SEISMIC BEHAVIOR OF FRAMES WITH INNOVATIVE ENERGY DISSIPATION SYSTEMS - FUSEIS 1

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1. SUMMARY

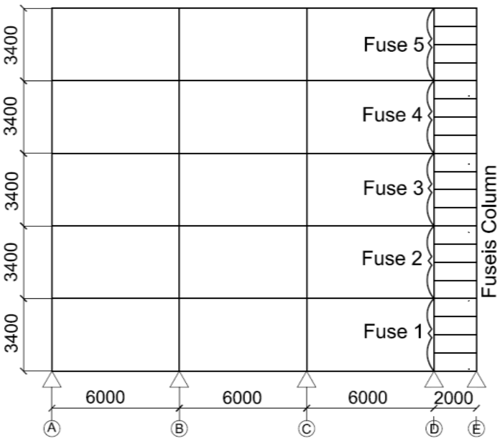
Modern seismic codes allow for inelastic deformations in dissipative zones during design earthquakes, accepting damage to a certain extent in the relevant structural parts. In the frame of the European Research Program "FUSEIS" two innovative seismic resistant systems were introduced and relevant design guides developed. This paper reports on the seismic performance of buildings with FUSEIS 1 system that consists of a pair of closely spaced strong columns jointed together by fuses. The system is further subdivided in FUSEIS 1-1 and 1-2, where the dissipative fuses are multiple beams or respectably short pins. Experimental investigations on the latter system were presented analytically at the 7th National Conference of Steel Structures. Non-linear static and dynamic analyses based on the test results are presented and design recommendations and appropriate behavior factors are provided. The results indicate that the system has, under certain conditions, self-centering properties in addition to good performance.

2. DESIGN OF CASE STUDIES

The case studies are based on the extraction of a plane frame from a five-story composite building shown in Figure 1. At the frame end one FUSEIS 1 system is used that provides seismic resistance ([1], [2], [3], [4]). The system consists of a pair of hollow strong columns and five devices per story. Steel grade is S 235 for the dissipative elements and S355 for all other structural elements. The devices are rigidly connected to the system's columns. The FUSEIS 1-1 beams are SHS sections reduced near the ends by approximately 30% (RBS) and the FUSEIS 1-2 devices consist of circular pins and receptacle beams with hollow sections. The dead and live loads considered are equal to 2.00kN/m^2. Considering that equal plane frames are placed at a distance of 8 m in the building, the corresponding line loads on the beams are 16.00kN/m. Figure 1 also includes the assumptions for the seismic loads. The main properties of the structural model for analysis and design that was implemented in the SAP 2000 software Code [5] are the following:

- All structural elements are represented by beam elements.

- The main frame floor beams are subdivided to three parts; steel sections are assigned at the ends, where negative moments develop, and composite beam sections in the middle part.
- Rigid end length offsets are included at the beams to consider their clear length.
- Columns bases are pinned to prevent a moment transfer to the foundation.
- The joints between main frame floor beams and columns are semi rigid. Accordingly springs are assigned at beams' ends to introduce these partial fixity conditions with properties determined in accordance with EN 1994 [6] and EN 1993 [7]. The structure is designated as "FUSEIS+PF" to indicate the fact that the seismic resistant system is a combination of the FUSEIS system and a partially fixed moment frame.
- The beam elements representing the FUSEIS 1-1 beams are divided in five parts that represent the full sections (ends – middle) and the RBS-sections.
- The beam elements representing the FUSEIS 1-2 devices are divided in three parts with different cross sections: the receptacle beams at the ends and the weakened pin in the middle.
- The joints between floor beams and system columns are considered as simple.



Seismic loads		
Elastic response spectra	Type 1	
PGA	A=0,36g	
Importance class	II	
Ground type	B	
Behavior factor q	FUSEIS1-1	FUSEIS1-2
	5,00	3,00
Damping	5%	
Factors of operating loads	φ=1,00 (roof), φ=0,80 (correlated occupancies)	

Figure 1: 2D building frame and assumptions for seismic loads

The method of analysis employed for design was multi modal response spectrum analysis, the first mode of vibration activates around 80% of the mass, the second around 15%, so that the two first modes activate approximately 95% of the mass. Structural design for the basic and seismic combinations of actions was performed in accordance with EN 1993-1 [8] and EN 1998-1 [9], with additional checks for this system included in the Design Guide [2]. Both ultimate and serviceability limit states were considered for gravity and seismic loading. The resulting cross sections of the main frame were HEA260 for beams and SHS200x20 for columns. Table 1 and Table 2 summarize the system fuses used and the system columns. As expected, the sections get smaller from lower to higher stories.

Table 1: FUSEIS sections

Fuse No	FUSEIS1-1	FUSEIS1-2
1	SHS300x10	D95
2	SHS280x8	D90
3	SHS260x8	D85
4	SHS240x8	D80
5	SHS180x8	D70

Table 2: System columns' sections

	Stories	System columns	Receptacle beams
FUSEIS1-1	1-5	RHS 400x300x20	-
FUSEIS1-2	1-2	RHS 400x300x35	RHS 260x220x25
	3-5	RHS 400x300x20	SHS 240x20

The FUSEIS devices were designed to assure the development of a bending mechanism and fulfill the conditions of EN 1998-1 [9] and the relevant Design Guide [2]. The system columns, the receptacle beams and the connections were capacity designed with an overstrength factor Ω equal to $M_{pl,fuse}/M_{Ed}$. For FUSEIS 1-2 system an additional overstrength factor, a=1,5, derived from the nonlinear analysis was used to ensure that the failure of the pins occurs first.

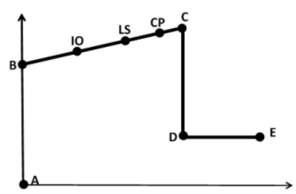
3. NON- LINEAR STATIC (PUSHOVER) ANALYSES - EVALUATION OF THE BEHAVIOR FACTOR

Non-linear static (pushover) analyses were performed to verify the collapse mechanism and check the behavior factor used in the linear analyses. Analysis was performed in accordance with the first mode of vibration including P–Delta effects. Rigid plastic hinges were introduced at the ends of all the structural members.

Plastic hinge properties of the columns were of P-M3 type, taking into account the interaction between bending moments and axial forces, while in the receptacles of FUSEIS 1-2 they were of bending type (M3 hinge). These properties were calculated according to FEMA 356 [10].The hinge properties of the rotational springs that simulated the semi rigid joint were of bending type (M3 hinge) and were calculated for positive and negative moments. M3 hinges were also assigned at the FUSEIS beams/pins, their properties being determined from calibration of experimental results (Figure 2). The adopted properties of the pins indicate that they develop, due to strain hardening and catenary action, considerable overstrength.

In order to evaluate the effect of the type of floor beam-to-column joints on the performance of buildings with FUSEIS 1 systems two additional case studies were examined as following: a) fully restrained joints, where a moment frame works in combination with the FUSEIS system (FUSEIS+FR) and b) simple joints where the FUSEIS system is the only seismic resisting system (FUSEIS).

	FUSEIS1-1 (SHS beams)		FUSEIS1-2 (Circular pins)	
Point	$M/M_{pl,RBS}$	$\theta/\theta_{pl,RBS}$	$M/M_{pl,pin}$	$\theta/\theta_{pl,pin}$
A	0	0	0	0
B	0,6	0	1	0
C	α_{pl}	25	2	100
D	0,4	25	0,5	100
E	0,4	30	0,5	150
Acceptance Criteria (θ/θ_{pl})				
IO	5		30	
LS	12		45	
CP	18		60	



(α_{pl} =shape factor)

Figure 2: Non-linear hinge parameters for the FUSEIS

Figure 3 shows the comparison of the capacity curves (ATC40 [11]) for the three above mentioned connection types (FUSEIS+FR, FUSEIS+PF, FUSEIS). In each curve three points that will be used later in the determination of the behavior factor are indicated: the performance point, the life safety and the point where the experimental drift at ULS is reached. The MRF action (FUSEIS+FR or FUSEIS+PF) increases the capacity of the frame and leads to lower drifts compared to the hinged frame (FUSEIS). However, when

FR connections are used the beams of the main frame have to be capacity designed to resist lateral loads and so the use of a second system like FUSEIS leads to a heavier, more expensive structure and may be omitted. On the contrary, hinged connections are optional but more unfavorable for the FUSEIS system. The most effective solution is the PF frame that exploits the advantages of both the MRF and the FUSEIS system and is easier to realize in practice compared to FR. Figure 4 shows the hinge formation of the "FUSEIS+PF" frame when the experimental drift is reached.

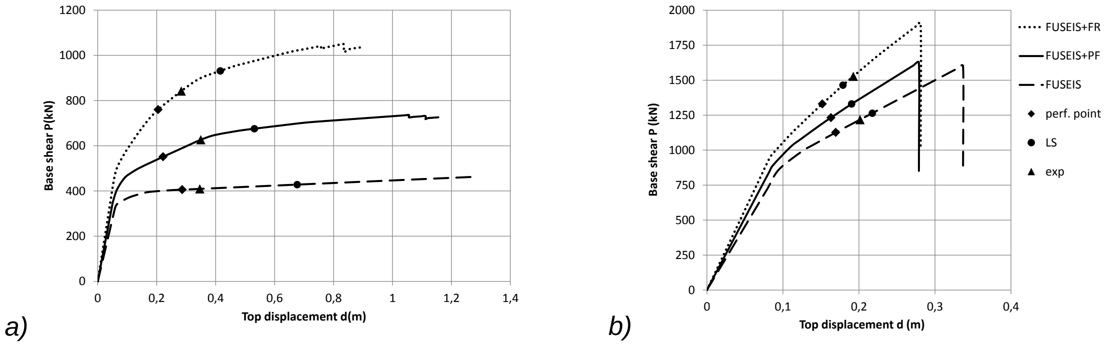


Figure 3 : Comparison of capacity curves a) FUSEIS1-1 b) FUSEIS1-2

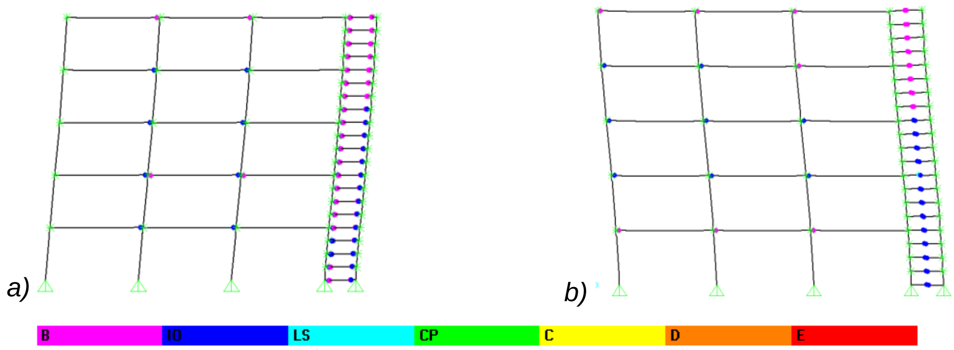


Figure 4: Hinges of the PF frame at the experimental drift a) FUSEIS1-1 b) FUSEIS1-2

The non-linear static analysis allows the evaluation of the behavior factor (q factor) of the structure. This may be defined as the product between the ductility (q_{μ}) and the overstrength (Ω). The ductility q_{μ} is determined as the ratio between the actual displacement when the beam/pin rotations reach the LS performance level or the experimental drift at ULS (whichever is more unfavorable) to the yield displacement of an equivalent bilinear system. Overstrength is defined as the ratio between the yield force ($V_{LS,Exp}$) of the bilinear system to the design force (V_d).
The calculated ductility, overstrength and behavior factors, denoted as q, are given in Table 3. The calculated q-factors are above the values considered in design. It is recommended to adopt q=5 for a FUSEIS 1-1, since higher values, although possible, would result in a more flexible frame and lead to increased θ and drift values.
In order to check the structural performance at smaller or higher seismic excitations, three design levels, Serviceability, Ultimate, Collapse, were introduced. This is done by application of a scaling factor to the PGA of the design earthquake equal to: 0,5 for SLS, (EN 1998-1 §4.4.3 [9]), 1,0 for ULS, 1,5 for CPLS. For these design levels the performance points were determined and the interstory drifts recorded. Table 4 shows the maximum values of interstory drifts for all design levels as well as the experimentally recorded drifts. The experimental drift at SLS is defined as the drift at which the experimental skeleton curve shows the first significant yielding. The drift at ULS is

defined at the state when the experimental curve reaches its maximum load. The drift at CPLS is the maximal drift attained in the tests, where the specimen had still significant strength reserves. It may be noted that the experimental drift values for FUSEIS 1-1 are similar to the values given by FEMA-356 [10] for Steel Moment Frames while for FUSEIS 1-2 they are between the values for Steel Moment Frames and the Steel Braced Frames. The comparison of the experimental drifts with those determined from the analysis indicates that at the performance points all drifts were below the values that were reached in the tests.

Table 3: Behavior factors

FUSEIS	q_μ	Ω	q
1-1	3,83	2,00	7,66
1-2	1,48	2,08	3,07

Table 4: Experimental, analytical and FEMA drifts (%)

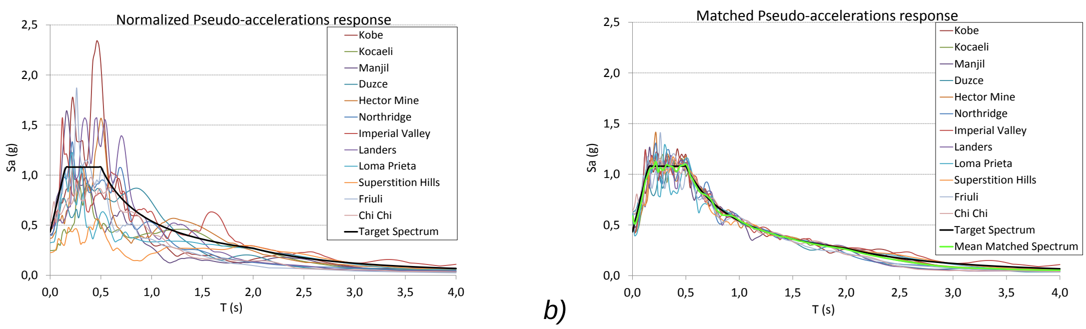
Perf. Levels	FUSEIS1-1		FUSEIS1-2		FEMA	
	Exp.	An.	Exp.	An.	MRF	Braced
SLS	1,00	0,81	0,66	0,66	0,70	0,50
ULS	2,40	1,60	1,38	1,19	2,50	1,50
CPLS	4,70	2,94	2,25	1,82	5,00	2,00

4. NON- LINEAR DYNAMIC ANALYSES (TIME-HISTORY)

Using characteristic seismic records from real strong motions, non - linear dynamic analyses on the examined building frames were performed to assess whether the elastic design with behavior factors meets the seismic performance objectives.

4.1 Ground motion records and simulation

The records were obtained from the Far-Field record set proposed by FEMA 695 [12] since it is considered appropriate for collapse evaluation of buildings. Scaling of ground motion records is a necessary element of nonlinear dynamic analysis and involves two elements: normalization with respect to the value of peak ground velocity (PEER NGA database) and scaling to a specific level of ground motions. The latter was achieved through the software SeismoMatch [13] which is able to adjust ground-motion records so that their spectral acceleration response matches a target response spectrum based on the EN 1998-1 [9] rules. Twelve records were selected, a number that for mid-rise buildings is generally considered enough to provide sufficient accuracy (Vamvatsikos and Cornell [14], Shome and Cornell [15]). Figure 5.a and b display the response spectra of the normalized far field records and the matched spectra along with the target spectrum respectively.



a) b)

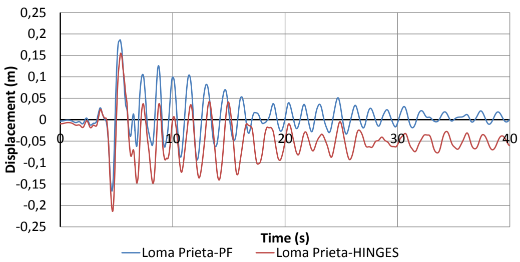
Figure 5: Response acceleration spectra a)Normalized b)Matched and mean matched

The matched records are stronger than the initial as they derive from matching them to the peak values of the target response spectrum. Even though this approach is unfavorable and leads to conservative results it was considered to be suitable to verify the design and to evaluate the performance of the FUSEIS system on the safe side.

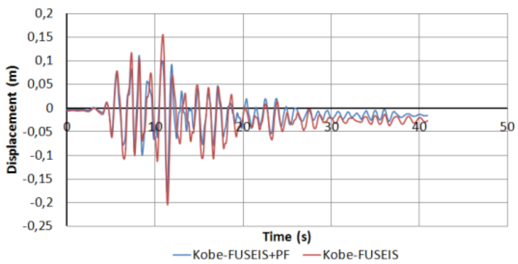
The models used in the previous analyses were appropriately modified to include the hysteretic behavior of the FUSEIS devices. Nonlinear links with multi-linear kinematic plasticity properties, determined experimentally, were assigned at the ends of the RBS/pins. Similar to the non-linear static analysis, simple floor beam-to-column joints were examined (FUSEIS) in addition to the semi-rigid ones (FUSEIS+PF).

4.2 Residual roof drifts

A structural system may be characterized as self-centering if it is capable of leaving the structure with little to no residual drifts after a major earthquake. Table 5 gives the residual global drifts, obtained by dividing the roof displacements by the building height. It may be seen that residual global drifts are close to zero and lower than the limit value of 0,5% given by FEMA 356 [9] for Braced Steel Frames at IO performance level regardless of the type of floor beam-column joints (FUSEIS or FUSEIS+PF). In Figure 6 the roof displacement time histories of the most unfavourable seismic excitations for FUSEIS 1-1 and 1-2 are given. These analyses confirm that the floor beams and the columns remain elastic and do not participate in the lateral resistance of the building. On the contrary, inelastic deformations concentrate only in the FUSEIS RBS/pins, while the strong system columns and the receptacle beams are capable of self-recentering the structure. These results indicate that the FUSEIS 1 system may be considered to possess self-centering properties.



a)



b)

Figure 6: Roof displacements a)FUSEIS 1-1 Loma Prieta b)FUSEIS 1-2 Kobe record

Table 5: Residual global drifts (%)

Earthquakes	FUSEIS1-1		FUSEIS1-2	
	FUSEIS + PF	FUSEIS	FUSEIS + PF	FUSEIS
Northridge	0,044	0,028	0,064	0,003
Duzce, Turkey	0,019	0,096	0,045	0,084
Hector Mine	0,215	0,063	0,062	0,040
Imperial Valley	0,104	0,131	0,027	0,073
Kobe, Japan	0,018	0,116	0,090	0,156
Kocaeli, Turkey	0,344	0,212	0,021	0,117
Landers	0,108	0,342	0,052	0,069
Loma Prieta	0,001	0,354	0,034	0,013
Manjil, Iran	0,064	0,107	0,023	0,048
Superstition Hills	0,051	0,122	0,010	0,011
Chi-Chi, Taiwan	0,155	0,132	0,043	0,026
Friuli, Italy	0,159	0,241	0,043	0,053
Average values	0,107	0,162	0,043	0,058
Standard Deviation	0,099	0,104	0,022	0,046

4.3 Interstory drifts

The residual interstory drift values are close to zero, similar to the residual roof drifts. The interstory drifts for the “FUSEIS+PF” case are lower compared to those when the FUSEIS system works alone. Figure 7 shows that the maximum drifts for the “FUSEIS+PF” system and the “FUSEIS” system are close to the experimental limit value at ULS (2,40%) for FUSEIS 1-1 and between ULS (1,38%) and CPLS (2,25%) for FUSEIS 1-2. The residual and the maximum interstory drift curves for the most unfavorable seismic excitations for FUSEIS 1-1 and 1-2 are also shown in Figure 7.

Earthquakes	FUSEIS1-1		FUSEIS1-2	
	FUSEIS+PF	FUSEIS	FUSEIS+PF	FUSEIS
Northridge	1,43	1,84	1,56	1,80
Duzce, Turkey	1,74	2,58	1,59	1,83
Hector Mine	1,61	2,10	1,41	1,52
Imperial Valley	1,31	1,62	1,34	1,56
Kobe, Japan	1,36	1,59	1,37	1,61
Kocaeli, Turkey	1,64	2,18	1,47	1,61
Landers	1,53	2,02	1,53	1,79
Loma Prieta	1,44	2,11	1,48	1,58
Manjil, Iran	1,28	1,81	1,02	1,17
Superstition Hills	1,41	1,59	1,34	1,60
Chi-Chi, Taiwan	1,41	1,67	1,63	1,91
Friuli, Italy	1,57	2,13	1,47	1,79

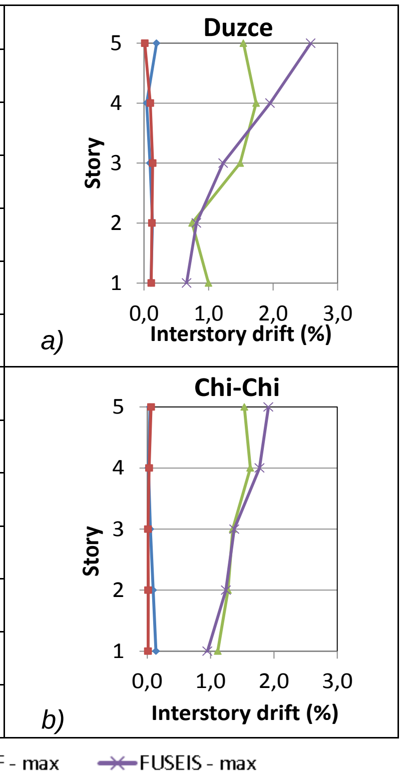


Figure 7: Maximum and Residual Interstory drifts (%) a)FUSEIS 1-1 Duzce b)FUSEIS 1-2 Chi-Chi

4.4 IDA curves and processing

The response of the system was further evaluated through the IDA method according to Vamvatsikos and Cornell [14]. In order to generate the IDA curves the ground motions of section 4.1 were scaled to increasing intensities until numerical non-convergence was encountered. Each IDA curve is defined by the most representative ground motion Intensity Measure (IM) and Damage Measure (DM), which correspond to the 1st-mode spectral acceleration Sa (T_1,5%) and the maximum interstory drift respectively (Figure 8). It is obvious that the IDA curves are conservative in terms of IM and have small dispersion justified by the matching method described in Section 4.1. All curves end with a “flatline” at the highest numerically conveging run. In order to be able to evaluate the performance of the system three limit states were defined on the IDA curves based on the maximum experimental drifts: Immediate Occupancy (IO), Life Safety (LS), Collapse Prevention (CP). The IDA curves and their correspondent limit state capacities are summarised to a median curve depicted in Figure 9. It can be observed that the IO, LS and CP points are very close to the median curve verifying the definition of these limit states.

Figure 10 displays the median peak interstory drift ratios at three Sa(T_1,5%) levels corresponding to 0,5, 1,0, 1,5 times the PGA of the design earthquake. At low IM the deformations are small and uniform for all stories. Higher IM demonstrate that the 3rd floor is the weakest and suffers significant deformation.

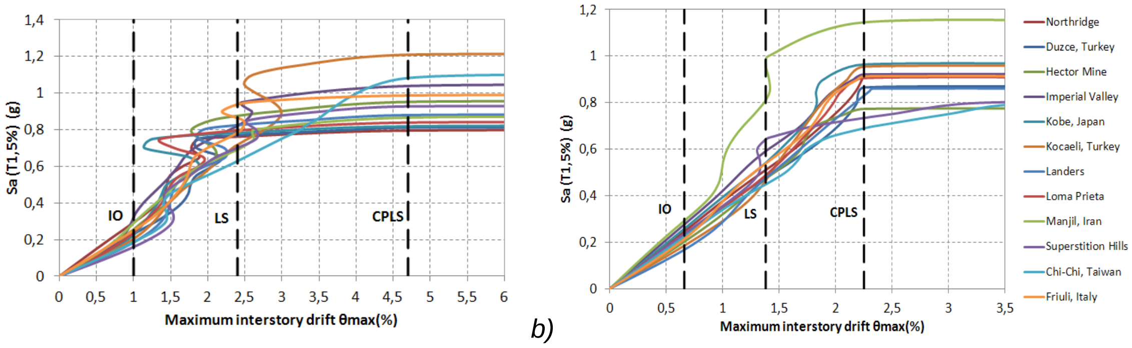


Figure 8: All IDA curves and limit state capacities a)FUSEIS 1-1 b)FUSEIS 1-2

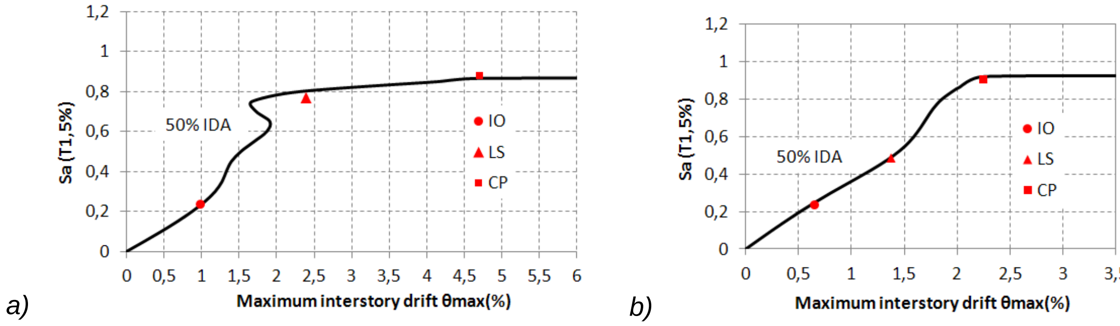


Figure 9: Median IDA a)FUSEIS 1-1 b)FUSEIS 1-2

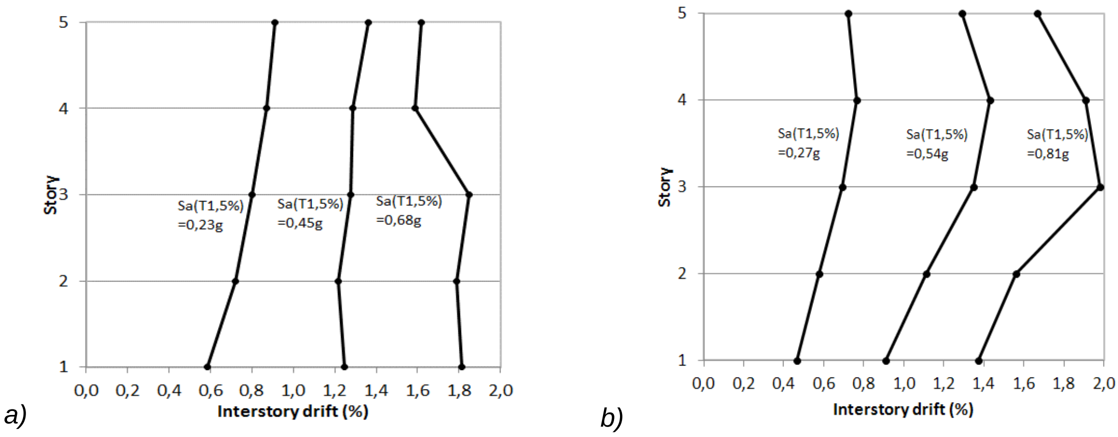


Figure 10: Median peak interstory drifts at three Sa(T_1,5%) levels a) FUSEIS 1-1 b) FUSEIS 1-2

Using collapse data obtained from IDA results, the collapse fragility was defined through a cumulative distribution function (CDF), describing the probability of collapse as a function of the ground motion intensity. A lognormal distribution was applied using the median collapse intensity ($SCT_{FUSEIS1\text{-}1}$=0,83g and $SCT_{FUSEIS1\text{-}2}$= 0,86g) and the standard deviation parameter, both of which are obtained from the IDA data considering uncertainties of modeling, design requirements, test data and records (Figure 11(1)). Similar to the median IDA curve the fragility curve is also conservative and underestimates the system's performance. The fragility curve is shifted to the right to account for the spectral shape effects (Figure 11(2)), by multiplying the CMR by the spectral shape factor SSF. The result is a significant reduction in the probability of collapse.

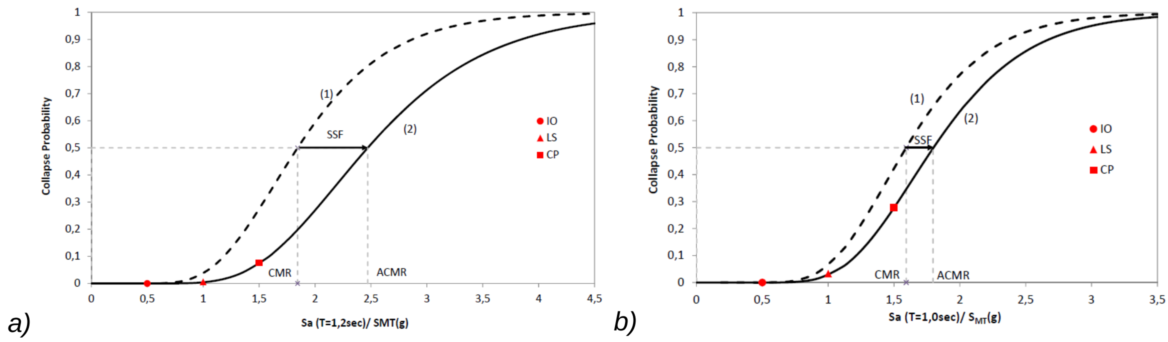


Figure 11: Collapse fragility curves a)FUSEIS 1-1 and b)FUSEIS 1-2 modified to account for (1) total system collapse uncertainty and (2) spectral shape effects

The new median point, called the Adjusted Collapse Margin Ratio (ACMR) was calculated according to FEMA P695 [12] to verify the seismic performance factor $q_{FUSEIS1-1}$=5 and $q_{FUSEIS1-2}$=3 employed. ACMR was equal to 2,47 for FUSEIS 1-1 and 1,8 for FUSEIS 1-2 which exceed the acceptable collapse margin ratio, ACMR20%, of 1,31 and 1,29 respectively fulfilling the acceptance criteria.

5. SUMMARY AND CONCLUSIONS

FUSEIS 1 is an innovative seismic resistant system with possibly self-centering capabilities that uses replaceable fuses to provide energy dissipation. This paper proceeds to the design of two case studies where linear, non-linear static and non-linear dynamic time history analyses were performed in order to investigate the system response.
Following conclusions may be drawn:

- The system is easy to implement and versatile.
- The dissipative fuses are small with a simple detail which facilitates their fabrication, installation and removal.
- Inelastic deformations are restricted to the beams/pins leaving all other structural members of the main frame and the system (beams, columns) respond elastically.
- The FUSEIS1-1 system has high ductility and its behavior factor is proposed as 5.
- The FUSEIS1-2 system has high overstrength and its behavior factor is proposed as 3.
- The system exhibits a self-centering behavior with minimal residual drifts allowing for immediate occupancy after earthquake. For its confirmation more studies are needed.

6. ACKNOWLEDGMENTS

Part of the presented research was done in the framework of the European Research Project RFSR-CT-2008-00032: “Dissipative devices for seismic resistant steel frames (Fuseis)”. The financial contribution of the Research Fund for Coal and Steel of the European Community is gratefully acknowledged. The other partners of the project were RWTH Aachen University, Germany, Politecnico di Milano, Italy, Instituto Superior Tecnico Lisbon, Portugal and SIDENOR S.A, Greece.

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ΑΝΤΙΣΕΙΣΜΙΚΗ ΣΥΜΠΕΡΙΦΟΡΑ ΠΛΑΙΣΙΩΝ ΜΕ ΤΑ ΚΑΙΝΙΤΟΜΑ ΣΥΣΤΗΜΑΤΑ - FUSEIS 1

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ΠΕΡΙΛΗΨΗ

Στους σύγχρονους αντισεισμικούς κανονισμούς προβλέπεται η εμφάνιση ανελαστικών παραμορφώσεων σε ζώνες απορρόφησης ενέργειας κατά τη διάρκεια του σεισμού σχεδιασμού και επιτρέπονται βλάβες μικρής έκτασης σε συγκεκριμένα μέλη. Στο πλαίσιο του Ευρωπαϊκού ερευνητικού προγράμματος "FUSEIS" αναπτύχθηκαν δύο καινοτόμα αντισεισμικά συστήματα και κανόνες σχεδιασμού τους (Design Guide). Η αντισεισμική συμπεριφορά κτιρίων με το σύστημα FUSEIS 1 έχει μελετηθεί πειραματικά και αναλυτικά. Το σύστημα αποτελείται από δύο ισχυρούς στύλους σε μικρή απόσταση, συνδεόμενους με οριζόντιες δοκούς καθ’ ύψος του ορόφου. Οι δοκοί μπορεί να είναι συνεχείς FUSEIS1-1 μεταξύ των υποστυλωμάτων ή εναλλακτικά να διακόπτονται και να συνδέονται με πείρους στο μέσο FUSEIS1-2. Τα αποτελέσματα των πειραματικών διερευνήσεων του συστήματος παρουσιάστηκαν στο 7ο Εθνικό Συνέδριο Μεταλλικών Κατασκευών. Στην παρούσα εργασία παρουσιάζονται αποτελέσματα μη-γραμμικών στατικών και δυναμικών αναλύσεων και προτείνονται κατάλληλοι συντελεστές συμπεριφοράς. Το σύστημα συνδυάζει την αντοχή με τη δυσκαμψία και την πλαστιμότητα και υπό ορισμένες προυποθέσεις είναι σε θέση να επαναφέρει το κτίριο στην αρχική του θέση μετά το σεισμό (self-centering).