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PERFORMANCE OF MULTI-STOREY COMPOSITE STEEL-CONCRETE FRAMES WITH DISSIPATIVE FUSE DEVICES

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Abstract. *Repair work of conventional steel-concrete composite frames can be very expensive, sometimes impossible due to practical problems. EU-funded research project FUSEIS (RFCS-CT-2008-00032) has introduced two innovative types of seismic resistant steel-concrete composite frame types with dissipative fuses, which are cost-effective and robust alternatives to conventional frame types. In these frames, damage concentrates mainly in the fuses, which are easily and inexpensively replaceable after strong seismic events. In this study, performance of the type-2 fuse device from FUSEIS project have been assessed inside a multi-level 2D steel-concrete composite frame, by means of nonlinear time history analysis. Performance of a benchmark building with and without fuse devices has been compared in terms of damage, floor displacements and drifts.*

1 INTRODUCTION

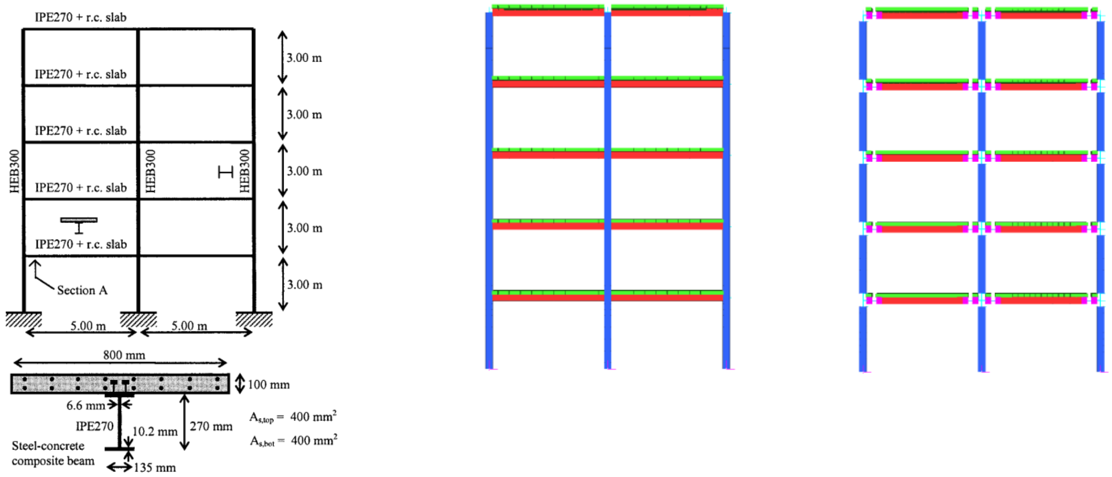
In general, it can be stated that the behavior of steel buildings during devastating earthquakes of the last three decades was quite satisfactory, if they were designed according to the modern seismic codes [1]. However, in most of the cases, despite not causing global collapse, main structural elements (steel beams, columns and concrete slabs) suffered costly damages. Repair work in these cases was most of the time not feasible, if not too expensive.

In order to enhance the seismic performance of steel-concrete composite frames, and facilitate the repair work after strong seismic events, two innovative dissipative devices were developed within European research program FUSEIS (Dissipative devices for seismic resistant steel frames) [2]. The first device type (fuseis 1) is used as a dissipative “shear wall” [3], whereas the second device type (fuseis 2) resembles “replaceable plastic hinges” for moment resisting frames [4]. Both systems aim at controlling and concentrating inelastic deformations in the “fuse devices”, without imposing significant damage in the rest of the structure (steel columns and beams, concrete slab). In this way, after the seismic event, the damaged fuse devices can be replaced with the new ones, instead of the costly operation of repairing major structural beams and columns. Therefore, the building with new fuses can turn back to service with a relatively minor repair cost.

Behaviour of these fuse devices was studied numerically and experimentally during the FUSEIS research project, and the results have been published in several articles [2-8].

This article presents the performance of a benchmark building, with and without fuse devices (figure 1b, 1c), in terms of damage, floor displacements and drifts, by means of nonlinear transient dynamic analysis. Post processing of the results of this study is still underway, with which the authors aim to provide the energy dissipation characteristics of the building with fuses, based on the criteria suggested by Castiglioni et. al [9], and Ballio et. al. [10]

The benchmark building is a five-storey and two-bay steel-concrete composite frame, which was studied numerically by Zona et.al [11]. Each bay has a span of 5.00 m and story 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, top and bottom reinforcements of 400 mm^2 and a concrete cover of 30 mm (figure 1). Yield stress of column and beam steel is 275 MPa, reinforcement steel is 430 MPa, and compressive strength of concrete is 33 MPa.



a. Benchmark steel-concrete composite frame [11]

b. Numerical model of building without fuse (NOFUSE)

c. Numerical model of building with fuse (FUSE)

Figure 1 Benchmark frame and its representation with and without fuse devices

2 NUMERICAL SIMULATION

Parametric analysis of the fuse devices were performed with refined numerical models at Politecnico di Milano, which can only be used for specific research applications. In order to study the performance of whole building structures, a simplified model has been proposed hereafter and validated against experimental results from previous studies.

Numerical models of this study have been developed with the commercial software package Straus7 [12]. Nonlinear transient dynamic analyses have been performed, considering material and geometrical nonlinearities, using inelastic fiber-based cross sections for the beam and column elements, and plastic links for fuse elements.

Fiber based modeling distributes plasticity by numerical integrations through the member cross sections and along the member length, and with a “plane sections remain plane” assumption [13]. Uniaxial material models are used to capture the nonlinear hysteretic axial stress-strain characteristics in the element cross sections. Fibers are numerically integrated over the cross section to monitor the axial force and moments, incremental moment-curvature and axial force-strain relations (figure 2). The cross section parameters are numerically integrated at several sections along the member length, using displacement or force interpolation functions. This approach allows performing nonlinear analysis considering both geometric and material nonlinearity, within a time much more limited than a 3D continuum finite element analysis. However, using this approach local behaviour such as degradation due to local buckling is difficult to capture without sophisticated models. Fiber-based modeling approach with distributed plasticity (DPE) offers a good compromise in terms of accuracy and computational time to model hysteresis behavior of steel struts. Kanyilmaz [14] has demonstrated the accuracy and efficiency of this modeling approach for steel beam-column elements.

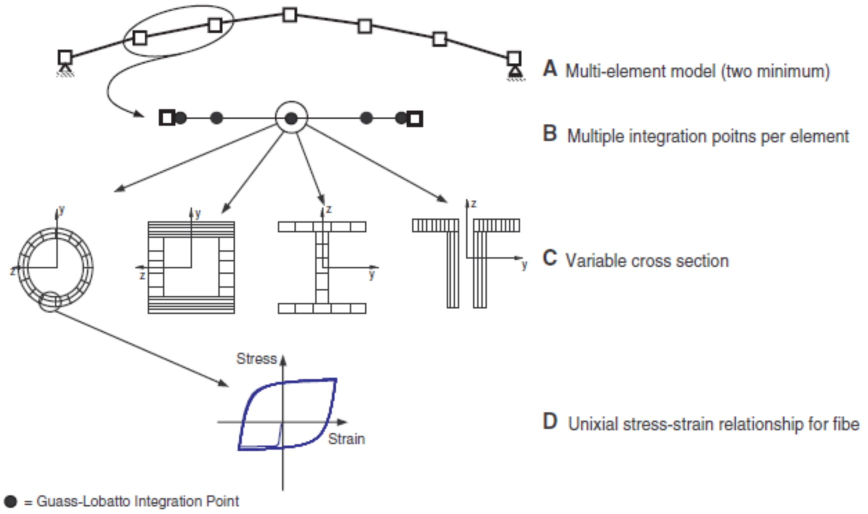


Figure 2 Fiber based distributed plasticity approach [15]

In order to investigate the effect of using fuse devices in building structures, two buildings have been analyzed:

Model 1: Benchmark building with conventional moment-resisting steel-concrete composite frame (NOFUSE)

Model 2: Benchmark building with Fuse devices that are placed close to the end of composite beams (FUSE).

Since the sliding between steel beam and concrete slab has a negligible influence on the global response of the building, a simplified procedure has been used to model the composite steel beam. The concrete slab has been modeled as a beam element accounting for its effective width and height. An additional beam element has been used to simulate the two rows of steel reinforcement, placed at the centroid of concrete slab. The distance between the centroids of concrete and steel beam is taken into account applying an offset to the concrete slab element (figure 3).

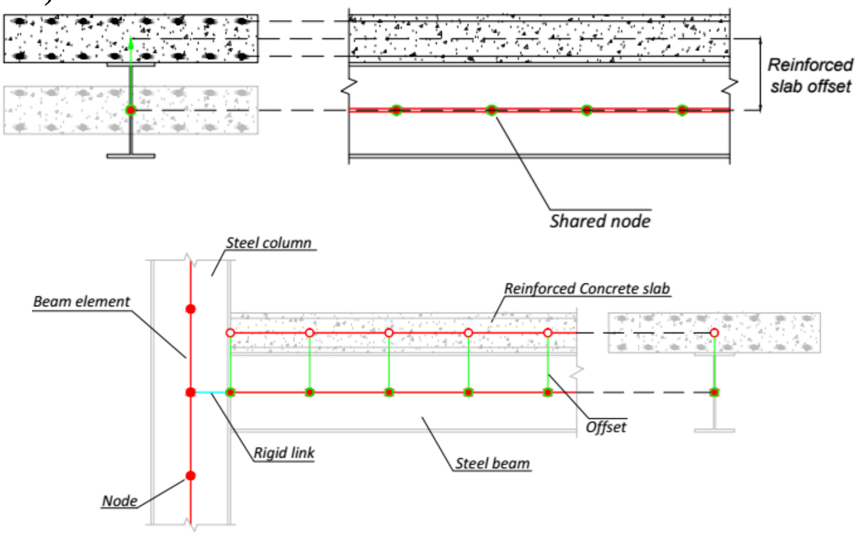


Figure 3 Numerical modeling of composite steel-concrete beam

Elasto-plastic material has been used for all structural elements in the model, with kinematic hardening. This approach will give an insight on the distribution of plasticity in the structure under nonlinear time history analysis. Figure 4 presents the constitutive models of the material properties used for steel profiles, concrete slab, and steel reinforcement bars. The exact values of each material property are shown in the reference articles.

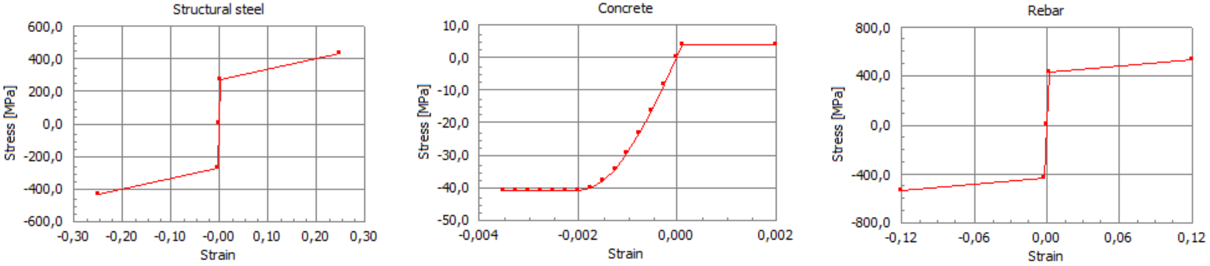


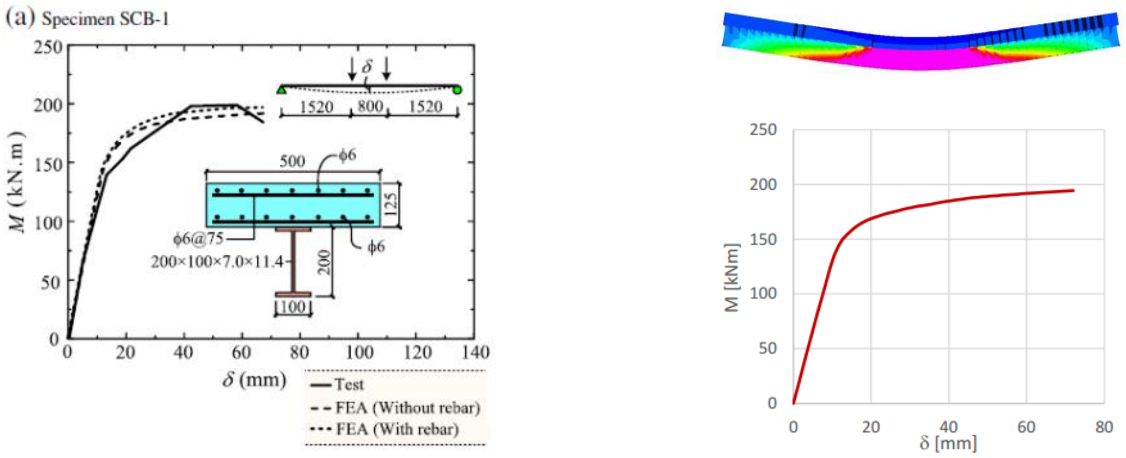
Figure 4 Material properties used in the models

This numerical modeling procedure has been validated with reference to the previous experimental studies [16-18]. A summary of the validation process is given in the following sub-chapters.

2.1 Validation of Steel-Concrete Composite Frame Models

The proposed modeling procedure has been validated with the experimental results of tests performed with simply supported beams (i) [16], continuous beams (ii) [17], and plane frames (iii) [18].

i) The nonlinear static behaviour of a simple composite steel-concrete beam has been simulated under monotonic loading, and the numerical results have been compared with the experimental results. Figure 5 shows the test specimen, experimental and numerical results of the original study, and numerical results of this study. Initial stiffness and moment capacity behaviour of the test specimen have been captured adequately.

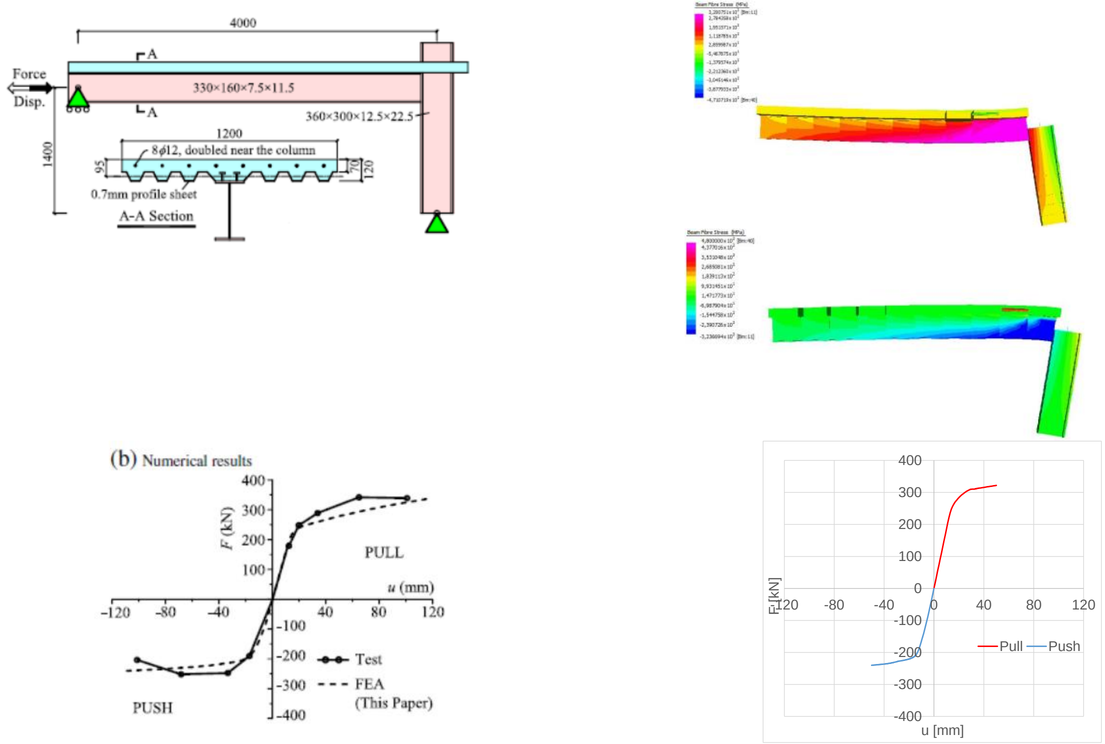


a. Experimental and numerical results of the original study [16]

b. Numerical results of this study

Figure 5 Numerical validation of composite simple beam element

ii) The nonlinear behaviour of a composite steel-concrete joint has been simulated under cyclic loading, and the numerical results have been compared with the experimental results. Figure 6 shows the test specimen, experimental and numerical results of the original study, and numerical results of this study. Initial stiffness and moment capacity behaviour of the test specimen under sagging and hogging have been interpreted adequately.

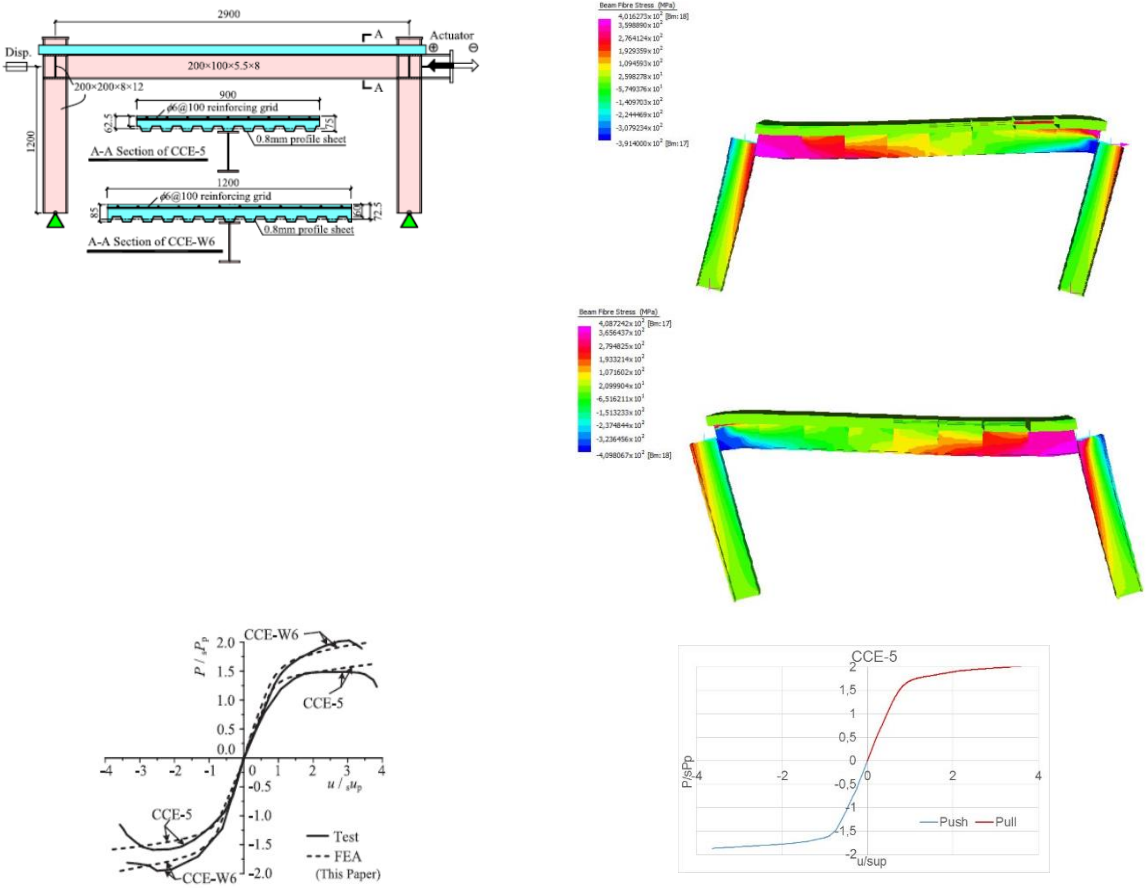


c. Experimental and numerical results of the original study [17]

d. Numerical results of this study

Figure 6 Numerical validation of composite joint

iii) The nonlinear static behaviour of a 2D planar composite steel-concrete frame has been simulated under cyclic loading, and the numerical results have been compared with the experimental results. Figure 7 shows the test specimen, experimental and numerical results of the original study, and numerical results of this study. Numerical models overestimate the moment capacity of the specimen, yet the results are quite reasonable to be used in a global model, for the performance comparison objective.



a. Experimental and numerical results of Nie et. al [18]

b. Numerical results

Figure 7 Numerical validation of planar frame

2.2 Validation of Numerical Models of Composite Steel-Concrete Frames with Fuse Devices

A numerical model has been developed, which simulates the nonlinear cyclic behavior of the full-scale specimen tested during the FUSEIS project. The specimen is a two dimensional portion of a storey of a composite steel multistory building, the configuration of which can be seen in figure 8. The frame consists of four HEB240 steel columns, two IPE300 steel beams, and a 150 mm thick reinforced concrete slab. The slab is supported by IPE160 transverse beams placed every 1.4 meters, in addition to a pair of transverse beams that are placed at each beam-column connection. Full shear connection is provided between the slab and the steel beam by means of IPE100 sections welded on top of the beam flange, acting as shear studs. To connect the steel plates to the beams in the fuse parts, high strength friction grip (HSFG) bolts are used.

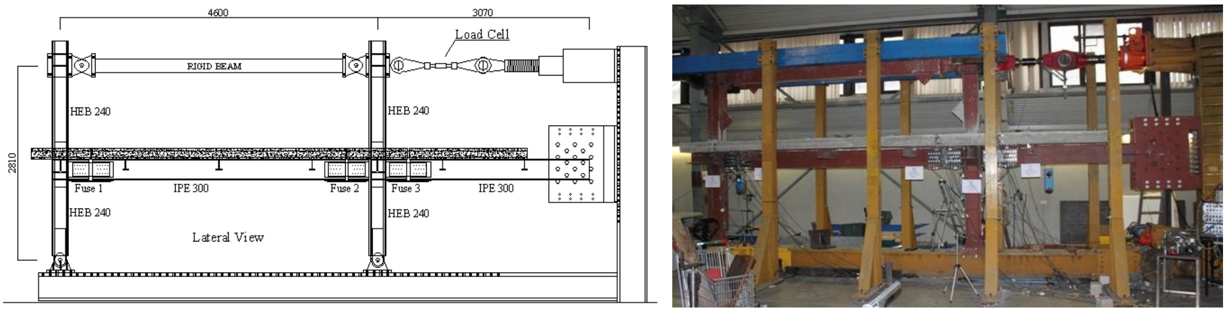
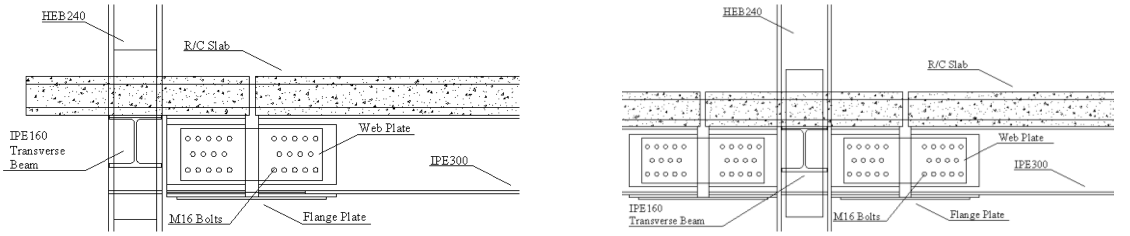


Figure 8 Full scale test specimen of FUSEIS project [1]

The column bases are restrained against horizontal and vertical displacement through pin connections. The beam-to-column connections are welded off site and can be considered as rigid connections. The IPE300 beam connected to the right column by fuse device n.3 is restrained only against the vertical displacement, but free to slide in horizontal direction. The out of plane stability of the frame is provided with transversal elements providing a pinned joint free to slide longitudinally on the reaction frame of the laboratory (Figure 8).

The fuse devices are obtained by means of plates bolted to the web and the lower flange of the beam. They are installed within the distance of a beam depth to the beam-to-column connection. The part of the beam near the connection is reinforced with steel plates welded to the web and to the flanges. These plates are positioned in order to avoid yielding of the beam due to bearing. Their thicknesses are chosen in order to provide, in that section, a total thickness two times the one of the web and the lower flange of the main beam respectively. In this way no plasticization is expected to occur in the beam but only in the replaceable part where the failure is expected to take place. Also the part of the column near to the connection is reinforced in order to obtain a rigid joint and hence concentrate all the damage on the fuse device. The interior and exterior fuse connection details are shown in figure 9. Thanks to this connection arrangement, the center of rotation at the fuse device is shifted above, and it stays in between the two reinforcement layers. As a result, the steel plates in the fuse devices can be easily deformed and buckled, causing energy dissipation without damaging the whole structure. At the same time the reinforced concrete slab does not get a significant damage due to large story drifts which cause large rotations in the fuse devices.



a. Exterior Beam-Column Joint Detail

b. Interior Beam-Column Joint Detail

Figure 9 Details of fuse devices

To avoid cracking of the concrete in the fuse section due to flexural deformation, a gap of 50 mm is left in the concrete slab in the section of the fuse. The steel reinforcement is not interrupted in the gap section. The scope of this gap is to allow concentrated rotational deformation to occur in the gap section, avoiding both crushing of the concrete as well as damage to the floor finishes (like tiles, or other). For this reason, the gap is conceived to exist anywhere there is a need to accommodate concentrated rotational deformation according to the global deformed shape of the building under seismic action, provided that diaphragm action is ensured.

Columns, steel beams, concrete slab, and reinforcement steel are modelled with fiber-based inelastic beam elements. Reinforced parts of the structure (beam-column joints, and segment of beams close to the fuse device) are modeled with rigid links, and the beam element representing the concrete slab has been introduced with an offset. Fuse elements are simulated as a "connection element" (figure 10).

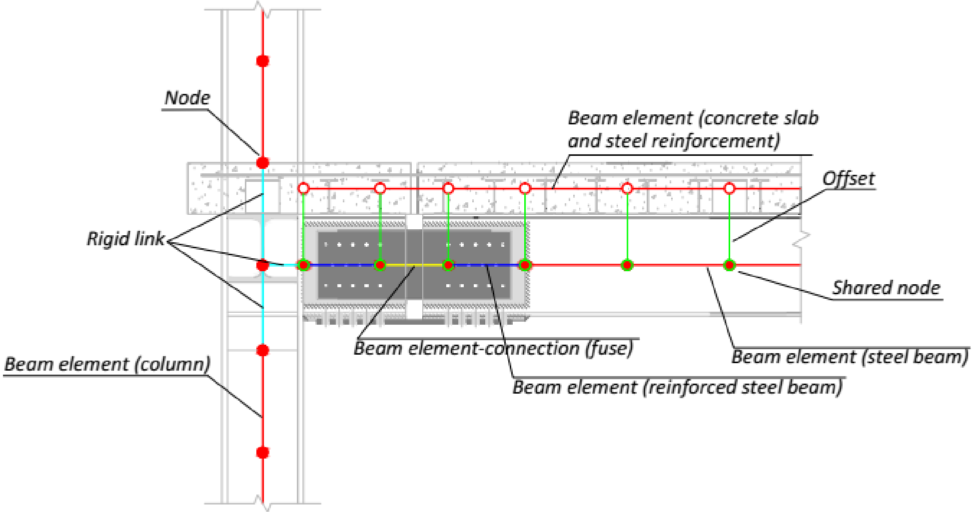


Figure 10 Numerical modeling of composite steel-concrete beam joints with fuse devices

Elasto-plastic material has been used for all structural elements in the model, with kinematic hardening. This approach will give an insight on the distribution of plasticity in the structure under nonlinear time history analysis. Figure 11 presents the material properties used for steel profiles, concrete slab, and steel reinforcement bars.

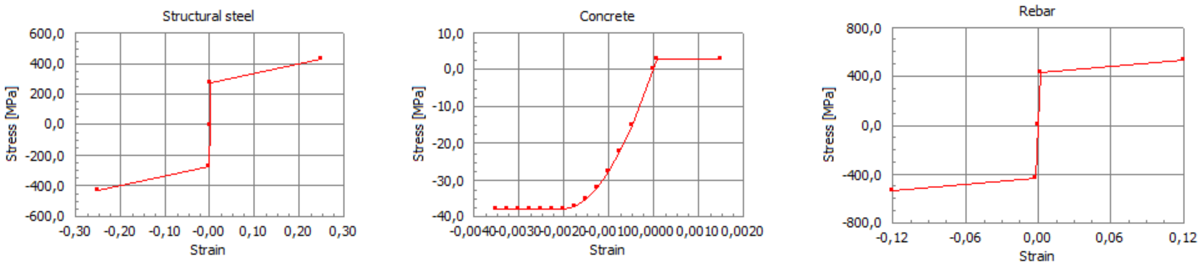


Figure 11 Material properties used in the numerical model

Connection elements have multilinear plastic moment-rotation input, which are calculated analytically according to design guide of fuse-2 device [19], and calibrated according to the results of the component tests performed during FUSEIS project [20] (figure 12).

Figure 12 Moment rotation input for fuse devices

The conformity of numerical and experimental results in terms of global force-displacement behavior is shown in figure 13. The differences in the initial stiffness and amplitude of the hysteresis plots are due to pinching phenomena occurred in the connections between the fuse devices, which were not accounted for in the numerical model.

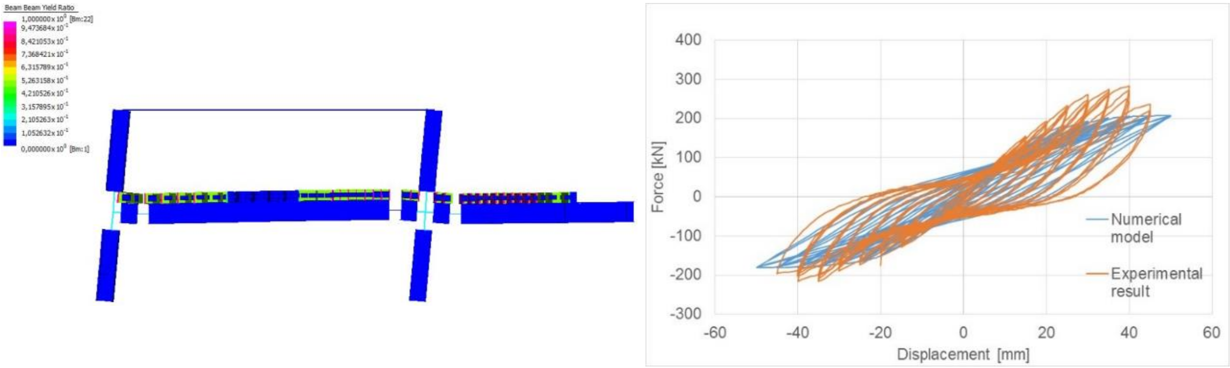


Figure 13 Numerical validation of FUSEIS test specimen

The results of these simple models validate the modeling procedures used in this study, which can be used to study the nonlinear dynamic behavior of multi-story steel-concrete composite frames.

3. BUILDING MODELS

To evaluate the performance innovative multi-story building frames with fuse devices under seismic actions, nonlinear dynamic analyses have been performed for the two building models:

1) Benchmark building with conventional moment-resisting steel-concrete composite frame (NOFUSE)
2) Benchmark building with Fuse devices that are placed close to the end of composite beams (FUSE). Fuse device is the same, the validation of which is shown in chapter 2.

All the elements in both models are inelastic beams with fiber-based formulation, so that the differences between two models in terms of “plasticity” index can be seen. Figure 14 shows the input material properties.

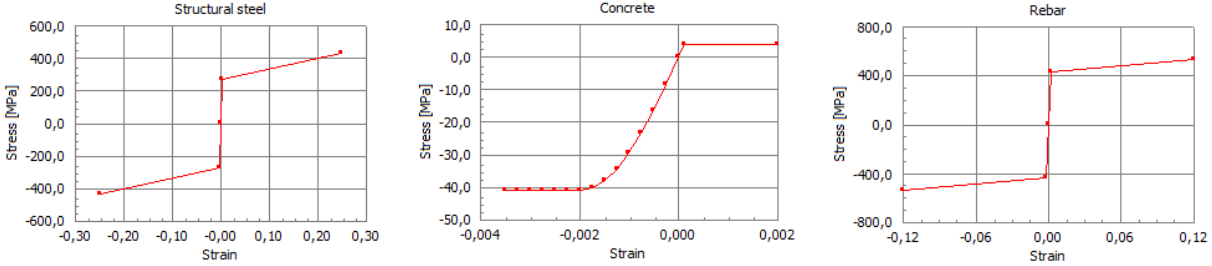


Figure 14 Material properties used in the building models

The analyses have been performed under artificial accelerograms produced according to the design criteria of the benchmark building previously studied by Zona et.al. [11]. This building was designed according to Eurocode 4 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, uniformly distributed along the composite beams), and seismic forces were evaluated using response spectrum analysis with peak ground acceleration=0.35 g, Type 1 spectrum of Eurocode 8, modal damping ratio=0.05, and soil class B. Accelerograms have been produced with Gosca software [21]. Response spectra can be seen in figure 15.

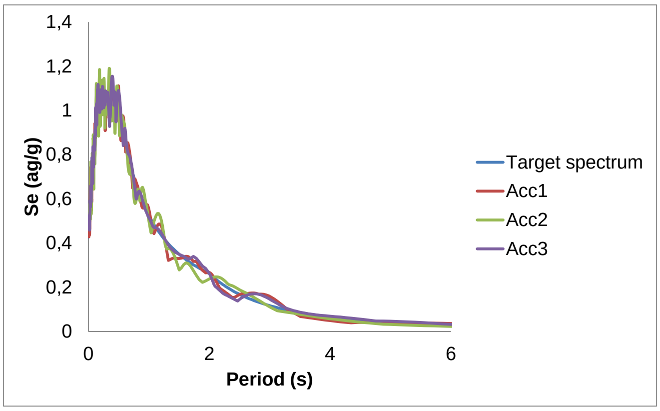


Figure 15 Response spectra for 0.35g, Type 1, Soil type B, Damping ratio 5%.

Each accelerogram has the last 5 seconds with zero accelerations, which will help to observe the residual displacements and drifts in the buildings, after the seismic event (figure 16).

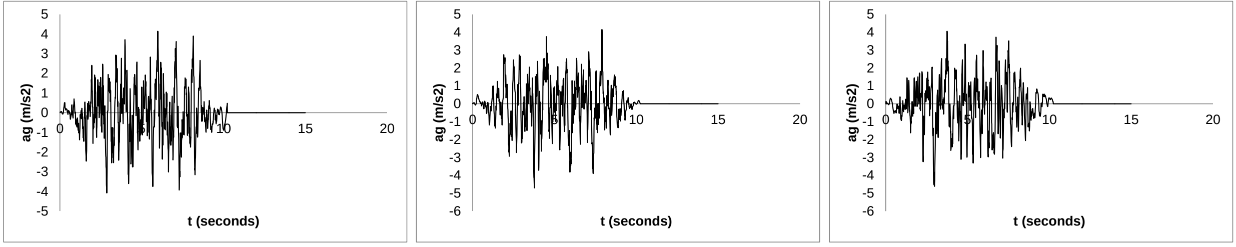
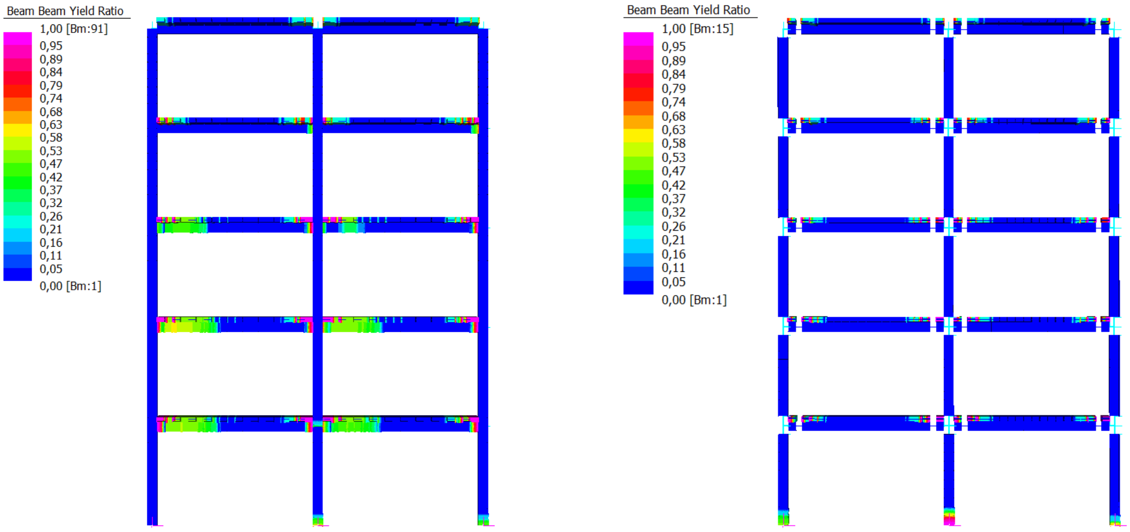


Figure 16 Three artificial accelerograms

Results show the capability of innovative frame (FUSE) in controlling and concentrating the plasticity in the fuse devices. Figure 17 shows the yield index obtained at the end of the seismic event (acc2), for both frames. While in the NOFUSE frame, main steel beams and concrete slabs are significantly damaged, in FUSE frame, the damage is mainly concentrated in the “connection elements” that represent the fuse devices. Indeed, numerical convergence problems occurred in the former case, which probably indicates a “collapse” or very large damage in the structural elements. All three analyses of FUSE frame were finalized without convergence problems. Plasticity observed in the concrete slab of FUSE model are due to tension cracks.

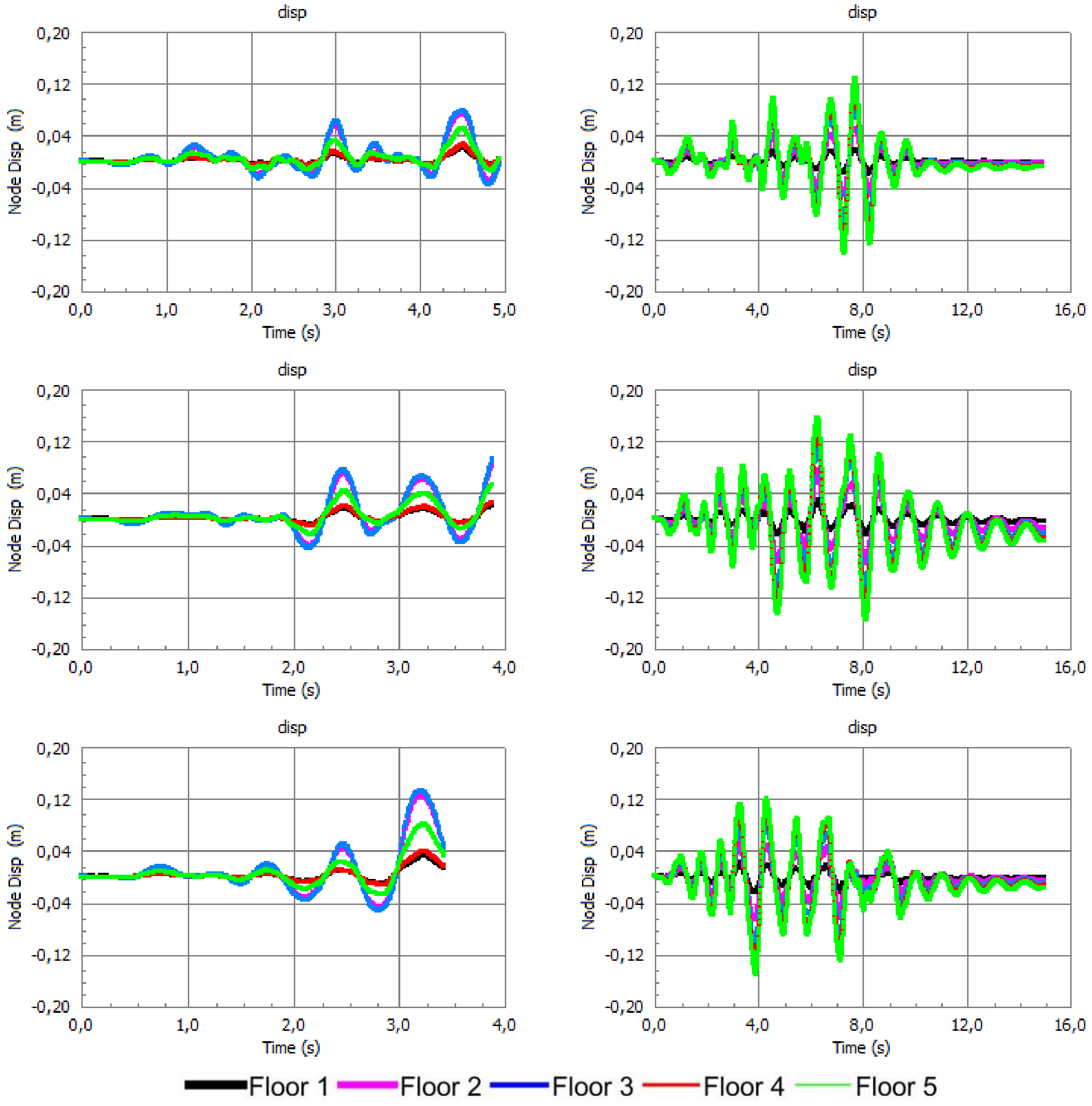


a. NOFUSE frame

b. FUSE frame

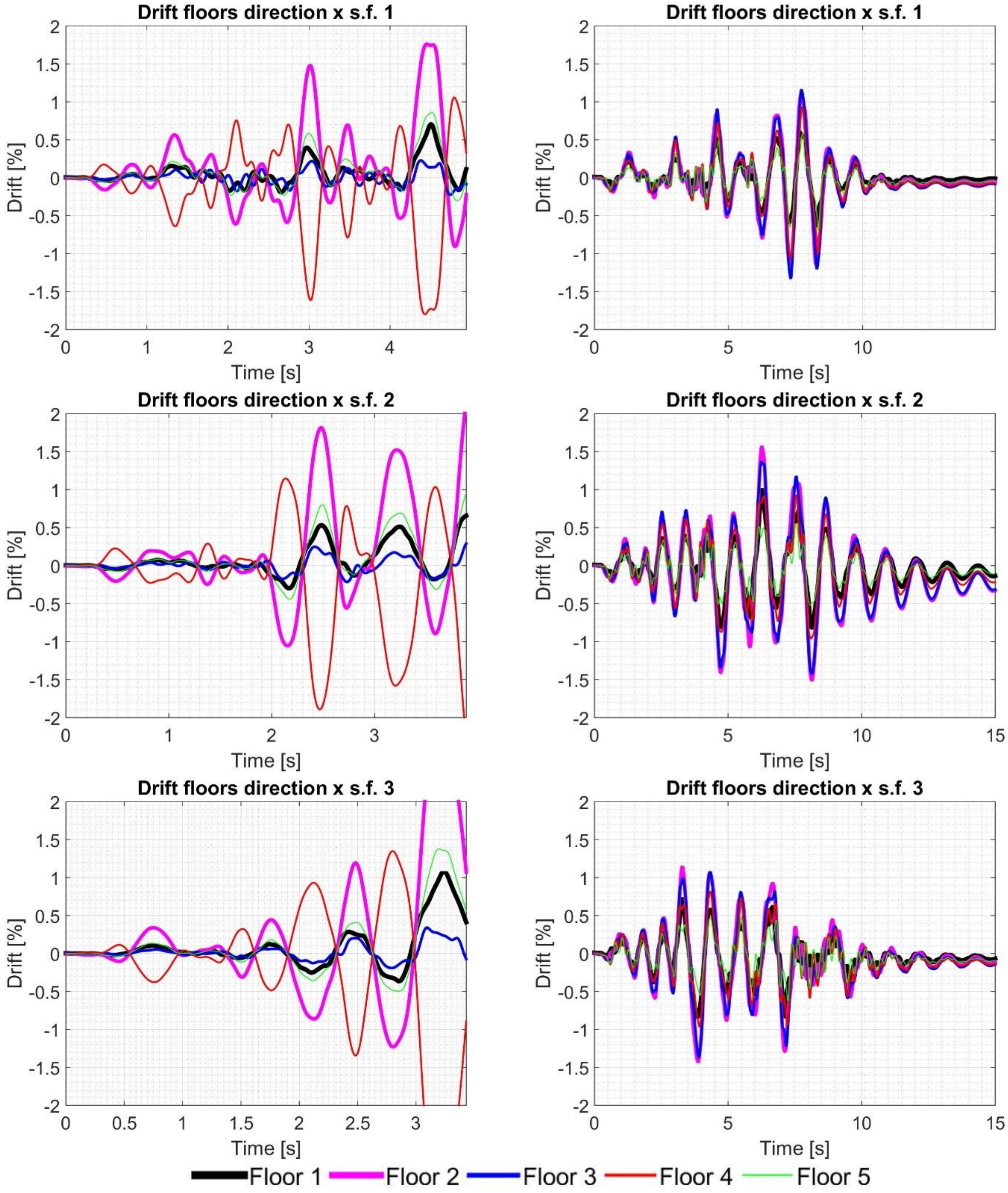
Figure 17 Comparison of cumulative ratios between two building models

Figure 18 and 19 show comparisons in terms of floor displacements and drift ratios between two cases. It is seen that the structure with fuse devices reaches the end of the seismic event with limited displacements and drifts, whereas original building suffers from non-homogenuous distribution of floor displacements and drifts (which can indicate a soft-storey behaviour). It is also interesting to see that after the seismic event, the residual floor drifts in the FUSE frame remain insignificant. In practice, this means that the repair work should be easy, which would not be the case for the NOFUSE frame.



a. NOFUSE frame b. FUSE frame

Figure 18 Comparison of floor displacements



a. NOFUSE frame b. FUSE frame

Figure 19 Comparison of floor drift ratios

4. CONCLUSIONS

This article compared the yield index, and floor displacements and drifts of a benchmark composite steel-concrete frame by means of nonlinear transient analysis, with and without fuse devices that were developed in FUSEIS project [19].

First, the proposed modeling procedure has been validated with the results of previously performed experiments. Then a numerical model has been developed, which successfully simulates the nonlinear cyclic behavior of the full-scale specimen tested during the FUSEIS project. Based on the results of these numerical validations, multi-story steel-concrete composite frames have been modeled with and without fuse devices. Nonlinear transient dynamic analyses have been performed to study the seismic performance of these building frames.

In the FUSE frame, damage was mainly concentrated in the fuse devices, the rest of the structure remaining mostly elastic. In this case, only concrete slabs had tension cracks, and column bases had some plasticity. Whereas conventional frame without fuse devices suffered from highly yielding beams and crushed concrete slabs which resulted in excessive story drifts.

These results show that, using fuse devices, inelastic deformations can be controlled and concentrated in “replaceable fuses”, without damaging the main structural elements of a composite steel-concrete frame. Authors suggest to investigate the problem of column base yielding, which can again be solved through an innovative replaceable technology. Further analyses, which investigate the behavior of 3D buildings, are underway, and will be published soon.

5. ACKNOWLEDGMENT

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