Nonlinear dynamic analysis of steel-concrete composite frame structures under earthquake excitation

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ABSTRACT: Composite beams with deformable shear connection were specifically introduced as an extension of conventional monolithic beam models for the analysis of steel-concrete composite (SCC) structures in which the flexible shear connection allows development of partial composite action influencing structural deformation and distribution of stresses. The use of beams with deformable shear connection in the analysis of frame structures raises very specific modeling issues, such as the characterization of the cyclic behavior of the deformable shear connection and the assembly of composite beam elements with conventional beam-column elements. In addition, the effects on the dynamic response of SCC frame structures of various factors such as the shear connection boundary conditions are still not clear and deserve more investigation. The object of this work is to provide deeper insight into the nonlinear seismic response behavior of SCC frame structures and how it is influenced by various modeling assumptions.

1 INTRODUCTION

Among the various models available for the analysis of composite structures (Spacone & El-Tawil 2004), frame models allow to obtain significant information at reasonable computational cost compared to more sophisticated two-dimensional (plate/shell) and three-dimensional (solid) finite element (FE) models. Even if frame elements can only account approximately for local effects (e.g., shear lag, local instabilities in the compressed portion of the steel beam, cracking and crushing of concrete), good test-analysis correlation results were obtained by a number of researchers (e.g., Liew et al. 2001, Kim & Engelhardt 2005) for quasi-static tests and global response quantities. As an extension of conventional monolithic beam models, beams with deformable shear connection were specifically introduced and adopted for the analysis of SCC beams. Flexible shear connectors allow development of partial composite action (Viest et al. 1997, Oehlers & Bradford 2000), influencing structural deformation and distribution of stresses under service and ultimate load conditions. Furthermore, the shear connection can be responsible for collapse, e.g., when partial shear connection design is adopted, connectors fail due to limited ductility. Thus, a composite beam model with deformable shear connection has some important advantages over the common Euler-Bernoulli monolithic beam model, i.e., it allows a more accurate modeling of the structural behavior, provides information on the slab-beam interface slip and shear force behavior, permits to evaluate the effects of the interface slip on stress distribution, and enables to model damage and failure of the connectors. Up to date, applications of beam elements with deformable shear connections to the analysis of SCC frames have mainly been limited to quasi-static behavior, e.g., recent work by Dissanayake et al. (2000), Ayoub & Filippou (2000), and Salari & Spacone (2001). On the other hand, there is limited experience on nonlinear dynamic analysis of SCC frames based on beam elements with deformable shear connection (Bursi et al. 2005). Furthermore, some modeling aspects deserve further investigation. In fact, the use of beams with deformable shear connection in the analysis of frame structures raises very specific modeling issues, such as the characterization of the cyclic behavior of the deformable shear connection and the assembly of composite beam elements with conventional beam-column elements. In addition, the influence of various factors (e.g., shear connection boundary conditions, mass distribution between the two components of the composite beam) on the dynamic response of SCC frame structures needs to be better understood through a systematic parametric study.

The objective of this work is to provide deeper insight into nonlinear dynamic analysis results of SCC structures and how different modeling assumptions affect these results. For this purpose, a materially-nonlinear-only FE formulation for static and dynamic analysis of SCC structures using displacement-based locking-free elements with deformable shear connection (Dall'Asta & Zona 2002) is employed. Realistic uni-axial cyclic constitutive laws are adopted for the steel and concrete materials of the beams and columns and for the shear connection. Nonlinear dynamic seismic analysis results of two-dimensional moment resisting frames made of steel columns and composite beams are provided. These results and their discussion focus on: (i) the influence of partial composite action on the dynamic nonlinear analysis of SCC frames; (ii) the effects of different modeling assumptions related to SCC structures.

2 MODELING AND ANALYSIS OF STEEL-CONCRETE COMPOSITE FRAMES

2.1 *Beam model with deformable shear connection*

The formulation for two-dimensional beams with deformable shear connection is based on the Newmark et al. (1951) model in which (i) Euler-Bernoulli beam theory (in small deformations) applies to both components of the composite beam, and (ii) the deformable shear connection is represented by an interface model with distributed bond allowing interlayer slip and enforcing contact between the steel and concrete components.

2.2 *Finite Element Formulation*

The present study makes use of a simple and effective two-dimensional 10 nodal degrees-of-freedom (DOFs) displacement-based SCC frame element with deformable shear connection (Dall'Asta & Zona 2002). This element was proven to produce accurate results even for local response quantities (e.g., section deformations, section stress resultants, shear force distribution at the steel-concrete interface, etc.) provided that a sufficiently refined mesh is adopted (Dall'Asta & Zona 2004c). The same element was employed for finite element response sensitivity analysis of SCC structures (Zona et al. 2005) under both monotonic and cyclic loading conditions. Its response results were validated through test-analysis comparison studies (Zona et al. 2005). Results and observations provided in the present paper are, however, not restricted to the specific finite element adopted.

In the finite element adopted, the concrete slab and steel beam components have their own user-defined reference system allowing proper selection of the position of the axial DOFs to be used in the FE assembly of beam and column elements. Typical choices are: reference systems located at the centroid of each component; reference systems both located at the interface of the two components. Choice of the reference system does not influence the precision of the element, since the element shape functions in different reference systems are consistent so as to avoid the eccentricity issue and slip locking problems (Dall'Asta & Zona 2004b). Another useful feature of the FE formulation adopted is an internal constraint that can be introduced at each beam end independently. This internal constraint enforces zero slip at the beam end at which it is applied and can be used in modeling many real situations.

2.3 *Modeling of inertia and damping properties*

In this study, the inertia properties are modeled via translational (horizontal and vertical) masses lumped at the DOFs of the frame elements' external nodes. A lumped mass matrix formulation is preferred to a consistent mass matrix formulation for several reasons: (i) using a lumped mass matrix formulation, the inertia properties of the finite element model are independent of the finite element types employed, and thus the same structure mass matrix is obtained using displacement-based, force-based or mixed-formulation frame elements; (ii) a lumped mass matrix formulation yields a diagonal mass matrix at the structure level and entails significant computational advantages (over a fully populated consistent mass matrix) for calculations involving the inverse of the mass matrix such as in eigen-analysis and in explicit time integration methods; (iii) the use of consistent mass matrices at the element level presents some additional complications when internal DOFs need to be condensed out at the element level, as in the case of the locking free composite frame element used herein.

Information about damping properties of SCC frame structures inferred from experimental dynamic data is very limited (Bursi & Gramola 2000, Dall'Asta et al. 2005). This study assumes Rayleigh-type proportional viscous damping (Chopra 2001). The Rayleigh damping matrix used herein is proportional to the mass matrix and initial stiffness matrix.

2.4 *Hysteretic Modeling of Structural Materials and Shear Connection*

The selected constitutive model for the steel material (reinforcement and beam steel) is the uni-axial Menegotto & Pinto (1973) constitutive model, a computationally efficient nonlinear smooth law able to model both kinematic and isotropic hardening (Filippou et al. 1983) and Baushinger's effect, which are typical of the cyclic behavior of structural steel materials. Further details on the model and its numerical implementation can be found in (Barbato & Conte 2006), where the model is extended for finite element response sensitivity computation. A typical cyclic response is shown in Figure 1.

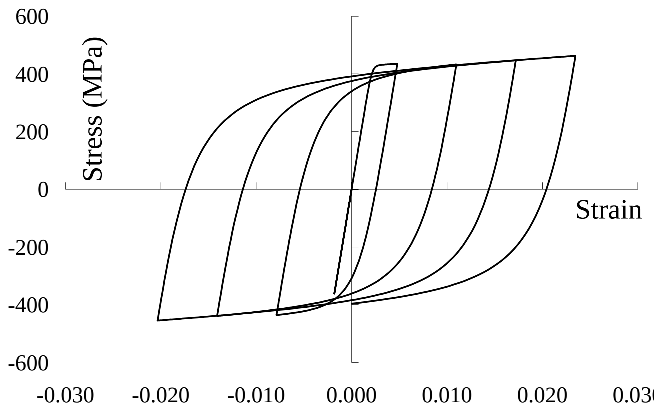

Figure 1. Menegotto-Pinto material constitutive model for structural steel: typical cyclic stress-strain response

The selected constitutive law for the concrete material is a uniaxial cyclic law with monotonic envelope defined by the Popovics-Saenz law (Balan et al. 1997, 2001). The details of the formulation of this constitutive model and related material parameters can be found in (Zona et al. 2006). A typical cyclic response is shown in Figure 2.

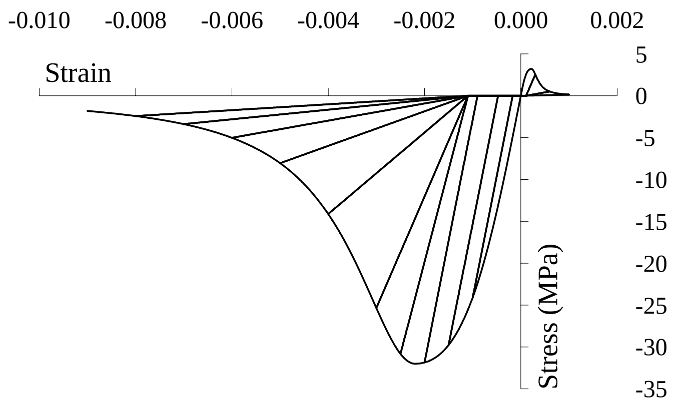

Figure 2. Hysteretic concrete material model: typical cyclic stress-strain response

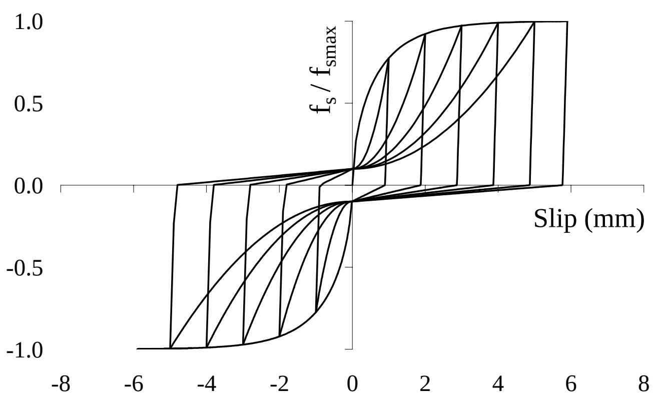

Figure 3. Hysteretic model of shear connection: typical cyclic shear force-slip response

The constitutive law used for the shear connectors is a slip-force cyclic law with monotonic envelope given by the Ollgaard et al. law (1971). The cyclic response of the shear connectors is a modified version of the model proposed by Eligenhausen et al. (1983). The details of the formulation of this constitutive model and related material parameters can be found in (Zona et al. 2006). A typical cyclic response is shown in Figure 3.

3 DYNAMIC RESPONSE SIMULATION OF SCC FRAME STRUCTURES

3.1 *Description of the SCC frames analyzed*

The basic testbed SCC frame structure considered in this section is a realistic 5-story 2-bay moment resisting frame made of steel columns and composite beams (Fig. 4). Each bay has a span of 5.00 m and each story has a height of 3.00 m. The steel columns are made of European HEB300 wide flange beams, while the composite beams are made of steel European IPE270 I-beams connected by means of stud connectors to a 100 mm thick concrete slab with an effective width estimated at 800 mm (kept constant along the beam), top and bottom reinforcements of 400 mm^2 and a concrete cover of 30 mm (Fig. 5).

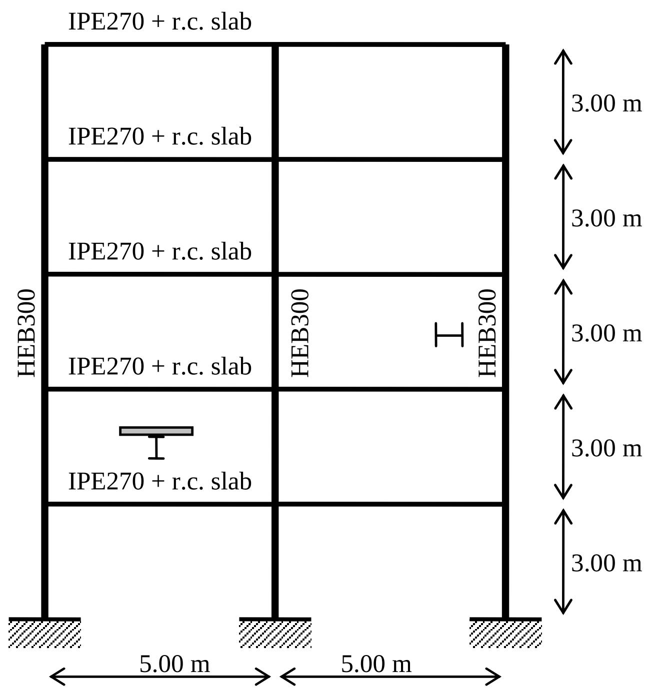

Figure 4. Configuration of testbed SCC frame structure analyzed

This SCC frame was designed according to Eurocode 4 (CEN 2004a) to resist the static loads (composite cross section self weight = 2.36 kN/m, permanent load G = 16 kN/m, and live load Q = 8 kN/m with G and Q uniformly distributed along the composite beams) and seismic forces evaluated using response spectrum analysis with peak ground acceleration = 0.35g, Type 1 spectrum of Eurocode 8 (CEN 2004b), modal damping ratio = 0.05, soil B, and behavior factor q = 3. A full shear connection (i.e., the ultimate strength of the composite section is not affected by the shear connectors) was designed using the plastic approach of Eurocode 4 (CEN 2004a). For the sake of simplicity, the shear connection strength was taken as constant along all composite beams.

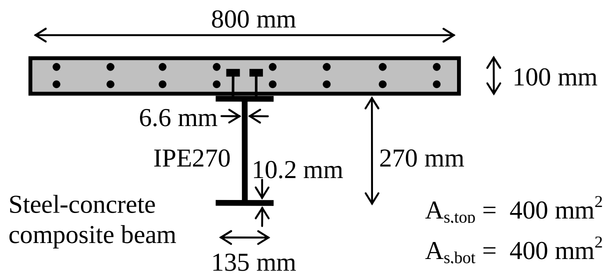

Figure 5. Composite beam cross-section definition of the test-bed SCC frame structure analyzed

All finite element models adopted in the response simulations illustrated use the same spatial discretization: four 10-DOF composite frame elements of equal length for each beam (between two adjacent columns) and two conventional displacement-based Euler-Bernoulli (monolithic) frame elements for each column (between two adjacent floors).

The inertia properties are modeled using lumped (horizontal and vertical) translational masses representing the mass of beams and columns, and the mass equivalent to both the permanent load G and the live load Q. The mass equivalent to the self-weight of the SCC beams is distributed between the slab and beam DOFs according to the actual mass of the steel beam and concrete slab components. The mass equivalent to the permanent load G is considered evenly divided between the slab and steel beam components. The mass equivalent to the live load Q is assigned entirely to the slab DOFs.

Three different slip boundary conditions are considered: (i) slip restrained at every beam-column joint (i.e., the relative slip between slab and steel beam is prevented at the face of every column); (ii) slip restrained at the central beam-column joints only (i.e., relative slip between slab and steel beam is prevented at the face of the central column only); (iii) slip free at each joint (i.e., due to sufficient space between columns and slabs, relative slip between slab and steel beam is not prevented). The slip constraint condition is not applied to the SCC beams at the roof level where the slab is free to slip. In this way, the present study aims at describing and analyzing the effects of different realistic modeling options (in terms of slip constraint conditions) at the joints between SCC beams and steel columns.

In addition, a finite element frame model with conventional Euler-Bernoulli monolithic beams (i.e., full shear interaction and full shear connection) was included in this study for comparison purposes.

3.2 *Nonlinear earthquake response analysis*

After quasi-static application of a vertical distributed load of 26.36 kN/m representing self weight, permanent and live loads, four nonlinear dynamic analyses were carried out for each frame model considered by using two ground motion accelerograms and two different levels of viscous damping in the structure. The two historic earthquake accelerograms used as base excitation are: (i) the 1994 Northridge earthquake recorded at the Pacoima Dam station with a peak ground acceleration (PGA) of 1.585g, corresponding to a return period of about 180 years (at the recording site), and (ii) the 1979 Imperial Valley earthquake recorded at the Bonds Corner station with PGA = 0.775g, corresponding to a return period of about 40 years (at the recording site). Rayleigh damping was assumed with a specified damping ratio of ξ at both the first and third vibration modes of the structure. This study uses the constant average acceleration method and the corresponding set of nonlinear algebraic equations is solved iteratively using Newton's method (Chopra 2001). A constant time step Δt = 0.005s was used for the numerical integration of the equations of motion in all the nonlinear dynamic analyses performed. Due to space limitation, in the sequel only selective results are presented.

The response of the frame model with rigid shear connection ($\psi = \infty$) is compared to the response of the frame model characterized by deformable shear connection (ψ = 1.0, full shear connection) and slip restrained at the central beam-column nodes only (frame model 10TC). In Fig. 6, time histories for the horizontal displacement of the left column at the roof level (z = 15 m) are reported. The differences between the two models are clearly noticeable. Taking the response of the monolithic frame as reference, the difference in magnitude of the positive and negative peaks is +0.2% and +31.5%, respectively. The maximum absolute difference over the duration of the earthquake is 129.9 mm, and the average absolute difference is 22.5 mm. It is observed that in the case of the monolithic frame, the displacement response oscillates around the static displacement response due to gravity loads only, which is not the case for the frame with deformable shear connection. To evaluate the effect of a different seismic excitation, Fig. 7 presents the roof displacement responses for the same models (monolithic frame and frame 10TC) as in Fig. 6 subjected to the (less intense) Imperial Valley earthquake. Again, the difference in the two model responses is evident (difference in magnitude of the positive and negative peaks = -1.5% and +40.8%, respectively, maximum absolute difference = 108.6 mm, average absolute difference = 51.2 mm).

Fig. 8 represents graphically the interstory drift and shear demands for the frame model with monolithic beams and frame model 10TC subjected to the Northridge earthquake input. It is found that response simulation of SCC frame structures modeled using frame elements with deformable shear connection (as compared to monolithic frame elements) leads to lower seismic demand in terms of interstory shears. As expected, the interstory shear demand increases with the overall stiffness of the frame model. Thus, response simulation of SCC frame structures modeled using frame elements with deformable shear connection (as compared to monolithic frame elements) leads to larger seismic demand in terms of floor displacements and interstory drifts. This conclusion is also corroborated by other analysis results (based on other combinations of modeling parameters/assumptions and the Imperial Valley earthquake input) not presented here due to space limitation.

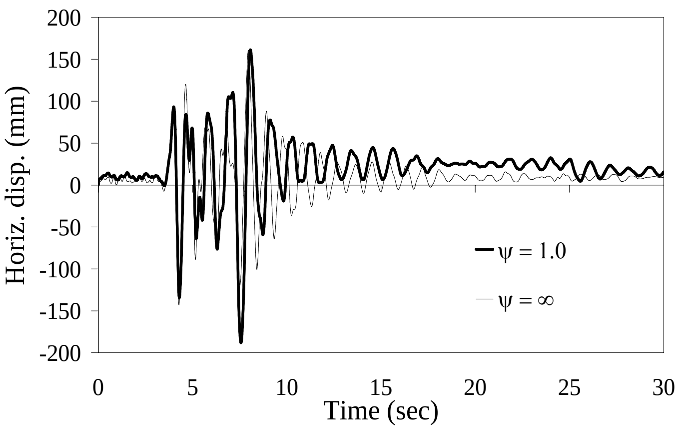

Figure 6. Horizontal displacement of left column at roof level: effect of deformable shear connection (Northridge seismic input)

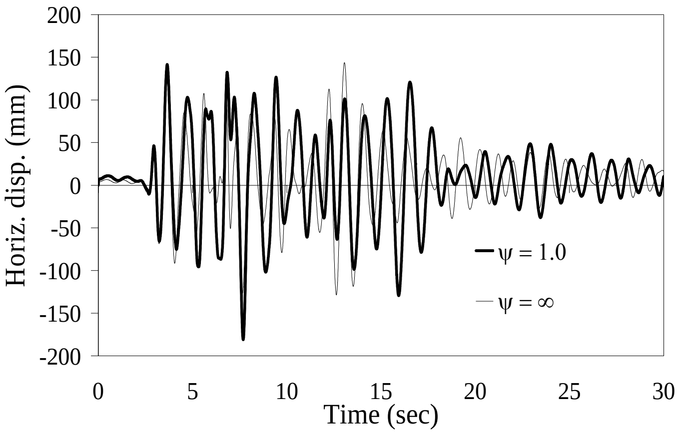

Figure 7. Horizontal displacement of left column at roof level: effect of deformable shear connection (Imperial Valley seismic input)

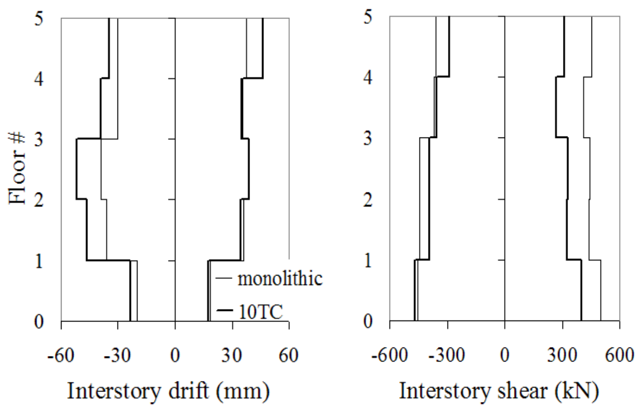

Figure 8. Interstory drift and shear demand: effect of deformable shear connection (Northridge seismic input)

Fig. 9 shows the effect of slip constraints on the roof horizontal displacement by considering two extreme cases, namely slip (i) restrained (frame model 10TA) and (ii) unrestrained (frame model 10TN) at all beam-column joints. Taking frame model 10TA as reference, the difference in magnitude of the positive and negative peaks is -16.5% and +5.7%, respectively. The maximum absolute difference over the duration of the earthquake is 95.8 mm, and the average absolute difference is 14.1 mm.

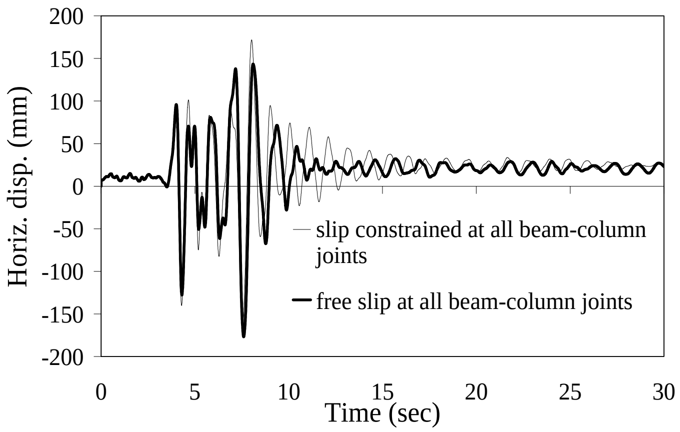

Figure 9. Horizontal displacement of left column at roof level: effect of slip constraint (Northridge seismic input)

4 CONCLUSIONS

From the above results it can be concluded that the inelastic partial composite action in the SCC frame structures considered in this study plays an important role on their global seismic (dynamic) response behavior. The shear connection deformability has a significant effect on global seismic response, i.e., increase of floor displacements and interstory drifts, and decrease of interstory shear demand. These effects are amplified when slip constraints are not present at any beam-column joints. Thus, a proper representation of the slip boundary conditions for all composite beams is crucial for accurate response simulation. In addition, a frame model with deformable shear connection not only accounts for the effect of partial composite action in the global response prediction, but can also provide useful information on the response of the shear connection (in terms of interface slip and shear force). This local response prediction, that can be obtained at a relatively low additional computational cost (as compared to frame models with monolithic beams), allows to asses the shear connection behavior under dynamic/seismic loads.

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