotational elements. Furthermore, the beam-slab connection is modeled using axial zero-length elements that lump together the shear force-slip relationship of all the shear studs per shear span. Referring to Figure 2, the parallelogram model\(^{10}\) may be adopted to simulate the hysteretic behavior of the beam-to-column web panel zone.

The sagging and hogging flexural demands on the composite-steel beam may be calculated as follows (see Figure 1),

$$M^\pm = M_b^\pm - 0.5F_s^\pm \cdot (h_b + h_s + h_p)$$  \hspace{1cm} (5)

in which, $M_b^\pm$ is the bending moment in the steel beam that considers the interaction of bending and axial load due to the presence of the slab as discussed later on; $F_s^\pm$ is the axial force in the slab due to the composite action. Referring to Figure 1(A), this force is assumed to act at the centerline of the concrete above the profiled steel deck; $h_b$ is the depth of the steel beam; $h_s$ is the thickness of the floor slab; and $h_p$ is the height of the profiled steel deck (see Figure 1A).

Figure 3(A) shows mechanism 1, that is, bearing of the slab on the column flange. The force transferred by direct compression spreads through the effective width of the slab under sagging bending, $b_{eff}^+$, as per EN-1998-1.\(^1\) The length of the compression strut along the direction of loading, is assumed to be $L_{M_1} = 0.5b_{eff}^+$.\(^{42}\) The force-displacement relationship of the axial nonlinear element is shown in Figure 3(B). The peak force, $F_{R1}$, transferred to the column through mechanism
1 is calculated as follows,

\[ F_{R1} = f'_c b_c (h_s - h_p) \]  \hspace{1cm} (6)

in which, \( f'_c \) is the concrete compressive strength and \( b_c \) is the width of the column flange.

Referring to Figure 3(B), the adopted constitutive relationship is a tri-linear backbone under monotonic loading. A peak-oriented hysteretic response represents the cyclic behavior of the connection. In the first segment of the backbone, the concrete strut is elastic up to a force of 0.6\( F_{R1} \). The corresponding displacement at 0.6\( F_{R1} \) is \( \delta_{el,1} = 0.6\left(\frac{f'_c}{E_c}\right) \cdot L_{M1} \) (\( E_c \) is the modulus of elasticity of concrete). In the second segment, \( F_{R1} \) is attained at a displacement of \( \delta_{p,1} = 0.2 \% \cdot L_{M1} \). The post-peak capping behavior assumes that the concrete is confined and therefore crushing is delayed. If the slab is properly detailed according to EN-1998-1\(^1\) the degradation in \( F_{M1} \) may be disregarded.\(^{22,24}\) On the other hand, Braconi et al.\(^{23}\) distinguished between confined and unconfined concrete below and above the reinforcing steel, respectively. The former exhibits elastic-perfectly plastic behavior while the latter exhibits degrading behavior. In the proposed model, the post-peak portion is linear and is characterized by a 15% strength drop at \( \delta_{u,1} = 2\% \cdot L_{M1} \) as shown in Figure 3(B). Validation studies with subassembly tests of composite beam-to-column connections featuring slab detailing that conforms to EN-1998-1\(^1\) showed that the assumed modeling assumptions result in a relatively good agreement with the experimental findings as discussed in the next section.

Referring to Figure 4(A), mechanism 2 (strut-and-tie) is composed of two compressive concrete struts and one steel tie in tension. Mechanism 2 can be considered as a combination of the compressive struts in series with the tension tie in the direction of force \( F_{M2} \). The strut inclination, \( \theta \), is assumed to be 45\(^0\) and the strut crushing resistance factor, \( \nu = 0.6 \).\(^{42}\) The length of the concrete struts, along the direction of loading, \( L_{M2} = 2h_c \), while the length of the steel tie \( L_{Tie} = 2L_{M2} + b_c \). The peak compressive force in the concrete struts, along the direction of loading, is shown below for \( \theta = 45^0 \) and \( \nu = 0.6 \),

\[ F_{R2} = 0.6f'_c h_c (h_s - h_p) \]  \hspace{1cm} (7)
a bi-linear elastic perfectly plastic behavior is assigned to the steel tie. The yield tensile force in the tie, $F_{Tie}$ may be calculated as follows,

$$F_{Tie} = 2A_t f_{ys}$$

in which, $A_t$ is the area of the seismic rebars within $h_c$ from the column face$^1$; and $f_{ys}$ is the expected yield stress of the seismic rebars.

Referring to Figure 4(B), the yield displacement, along the direction of loading is $\delta_{ys} = 0.5(f_{ys}/E_s) \cdot L_{Tie}$ ($E_s$ is the modulus of elasticity of steel). Furthermore, Figure 4(B) shows that the constitutive law of the concrete struts in mechanism 2 is akin to that of the concrete strut in mechanism 1. The difference lies in the peak resistance, $F_{R2}$, and the length along the direction of loading, $L_{M2}$ of the struts. Accordingly, $\delta_{el,2} = 0.6(f'/c/E_s) \cdot L_{M2}$, $\delta_{p2} = 0.2\% \cdot L_{M2}$ and $\delta_{u,2} = 2\% \cdot L_{M2}$. Referring to Figure 4(B), mechanism 2 is controlled by yielding of the ties prior to crushing of the struts. Noteworthy stating that mechanism 2 is not activated in perimeter composite-steel MRFs. The lack of slab continuity at the edge implies that the width of the floor slab is insufficient for the development of the strut-and-tie mechanism.$^{1,42}$

The nonlinear element corresponding to the slab longitudinal reinforcement is characterized by an elastic perfectly plastic axial force-displacement relationship. The yield force in the element is equal to $A_{sr}f_{yr}$; where $A_{sr}$ is the area of slab reinforcement within the effective width, $b_{eff}$; and $f_{yr}$ is the expected yield stress of the slab reinforcement. The elastic stiffness of the nonlinear element is evaluated according to.$^{24,46}$

The degrading response of the bare steel beam is simulated by the modified Ibarra-Medina-Krawinkler (IMK) deterioration model.$^{47,48}$ The monotonic backbone and cyclic degradation parameters of the rotational zero-length element of the bare steel beam are based on those developed by Lignos and Krawinkler$^{48}$ and the modeling guidelines by Elkady and Lignos.$^4$ The parameters are adjusted for the physical phenomena discussed earlier. The effective yield moments of the bare steel beam hinge, $M_{y,b}$, may be calculated as follows,

$$M_{y,b} = 1.1 \cdot (1 - \alpha) W_{pl,y} f_y$$

in which, $W_{pl,y}$ is the bare beam plastic section modulus and $f_y$ is the expected material yield strength; $\alpha$ is a strength reduction factor to account for the moment-axial force interaction in the steel beam due to composite action. The strength reduction factor may be computed as per ANSI/AISC 360-16$^{45}$ (see Equation 13). To that end, the axial load ratios in the steel beam under sagging ($P^+/P_y$) and hogging ($P^-/P_y$) bending are considered. Note that if the axial load ratios were to be calculated assuming the maximum force under sagging and hogging bending, $M_{y,b}$ would be overly conservative. The axial load ratio varies during the earthquake and the peak axial force is only transmitted once the peak force in the slab is attained. The axial load decreases as the concrete and/or shear stud strength degrades. Accordingly, an average axial force corresponding to half the maximum force under sagging and hogging bending, $M_{y,b}$, is considered; $P^+\leq 0.5n_oQ_u$ (10); $P^-\leq 0.5A_b f_{yr}$ is the plastic axial resistance of the steel beam with a cross-sectional area $A_b$, and an expected yield stress, $f_y$.

$$P^+ = 0.5 \cdot (F_{R1} + F_{R2}) \leq 0.5n_oQ_u$$

$$P^- = 0.5A_{sr} f_{yr} \leq 0.5n_oQ_u$$

in which, $n_o$ is the number of shear studs in the shear span of the composite beam; and $Q_u$ is the resistance of the shear studs according to EN-1998-1.$^1$

The proposed strength reduction factor, $\alpha$, is calculated as the average reduction under sagging ($\alpha^+$) and hogging ($\alpha^-$) bending as follows,

$$\alpha = (\alpha^+ + \alpha^-)/2$$

$$\alpha^\pm = \begin{cases} 1/8 \cdot (9P^+/P_y - 1), & P^+/P_y \geq 0.2 \\ P^\pm/2P_y, & \text{otherwise} \end{cases}$$

$$P^+ = 0.5 \cdot (F_{R1} + F_{R2}) \leq 0.5n_oQ_u$$

$$P^- = 0.5A_{sr} f_{yr} \leq 0.5n_oQ_u$$
On the other hand, the peak effective flexural resistance of the bare steel beam differs under sagging, $M_{c,b}^+$, and hogging, $M_{c,b}^-$, bending. The slab restrains the top flange of the beam against local buckling, thereby resulting in a higher peak flexural resistance of the bare steel beam under sagging bending than under hogging bending. Particularly,

$$M_{c,b}^+ = 1.4M_{y,b}^+$$  \hspace{1cm} (14)  

$$M_{c,b}^- = 1.1M_{y,b}^-$$  \hspace{1cm} (15)  

The pre- and post-peak rotations of the bare steel beam are modified to account for the slab effects according to Elkady and Lignos.\textsuperscript{4} As discussed earlier, under hogging bending, the upward shift in the neutral axis expedites local buckling in the bottom flange of the beam. Accordingly, the pre- and post-peak rotations are reduced. This reduction is dependent on several factors such as the beam depth, the slab reinforcement ratio, and the degree of composite action under hogging bending, $\eta^-$. Due to the lack of measured data, only the latter is considered herein; thus,

$$\eta^- = n_o \frac{Q_u}{(A_{sr}f_{yr})}$$  \hspace{1cm} (16)  

Composite-steel beams compliant with EN-1998-1,\textsuperscript{1} shall have $\eta \geq 1.0$. For beams with low $\eta^-$, the compressive force transferred from the rebars to the steel beam is minor; therefore, the behavior, under hogging bending, is akin to that of the bare cross section. The following modifications are applied to the pre- ($\theta_p$) and post-peak ($\theta_{pc}$) rotations calculated as per Lignos and Krawinkler,\textsuperscript{48}

$$\theta_p^- = \begin{cases} 
0.5\theta_p, & \eta^- \geq 50% \\
0.9\theta_p, & \text{otherwise} 
\end{cases}$$  \hspace{1cm} (17)  

$$\theta_{pc}^- = \begin{cases} 
0.8\theta_{pc}, & \eta^- \geq 50% \\
\theta_p, & \text{otherwise} 
\end{cases}$$  \hspace{1cm} (18)  

Under sagging bending, the slab augments the pre- and post- peak rotations. The modified $\theta_p^+$ and $\theta_{pc}^+$ equations are adopted from Elkady and Lignos,\textsuperscript{4}

$$\theta_p^+ = 1.8\theta_p$$  \hspace{1cm} (19)  

$$\theta_{pc}^+ = 1.35\theta_{pc}$$  \hspace{1cm} (20)  

The rates of cyclic deterioration are modified to account for the differences in flexural strength degradation under sagging ($D^+ = 0.75$) and hogging bending ($D^- = 1.0$).

The beam-slab connection is modeled using a single zero-length element that is assigned a peak-oriented hysteretic response of the modified IMK deterioration model.\textsuperscript{47} Shear studs in a shear span are all lumped to a single zero length element; hence redistribution of forces within the studs is not taken into consideration. Moreover, friction at the interface of the beam flange and the slab is disregarded. The force-slip displacement of the shear studs is obtained through calibration with available cyclic push-out tests on shear stud connectors that account for the stress state in the slab.\textsuperscript{33} The ultimate resistance of the beam-slab connection when the slab is in compression, $Q_{c}^+$, and tension $Q_{t}^-$, is calculated according to Suzuki and Kimura.\textsuperscript{33} Figure 5 demonstrates a comparison between the simulated and measured shear resistance versus slip displacement of typical headed studs under cyclic loading. The comparisons indicate a relatively good match. Referring to Figure 5(A), the following parameters are used for 16 mm headed shear studs: the effective yield strengths $Q_{c}^+ = 90\%Q_{t}^+$, the pre-capping slip capacities $\Delta_{c,p}^+ = 8$ mm, the post-capping slip capacities $\Delta_{c,pc}^+ = 13$ mm and $\Delta_{c,pc}^- = 8$ mm, the ultimate slip capacities $\Delta_{u,p}^+ = 15$ mm, the strength and stiffness deterioration parameters, $\lambda_s = 80$ and $\lambda_k = 30$ and the deterioration rate parameters $D^\pm = 1.0$. The parameters for 19 mm headed shear studs in Figure 5(B) are obtained from El Jisr et al.\textsuperscript{34} Note that the all shear studs considered herein are ductile with $h_{sc}/d > 4.0$ ($h_{sc}$ and $d$ are the height and diameter of the shear studs, respectively).
3.1 Validation studies

The proposed macro-modeling approach for composite-steel beams is validated with available physical experiments. These include both interior and exterior subassemblies. The slab configurations are either symmetric or asymmetric. Moreover, the tests comprise profiled sheeting that is either parallel or perpendicular to the primary steel girder. Finally, full, and partially composite-beams are considered. The reported measured material properties are used for the validations. Figure 6(A) shows the configurations of the employed specimens along with the relevant force-deformation parameters used to obtain the hysteretic response per test.

Referring to Figure 6, the proposed modeling approach predicts well the flexural stiffness of the composite-steel beams. For fully composite beams, the effective flexural strength is within 10% of that achieved in the tests regardless of the orientation of the deck. While cyclic deterioration in flexural strength of composite-steel beams is generally predicted well, in cases where the orientation of the deck is transverse to the primary girder (see Figure 6E), the proposed model underestimates the cyclic deterioration in flexural strength. This is due to extensive spalling that often occurs at the slab-column interface.

The proposed model tends to slightly overestimate the slip demands in the composite slab as it does not account for the redistribution of the shear force between the studs. Referring to Figure 6(H), in subassembly L.P.C the maximum reported slip is 10% (14 mm vs. about 13 mm) lower than that obtained by the macro-model under sagging bending and about 25% lower under hogging bending (8 mm vs. 6 mm). Considering the simplified assumptions of the proposed model, it captures the shear stud slip reasonably well.

4 PROTOTYPE COMPOSITE-STEEL FRAME BUILDINGS

A 6-story prototype building of importance class II is designed according to the European seismic provisions. The building consists of 3-bay space composite-steel MRFs in the East-West (EW) direction, and two perimeter CBFs in the North-South (NS) direction (Figure 7A). A typical elevation view of the composite-steel MRF is shown in Figure 7(B). The composite-steel MRFs are designed with full-strength rigid stiffened end plate beam-to-column connections. The beam-to-column web panel zones are designed to remain elastic. Columns are spliced at stories 3 and 5 (see Figure 7B). The prototype building is designed for a reference peak ground acceleration, $a_{gr}$, of 0.22 g. This value is identical for different European sites: Sion in Switzerland, Aikaterini in Greece and Braila in Romania. As per EN-1998-1, all three sites have the same Type I design spectrum (behavior factor $q = 3$, ground Type D and viscous damping ratio, $\xi = 2\%$). The prototype building is designed with response spectrum analysis. All members are fabricated by S355J2 steel (i.e., nominal yield stress $f_{yn} = 355$ MPa).

The floor slab has a total thickness of 125 mm and consists of a 56 mm profiled sheeting (Cofrastra 56) with ribs oriented parallel to the MRF girders. The characteristic compressive strength of concrete is 30 MPa. The slab reinforcement ($f_{yr} = 500$ MPa) includes two layers of longitudinal eight rebars at 150 mm spacing, as well as 5\#10 seismic rebars placed within $h_c$ from the face of the column. Details of the composite cross section near the interior MRF column are shown in

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**FIGURE 5** Calibration of a cluster of four (A) 16 mm and (B) 19 mm shear studs subjected to cyclic loading (Data from Suzuki and Kimura)
Figure 7(C). Three different designs with full ($\eta = 100\%$) and partial ($\eta = 80\%$ and $50\%$) composite action are considered. For the designs with partially composite-steel beams, the 25% reduction in the shear stud capacity required by EN 1998-1 is intentionally waived. Moreover, a lower degree of composite action results in lower flexural demand on the columns as well as the panel zones. Nevertheless, the difference between the column sizes is negligible because their design is mostly governed by the interaction of bending with the axial force. As such, the same member sizes are employed in the three designs.

5 | NONLINEAR BUILDING MODELS

The three prototype buildings are modeled using two-dimensional (2D) nonlinear models in the EW loading direction. The models are developed in the open-source simulation platform OpenSEES.\(^{56}\) Half of each prototype building is modeled. The interior and exterior composite-steel MRFs (see Figure 7A) are linked in series via axially rigid truss elements as shown in Figure 8. Both frames should be considered in the model to properly simulate the destabilizing effects due to gravity on the overall building response. For the steel members, the expected yield strength is obtained by multiplying the nominal yield strength $f_{yn}$ by a material overstrength factor of 1.17 for S355J2 steel.\(^{57}\) Inelastic deformations in the
FIGURE 7  (A) Typical plan view of the prototype buildings; (B) elevation view of the six-story composite MRF; (C) details of the composite cross section at interior MRF columns

FIGURE 8  OpenSEES nonlinear model of the prototype building

Steel columns are modeled as proposed in Lignos et al. The panel zone rotational springs are modeled by assuming an asymmetric behavior to account for different effective depths of the panel zone due to the presence of the slab under sagging and hogging bending. Composite-steel beams are modeled with the proposed modeling approach discussed earlier. The flexural stiffness of the exterior composite-steel MRF composite beams is lower than that in the interior composite steel MRF beams owing to the larger effective width in the latter. Gravity loads are assigned to the composite-steel MRF columns based on the respective tributary area around the columns.

A fictitious leaning column is added to properly consider the effect of gravity loading, $P'$, from the 6 m edge spans of the prototype building that are not part of the tributary areas around the columns. The leaning column is connected with axially rigid truss elements to the two composite-steel MRFs as shown in Figure 8 and a vertical load of $P'/2$ is applied to the leaning column at each story.

Rayleigh damping is incorporated in the model according to the approach summarized in Zereian and Medina. A 2% critical damping ratio is applied at the first and third modes of the building model as recommended by PEER/ATC.
Figure 9 (A) Seismic hazard curves at the three design locations; (B) selected records for $T_R = 2475$ years, Sion

Table 1 Average spectral acceleration, mean magnitude and distance obtained from disaggregation analysis for the three sites at 475 and 2475 years return periods

| $T_R$ [years] | Sion (Switzerland) | Aikaterini (Greece) | Braila (Romania) |
|--------------|-------------------|---------------------|-----------------|
|              | $S_{a_{avg}}$ [g] | $\bar{M}$ | $R$ [km] | $S_{a_{avg}}$ [g] | $\bar{M}$ | $R$ [km] | $S_{a_{avg}}$ [g] | $\bar{M}$ | $R$ [km] |
| 475          | 0.11              | 6.4         | 26.9 | 0.18              | 7.1         | 55.2 | 0.15              | 7.1         | 99.2 |
| 2475         | 0.22              | 6.5         | 15.5 | 0.30              | 7.2         | 32.0 | 0.23              | 7.2         | 89.1 |

6 | SITE-SPECIFIC PROBABILISTIC SEISMIC HAZARD ANALYSIS AND GROUND MOTION SELECTION

Site-specific ground motion records are selected for nonlinear response history analysis (NRHA) that is discussed later on. For this reason, probabilistic seismic hazard analysis is performed for the three design sites. In brief, the ground motion prediction equation (GMPE) of Boore and Atkinson is employed. The area source model of SHARE is adopted and all sources within 200 km from the specific site are accounted for. The average shear wave velocity (150 m/s) corresponds to ground type D. Seismic hazard and disaggregation analyses are performed using OpenQuake.

Seismic hazard analysis is performed for each site for the average spectral acceleration, $S_{a_{avg}}$. This intensity measure (IM) is computed as the geometric mean of spectral acceleration ordinates over a period range of 0.4–4.4s with an increment size of 0.1s. Figure 9(A) shows the seismic hazard curves for the three considered sites. While $a_{gR}$ is the same at all three sites, the seismic hazard is quite different. Particularly, the design site at Braila is mostly controlled by more frequent earthquakes, whereas the one in Sion is strongly influenced by seismic events with a low probability of occurrence. At Aikaterini, the seismic hazard is higher than the other two design sites. The annual rate of exceedance is close to that in Braila for $S_{a_{avg}} < 0.15$ g and slightly higher than that in Sion for $S_{a_{avg}} > 0.4$ g.

Two earthquake record sets corresponding to $S_{a_{avg}}$ levels at two return periods ($T_R = 475$ years and 2475 years) are selected at each design site. The record selection utilizes the conditional spectrum (CS) with mean magnitude ($\bar{M}$) and distance from rupture ($\bar{R}$) as the target spectrum. The values of $\bar{M}$ and $\bar{R}$ for record selection are listed in Table 1 along with the $S_{a_{avg}}$ values at 475 and 2475 years return periods. No limitations on parameters $\bar{M}$, $\bar{R}$ and $v_{s30}$ of the selected records are considered; it is assumed that the spectral shape can define all the characteristics of the hazard at each site.

Each ground motion set consists of 40 records and was assembled from the NGA-West database. Figure 9(B) shows the ground motion set for Sion for a return period of 2475 years. Both pulse-like and non-pulse-like records are chosen and the scaling factor for record selection is limited to a maximum of 10.

7 | NONLINEAR STATIC ANALYSIS

Nonlinear static analysis based on a first mode lateral load pattern is conducted for the prototype buildings. Figure 10(A) shows the pushover curve of the three nonlinear building models in terms of first story base shear versus roof displacement. The base shear, $V_{base}$, is normalized with respect to the seismic weight of the modeled frames (i.e., seismic weight of half the prototype building), $W_s = 10869$ kN. The roof drift ratio, $\delta_r/s$, is calculated as $\delta_r/H$, where $\delta_r$ is the roof displacement and $H$ is the total height of the prototype building. Figure 10(A) suggests that the nonlinear static response of the prototype buildings is not significantly influenced by the degree of composite action.
FIGURE 10  (A) Comparison of pushover curves for the three prototype composite-steel frames; (B) derivation of performance parameters

TABLE 2  Prototype composite-steel MRFs first mode periods and performance parameters obtained from nonlinear static analysis

| Parameter | $\eta = 100\%$ | $\eta = 80\%$ | $\eta = 50\%$ |
|-----------|----------------|----------------|----------------|
| $T_1$ [s] | 1.70           | 1.66           | 1.62           |
| $\delta_{y,\text{eff}}$ [% rad] | 1.3            | 1.4            | 1.4            |
| $\Omega$ | 3.8            | 3.8            | 3.7            |
| $\mu_T$ | 4.5            | 4.2            | 4.2            |

The static overstrength factor, $\Omega$, and period-based ductility factor, $\mu_T$, are obtained from the pushover curves as shown in Figure 10(B). The system level parameters are summarized in Table 2. The same table includes the first mode periods of each nonlinear building model based on standard eigenvalue analyses. Note that the first mode period increases with decreasing degree of composite action due to the decrease in the flexural stiffness of the composite-steel beams. Nevertheless, the difference between periods is less than 5%.

Referring to Table 2, the high values of the global yield drift are commonly observed in buildings featuring space MRFs. Unlike in buildings with only perimeter MRFs, beams that are part of space-frame systems tend to be of shallower depths (i.e., beam depths less than 500 mm) and have larger span-to-depth ratios (>8.0). For the prototype composite-steel MRF, the shear span-to-depth ratio varies between 8 in the bottom stories and 12.5 in the top stories.

Referring to Figure 10(B), the static overstrength factor, $\Omega$, is computed as the ratio between the maximum base shear, $V_{\text{max}}$, and the design base shear obtained through modal response spectrum analysis, $V_{\text{design}}$ [i.e., $V_{\text{max}}/V_{\text{design}}$]. Table 2 shows that is close to 4.0 regardless of the employed degree of composite action. The high $\Omega$ values are attributed to (i) the steel material overstrength; (ii) the discrete choice of beam and column cross sections; (iii) oversizing the beams and columns to resist gravity loads; and most importantly; and (iv) the drift control requirements. The above matters are elaborated in Osteraas and Krawinkler based on field reconnaissance and corroborating structural analyses of actual buildings after the 1985 earthquake in Mexico city. According to Elghazouli, a typical MRF designed according to EN 1998-1 can have considerable overstrength. European drift requirements are more stringent than those in the US. Moreover, the stability coefficient is fairly conservative, which leads to considerable overstrength. Haselton et al. noted that beams in space MRFs are designed for higher gravity loads than those in perimeter MRFs, which generally results in higher overstrength. In general, further research is required to refine the EN 1998-1 provisions through the benchmarking of the seismic performance of composite-steel MRFs.

Table 2 summarizes the period-based ductility, $\mu_T$, defined as the ratio between the roof drift at 20% drop in base shear and the roof drift ratio at yield, $\delta_{y,\text{eff}}/\delta_{80\%V_{\text{max}}}$ (see Figure 10B). Figure 11 shows the progression of the collapse mechanism of the prototype composite-steel MRF ($\eta = 100\%$) along its height. The three-story collapse mechanism, evident at a roof drift ratio of 8% rad, is attributed to the high strong-column-weak-beam (SCWB > 2) ratio in the first two stories. The column section sizes in these stories are governed by the moment and axial force demands. Despite the intrinsic value and simplicity of nonlinear static analysis, it cannot be used for the earthquake-induced collapse risk assessment of the composite-steel MRFs due to the associated limitations of the simplified analysis technique. For the above reasons, nonlinear response history analyses are conducted as discussed in the subsequent sections.
8 | COLLAPSE RISK ASSESSMENT

This section investigates the collapse risk of the prototype buildings. The dependence on the design site and the degree of composite action of the composite-steel beams are examined. To this end, incremental dynamic analysis (IDA) is conducted. The ground motion records discussed earlier are scaled incrementally with respect to the 5%-damped average spectral acceleration, $S_{avg}$, until dynamic instability occurs; that is, until a story, or a number of stories, deforms laterally and the story shear resistance becomes zero due to second order effects accelerated by structural component deterioration. This definition of structural collapse is consistent with prior shake table experiments in steel MRFs. Herein, the adopted IM is the average spectral acceleration because of its sufficiency and efficiency relative to other commonly used IMs.

Figure 12(A) shows the IDA curves for the prototype building ($\eta = 100\%$) in Sion. The $S_{avg}$ is plotted versus the absolute maximum story drift ratio ($SDR_{max}$). The median, 16th, and the 84th percentiles are also superimposed in the figure. Referring to Figure 12(B), the peak floor acceleration ($PFA$) caps as the building response becomes inelastic. Nevertheless, the median $PFA$ is high ($\sim 2.0$ g) due to the high system overstrengh (see Table 2).

The collapse fragility curves are obtained by fitting the cumulative probabilities of collapse corresponding to the 40 collapse intensities with a lognormal cumulative distribution. Figure 13 shows the dependence of the collapse fragility curves on the degree of composite action of the composite-steel beams and the design site. The median collapse intensities, $\mu S_{avg,c}$, and lognormal standard deviations, $\sigma_{ln S_{avg,c}}$, of the collapse intensities are summarized in Table 3 for the analyzed buildings. Referring to Figure 13(A), the collapse capacity of the prototype buildings is somewhat influenced by the degree of composite action of the composite-steel beams. The collapse capacity decreases with decreasing degree of composite action. This decrease is explained by the lower lateral stiffness of the composite-steel MRFs with lower degree of composite action. Consequently, global P-Delta effects are more prominent for lower degrees of composite action. However, the difference in the median collapse capacity is minor (within 15%) as shown in Table 3.

Figure 13(B) depicts the dependence of the collapse capacity of the prototype buildings on the design site. The choice of the ground motion record set influences their collapse capacity. Nevertheless, the use of $S_{avg}$ ensures that the...
record-to-record variability is not significant and hence the difference in median collapse capacity is around 15%. This is also reflected in the low $\sigma_{\ln S_{aav}}$ values, which are summarized in Table 3.

The collapse risk of the prototype buildings is evaluated through the mean annual frequency of collapse, $\lambda_c$. This is computed by integrating the collapse fragility curves over the corresponding hazard curves for a given design site,

$$\lambda_c = \int_0^\infty (P_c | S_{aav} = x) \cdot dS_{aav} (x)$$

(21)

In which, $(P_c | S_{aav} = x)$ is the probability of collapse given $S_{aav}$ equals to a seismic intensity $x$, obtained from the collapse fragility curve; and $dS_{aav} (x)$ is the mean annual rate of $S_{aav}$ exceeding $x$ obtained from the seismic hazard curve. The integral is solved numerically according to Eads et al. The corresponding probability of collapse, $P_c (50$ years), of the prototype buildings is calculated over a period of a 50-year building life expectancy, by assuming that earthquakes follow a Poisson distribution over time. Table 4 summarizes the values for all the examined cases. The $P_c (50$ years) is considerably lower than the 1% limit specified in ASCE/SEI 7–16 regardless of the design site. This is because the annual rate of seismic hazards associated with the median collapse intensities is practically insignificant ($\lambda < 10^{-5}$) compared to other design sites from around the world, such as southern California. Other reasons for the relatively low annualized probabilities of structural collapse relate to the stringent EN 1998-1 lateral drift requirements that also contribute to appreciable system-level overstrength (see Table 2) as well as the stability requirements with regard to global P-Delta effects. Of interest from a repairability standpoint are potential repair actions in designs featuring beams with partial degrees of composite action, given that the 25% reduction in the shear stud resistance has been waived.
9 PEAK SLIP HAZARD CURVES FOR BEAM-SLAB CONNECTIONS AND DAMAGE ASSESSMENT

The proposed macro-model approach allows for the explicit quantification of the peak slip demands of beam-slab connections along the building height, $\Delta_{sp}$. Figure 14 shows $\Delta_{sp}$ along the building height based on single-stripe analysis at a design-basis earthquake (DBE) for Aikaterini. The slip demands are generally higher in the lower stories. The slip demands can be quantified in the context of the performance-based earthquake engineering framework through the development of $\Delta_{sp}$ hazard curves. Potential cyclic deterioration in the shear resistance of beam-slab connections may result in high slip demands. Therefore, the $\Delta_{sp}$ hazard curves provide an effective means of quantifying the peak slip demands within the framework of performance-based earthquake engineering, and hence the damage in the beam-slab connections at return periods of interest to the engineering community. This is achieved by computing a mean annual rate of exceeding a slip value, $\lambda_{\Delta_{sp}}$, for any given return period that is associated with a seismic event (e.g., design basis and/or serviceability earthquakes). This relates to the concept of EDP hazard curves as described in prior-related work. For simplicity, $\Delta_{sp}$ is taken as the maximum slip demand on all beam-slab connections in the prototype buildings.

The beam-slab connection slip hazard curves are developed according to Krawinkler and Miranda. The slip demands obtained directly by the proposed model (see Figure 15A for a single simulation) are used for this purpose. The mean annual rate of $\Delta_{sp}$ exceeding $y$ is computed as follows,

$$\lambda_{\Delta_{sp}} (y) = \int P[\Delta_{sp} > y | S_{a_{avg}} = x] \cdot d\lambda_{S_{a_{avg}}} (x) \tag{22}$$

in which, $P[\Delta_{sp} > y | S_{a_{avg}} = x]$ is the probability of $\Delta_{sp}$ exceeding $y$ given $S_{a_{avg}}$ equals to a seismic intensity $x$; and $\lambda_{S_{a_{avg}}} (x)$ is the mean annual rate of $S_{a_{avg}}$ exceeding $x$ obtained from the corresponding seismic hazard curve. The probability $P[\Delta_{sp} > y | S_{a_{avg}} = x]$ is computed according to Equation 23. It is assumed that at the collapse intensities, $P(\Delta_{sp} > y) = 1$.

$$P \left[ \Delta_{sp} > y \mid S_{a_{avg}} = x \right] = \left( P_{c} \mid S_{a_{avg}} = x \right) + \left[ 1 - \left( P_{c} \mid S_{a_{avg}} = x \right) \right] \cdot \left( 1 - \Phi \left( \frac{\ln y - \ln \mu_{\Delta_{sp}}}{\sigma_{\ln \Delta_{sp}}} \right) \right) \tag{23}$$

In which, $(P_{c} \mid S_{a_{avg}} = x)$ is the probability of collapse given $S_{a_{avg}} = x$, and is obtained by the collapse fragility curves; $\Phi$ is the lognormal cumulative distribution function; $\ln \mu_{\Delta_{sp}}$ is the natural logarithm of the median of $\Delta_{sp}$ for the non-collapse cases; and $\sigma_{\ln \Delta_{sp}}$ is the standard deviation of the natural logarithm of $\Delta_{sp}$ for the non-collapse cases. Referring to Figure 15(B), the $\Delta_{sp}$ values at $S_{a_{avg}}$ obtained from the IDA curves are fitted with a lognormal distribution. The mean annual rate of exceedance, $\lambda_{\Delta_{sp}} (y)$ is then obtained by numerical integration. Figure 15(C) shows the $\Delta_{sp}$ hazard curves for two of the analyzed buildings with partially composite beams at the three design sites. The $\Delta_{sp}$ values are generally higher in composite-steel MRFs in Braila and Aikaterini than in Sion for annual rates $\lambda_{\Delta_{sp}} < 10^{-4}$. This is because the seismic hazard in the first two design sites is characterized by more frequently occurring seismic events. Conversely, Sion is characterized by seismic events with low probability of occurrence; hence, slip demands of beam-slab connections are lower than those determined in Braila.

The median $\Delta_{sp}$ values at three representative return periods ($T_R = 72, 475$ and $2475$ years) are summarized in Table 5. At a return period of 72 years (i.e., service level earthquake), the median $\Delta_{sp}$ values are less than 1 mm regardless of the
FIGURE 15 (A) Peak slip demands, $\Delta_s$, shown for the most critical beam-slab connection along the building height (Sion, $\eta = 80\%$); (B) distribution of $\Delta_s$, for selected $S_{a_{avg}}$ values (Sion, $\eta = 80\%$); (B) developed $\Delta_s$, hazard curves for the two prototype buildings with partially composite-steel beams at the three design sites; (D) typical fragility functions for the beam-slab connections.

TABLE 5 Comparison between the median $\Delta_s$ [mm] values of the prototype composite-steel MRFs at three representative return periods

| $\eta$ [%] | Sion (Switzerland) | Aikaterini (Greece) | Braila (Romania) |
|------------|---------------------|---------------------|------------------|
|            | 80      | 50      | 80     | 50     | 80    | 50    |
| $T_R = 72$ years | 0.3 | 0.4 | 0.5 | 0.8 | 0.6 | 0.9 |
| $T_R = 475$ years | 0.7 | 1.3 | 1.4 | 2.7 | 1.7 | 2.5 |
| $T_R = 2475$ years | 2.5 | 4.0 | 4.5 | 6.2 | 3.5 | 4.7 |

design location, and the degree of composite action. At return periods associated with a design-basis earthquake (DBE), the median $\Delta_s$ values are less than 2 mm for $\eta = 80\%$ and 3 mm for $\eta = 50\%$. On the other hand, at return periods associated with a maximum considered earthquake (MCE), the median $\Delta_s$ values vary between the three sites and range between 2.5 and 4.5 mm for $\eta = 80\%$, and 4.0 to about 6.0 mm for $\eta = 50\%$. In order to better comprehend the significance of the above values, these are correlated with slip-based fragility functions that estimate the likelihood of reaching or exceeding discrete damage states in beam-slab connections (see Figure 15D). These functions were derived from cyclic push-out tests that account for the stress state in the slab. Four damage states are considered: (i) DS1, light cracking in the concrete; (ii) DS2, extended cracking in the concrete accompanied with stud yielding and/or crushing near the base of the shear studs; (iii) DS3, low-cycle fatigue microcracking in the shear studs and extensive cracking in the concrete; and (iv) DS4, loss of shear load carrying capacity of the beam-slab connection.

Referring to Figure 16(A), for $\eta = 80\%$, beam-slab connections are very likely to experience light cracking in the slab (DS1) or no damage (30–95%), for seismic events with a 72-year return period (i.e., 50% probability in 50 years). For a 475-year return period associated with a DBE, the probability of extended cracking in the slab and stud yielding or concrete crushing near the base of the studs (DS2), is negligible for Sion and less than 10% for Aikaterini and Braila. Figure 16(B) shows the damage in beam-slab connections at the three design sites for $\eta = 50\%$. For a 72-year return period, the damage is minimal as in the design with $\eta = 80\%$. For a 475-year return period, the probability of extended cracking in the slab and stud yielding or concrete crushing near the base of the studs is up to 20% for Aikaterini and Braila. On the other hand, extensive damage in the slab and low-cycle fatigue microcracking (DS3) are unlikely to occur. The probability of the
loss of shear load carrying capacity in the beam-slab connections is negligible regardless of the design site and degree of composite action. The above findings demonstrate that while the 25% reduction in capacity of shear studs was disregarded in the design, the integrity of the beam-slab connections in shallow partially composite-steel beams \((d_b < 500 \text{ mm})\) is maintained for a DBE event.

10 | LIMITATIONS

It is worth mentioning that although the proposed macro-modeling approach can simulate the global frame and local component behaviors, it comes with several limitations. The present model cannot capture the following phenomena: (i) the redistribution of the demands on the shear studs; (ii) the slip profile in the beam-slab connections along the length of the beams; (iii) uplift of the shear studs; (iv) the effects of framing action, which has been shown to increase the plastic rotation capacity of the beams\(^{43,78}\); (v) fracture of the beam-to-column connection; and (vi) concrete spalling at the beam-column interface. Spalling of concrete leads to a loss of the bearing mechanism, which in turn decreases the shear demands on beam-slab connections at lateral drift demands higher than 3% rads. Furthermore, since the employed slip-based fragility functions were derived from a database of cyclic push-out tests, they tend to overestimate the expected damage in the beam-slab connections\(^{25,34}\). Hence, the probabilities of damage presented in this paper are upper-bound probabilities. The proposed modeling approach is valid for 2D nonlinear response history analysis. Further enhancements, that account for three-dimensional effects and the influence of transverse beams on the global frame and local component seismic behavior, should be carefully considered.

11 | CONCLUSIONS

This paper proposes a nonlinear macro-model for simulating the deteriorating behavior of fully restrained beam-to-column connections with composite-steel beams. The model incorporates the effects of the floor slab on the flexural strength through mechanics-based constitutive laws that account for damage in the slab. Furthermore, the model can capture the asymmetric hysteretic response and flexural strength deterioration that occurs in composite-steel beams. The proposed macro-model, which was thoroughly validated with available test data, can explicitly predict the slip demands in beam-slab connections.

System-level nonlinear static analysis as well as incremental dynamic analysis (IDA) through collapse are conducted to benchmark the seismic risk of prototype buildings with composite-steel MRFs as their primary lateral load resisting system. These are designed in accordance with EN 1998-1\(^{2}\) for three different European sites. The collapse risk of the prototype buildings is computed and its dependence on the degree of composite action and design sites is explicitly quantified. The results from nonlinear building simulations are also employed to develop novel peak slip hazard curves for beam-slab connections that enable the assessment of their integrity at characteristic return periods of interest to the engineering profession. While benchmarking the collapse risk relates to life safety in rare seismic events, the proper quantification of peak slip demands in beam-slab connections relates to structural repair actions in the aftermath of more frequently occurring earthquakes. The main findings are summarized as follows:
1. Results from nonlinear static analysis suggest that the static overstrength factor, $\Omega$, is close to 4 and the period-based ductility, $\mu_T$, ranges between 4 and 4.5. The high value of $\Omega$ is due to the lateral drift requirements in the current EN 1998-1.\(^1\)

2. The collapse assessment of the examined buildings indicates that their median collapse intensity is somewhat sensitive to the degree of composite action. While the design sites have the same design spectrum according to the current EN 1998-1 seismic provisions,\(^1\) the median collapse capacity between the examined cases is site dependent due to differences associated with the seismic hazard curves of each design location.

3. The collapse risk of the three prototype buildings at all design sites in terms of annualized probabilities of collapse over a 50-year building life expectancy is considerably lower than the 1% limit specified in ASCE/SEI 7–16.\(^1\) The highest values are noted in Aikaterini/Greece, $\eta = 50\%$ and are equal to 0.03%. The relatively low annualized probabilities of collapse are mostly attributed to the strict lateral drift limits according to EN 1998-1.\(^1\)

4. The peak slip demands in beam-slab connections are strongly influenced by seismic events with a high probability of occurrence. The developed beam-slab connection peak slip hazard curves suggest that for $\eta = 80\%$, the slip demands at a return period associated with a DBE event ($T_R = 475$ years) are up to 2 mm. On the other hand, for $\eta = 50\%$, the slip demands can reach up to 3 mm for the same return period.

5. In prototype buildings with partially composite-steel beams ($\eta = 80\%$ and $\eta = 50\%$), the beam-slab connections experience minimal or no damage for frequently occurring seismic events (i.e., $T_R = 72$ years). For DBE events, the probability of damage in beam-slab connections is mainly characterized by light cracking in the slab (>70%) for $\eta = 80\%$. For the same degree of composite action, no extensive damage or loss of shear-load carrying capacity of the beam-slab connections is expected. On the other hand, for $\eta = 50\%$, the likelihood of shear stud yielding and/or crushing may reach up to 20% for seismic events with a return period of 475 years. Moreover, the probability of extensive damage in the slab and low-cycle fatigue microcracking is less than 10% regardless of the design site.

The findings suggest that for composite-steel MRF designs with shallow steel beams ($d_b < 500$ mm), the 25% reduction in the capacity of shear connectors, which is recommended by current seismic provisions,\(^1,2\) is not substantiated. However, additional experimental and system-level studies with various slab configurations should be conducted to further support this finding.

This paper provides methodological developments for modeling composite-steel MRFs that can be utilized to refine current and future seismic design standards (e.g., EN 1998-1).\(^1\) Further system-level nonlinear simulations can be used to benchmark the seismic performance of composite-steel concrete buildings from the onset of earthquake damage to structural collapse, thereby leading to more effective seismic designs.

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DATA AVAILABILITY STATEMENT
The source code for the proposed macro-model along with the nonlinear models, ground motion records and simulation results are made publicly available in an online repository (https://doi.org/10.5281/zenodo.5979732).

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