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SEISMIC BEHAVIOUR OF INNOVATIVE ENERGY DISSIPATION SYSTEMS FUSEIS 1-1

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Abstract. *After strong earthquakes conventional frames used worldwide in multi - storey steel buildings (e.g. Moment resisting, Concentric/Eccentric braced frames) are not well positioned according to reparability. It is therefore advisable to develop structural systems that are simple to repair, i.e. to introduce the reparability as a new property. Two innovative systems for seismic resistant steel frames incorporated with dissipative fuses were developed within the European Research Program "FUSEIS". The FUSEIS systems are able to dissipate energy by means of inelastic deformations in the fuses and combine ductility and architectural transparency with stiffness. In case of strong earthquakes damage concentrates only in the fuses which are exchangeable. Repair work after such an event, if needed, is limited only to replacing the fuses. Experimental and numerical investigations were performed to study the response of the FUSEIS system.*

The first system, FUSEIS1, resembles a shear resistance wall and is composed of two closely spaced strong columns, rigidly connected to multiple beams. The beams run from column to column, FUSEIS 1-1, or alternatively are interrupted and connected by short pins, FUSEIS 1-2. In the second system, FUSEIS2, the devices are made by introducing a discontinuity on the composite beams of a moment resisting frame and connecting the two parts through steel plates.

This study presents the experimental research on overall frames with FUSEIS1-1 and numerical models calibrated on the experimental results. These models provided the design rules and verified the good seismic performance of building frames with FUSEIS1-1.

1 INTRODUCTION

Earthquakes lead frequently to damages to a large extent. Modern seismic codes allow for inelastic deformations in dissipative zones during design earthquakes, accepting damages to a certain extend in the relevant structural parts. As past experience shows, repair works are needed after strong earthquakes, which are usually larger than the design one. Structural systems that are easily repairable-replaceable, while maintaining the benefits of high ductility, are obviously beneficial in seismic regions. Therefore innovation in developing new seismic resistant systems is highly appreciated.

Steel-concrete composite systems (also called mixed or hybrid systems) have seen widespread use in recent decades because of the advantages against conventional construction. The optimal use of two materials and their individual properties result in a very efficient and economical structural solution. Reinforced concrete is inexpensive, massive and stiff, while steel members are strong, ductile, lightweight and easy to assemble. They also give the designer the freedom to form rigid or flexible connections and put additional bracing systems or shear walls.

Conventional systems have advantages and disadvantages. Despite their structural efficiency, conventional frames are not well suited for reparability after a strong earthquake event and this is a new property to introduce. In moment resisting frames the beams are the dissipative elements. They, or their end connections, have been proven to require repair after strong seismic events. However, they are elements resisting gravity loading and difficult to repair. Similar conditions apply to eccentric braced frames where the links, the short parts of the beams between braces, have to be repaired. In concentric braced frames, it is the braces that shall develop inelastic deformations. Damage is therefore expected in the braces, which are long and heavy elements, difficult to handle and repair. The main advantage of FUSEIS 1-1 is that inelastic deformations are strictly concentrated and controlled in zones that constitute easily replaceable fuses. Additionally they offer:
a) easiness in fabricating the fuse
b) maximum energy dissipation in cyclic behaviour
c) minimum number of exchangeable parts
d) reduction of weights, costs and difficulties in replacing the fuse parts.

This study presents the research on FUSEIS1-1, while FUSEIS1-2 is presented in detail in paper “SEISMIC BEHAVIOUR OF INNOVATIVE ENERGY DISSIPATION SYSTEMS FUSEIS 1-2”.

2 DESCRIPTION OF FUSEIS 1-1

This innovative system resembles a shear wall, having the additional advantages of energy dissipation through plastic deformation of the beams and ease of repair and even replacement if necessary. It is composed of two closely spaced strong columns, rigidly connected to multiple beams. The beams run from column to column (FUSEIS 1-1).

The system resists lateral loads as a vertical Vierendeel beam, mainly by combined bending and shear of the beams and axial forces of the columns. The dissipative elements of the system are the FUSEIS beams which are not generally subjected to vertical loads, as they are placed between floor levels.

The seismic resistance of a building may be obtained by appropriate provision of a number of such systems in the relevant directions (Figure 1). When beam-to-column connections of the building are formed as simple, this system provides alone the seismic resistance of the building. When the connections are rigid or semi-rigid, it works in combination with the overall moment resisting frame. In any case the connection between the floor beams to the FUSEIS columns shall be formed as simple or semi-rigid.

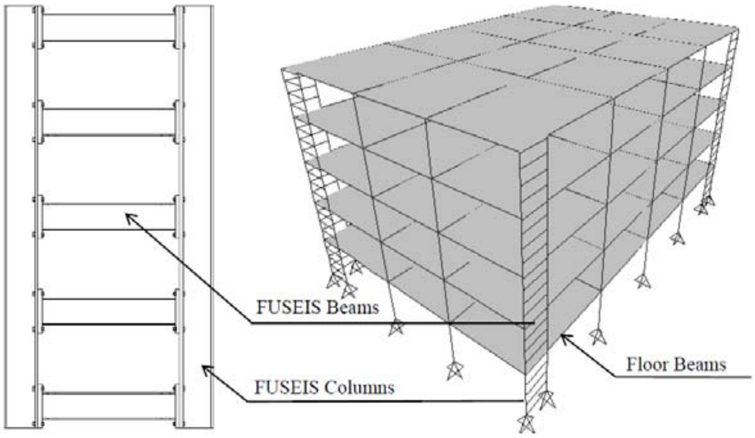


Figure 1: FUSEIS 1-1 system in a building

The fuses-to-column joints are formed as rigid to enable the Vierendeel action and are designed to have sufficient overstrength in order to achieve energy absorption only in the fuses. Bolted end-plate connections which enable an easy replacement of the beams should be used. Considering a typical floor height of 3,4 m, four or five beams may be placed per storey. The beam height depends on the required stiffness with the provision to leave the necessary vertical spacing between them. Additionally, the use of one fuse between columns just above foundation level prevents the moment transfer to the foundation.

FUSEIS columns may be of open or closed section. Open sections are more beneficial, since they offer an easier connection to the beams. When closed sections are used, a T-section can be welded to it to offer the advantage of easier connection.

FUSEIS beams may have hollow sections or open sections (I- or H- sections). To achieve a sequential plasticization of the fuses, beam sections may vary between floors, following the increase of storey shear from the top to the base of the building. In addition, beams may also vary within the floor, either in respect to their cross-sections or to their length.

Aiming to lead the plastic hinge to form away from the connection area and protect the beam to column connections of the system against fracture, beam flanges are reduced near the ends. Constant, tapered or radius cut shapes are possible to reduce the cross sectional area. As an alternative, the connection region could be strengthened by means of additional plates.

3 FULL SCALE TESTS ON FRAMES WITH FUSEIS 1-1

Analytical and experimental investigations on individual FUSEIS 1 -1 devices, i.e. on single beams, were performed in the Institute of Steel Construction of Aachen University (RWTH) which provided an insight of the local behaviour of the system. Full scale tests on a multi-storey building frame were therefore required in order to obtain a more realistic overview of the building's resistance under a seismic event. Six full scale tests on frames with FUSEIS1-1 were conducted in the Steel Structures Laboratory of NTUA. The tests were performed in an experimental rig, available in the Laboratory.

The experimental setup consisted of the test rig (Figure 2), a computer controlled hydraulic cylinder and the test frame. The test frame was designed as a part of a real building frame and consisted of two closely spaced strong columns pin jointed at both ends, rigidly connected to five beams that run from column to column. The dimensions of the structural elements were defined after analysis and seismic design of a complete building frame according to the provisions of EC3 [4] and EC8 [5]. The height of the frame was 3.40 m, the distance (L) between the centrelines of the columns was 1,50m and the length (l) of the fuses varied depending on the stiffness required.

The behaviour of a frame depends on the stiffness and strength of the specimens. Therefore the use of identical beam sections within a storey leads to the simultaneous yielding of all beams. The scope of the tests was to achieve a sequential yielding of the specimens, thus providing a better behaviour under cyclic loading and as a result a better response of the building during a broader spectrum of seismic events. The sequential yielding can be achieved either by using beams with the same cross section but with varying lengths, or by using beams with the same length and varying cross sections. In order to be able to compare and evaluate the behaviour of different types of sections, specimens of similar stiffness and resistance were selected for all tested frames.

Figure 3 shows the FUSEIS 1-1 system tested. The columns were stiffened by adding stiffening T-sections at both sides in order to remain elastic. The connection of the specimens to the column was obtained with the use of receptacle steel plates. The specimens were beams of IPE, SHS and CHS sections. In order to allow a plastic hinge formation away from the beam-column connection, the beam fuses had reduced sections at the ends (RBS). The beams were provided with end plates and were bolted to the columns, to ease the replacement of the fuses after the test.

Based on initial analyses on a variety of RBS sections, the RBS parameters a (distance from face of column) and b (length of RBS) were the same 50mm and 75mm respectively for all fuses beams, while the depth of cut, g was approximately 30% of the beam flange according to FEMA350 [6] and EC8 [7] provisions. The RBS types are also given in Figure 3.

Figure 2: Test rig

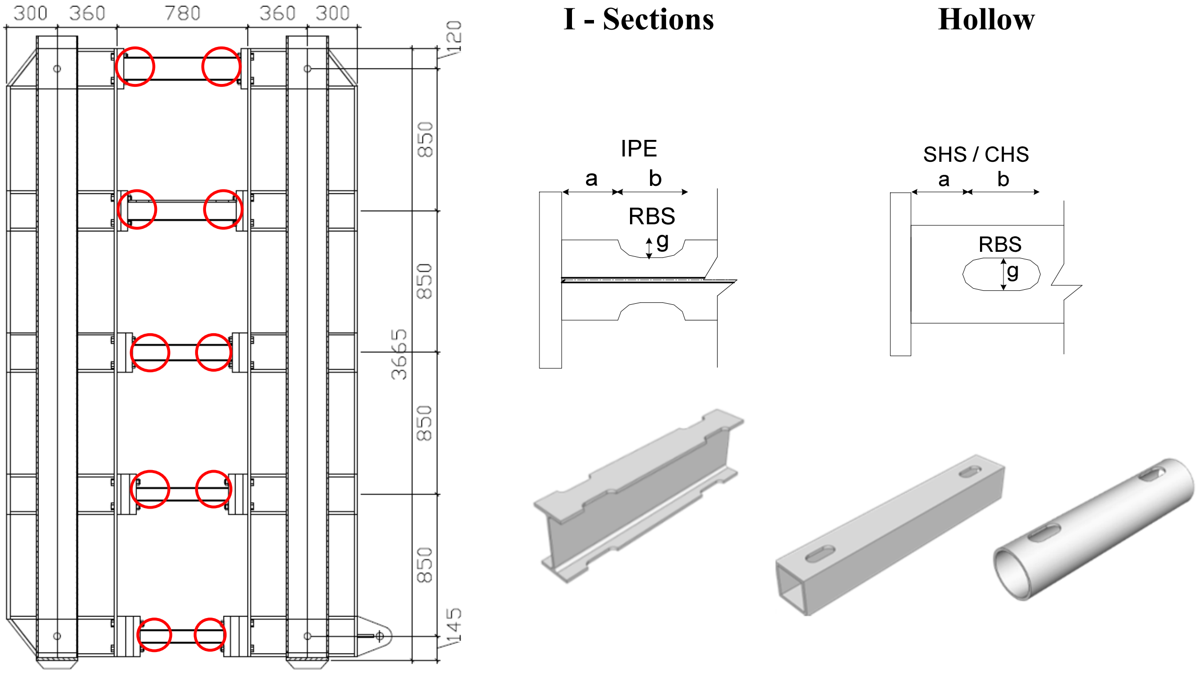


Figure 3: Tested system and beam specimens

The characteristics of the specimens, meaning the specimens sections and lengths, are given in Table 1.

Specimens with variable lengths						
Test	Type	L (mm)	L (mm)	L (mm)	L (mm)	L(mm)
M1	IPE160	700	650	600	550	500
M2	SHS120x8	700	650	600	550	500
M3	CHS 141,3x6	700	650	600	550	500
Specimens with variable sections						
Test	L	Type	Beam	Beam	Beam	Beam
A1	600	IPE	180	160	140	120
A2	600	SHS	140/8	120/8	100/8	80/6
A3	600	CHS	168.3/6	141.3/6	114.3/6	88.9/6

Table 1: Test Matrix

A basic parameter of the design of energy dissipation systems is the steel grade. Low grade steel such as S235 was preferred. Material tests for the test specimen were conducted in order to determine the mechanical characteristics of steel.

The actuator was a computer controlled hydraulic cylinder with a maximum capacity of ±250mm positioned horizontally between the column and a specially designed base via two circular hinges in order to transfer only horizontal forces to the frame. It was calibrated appropriately at the beginning of each test, meaning that it was set to the middle course position.

The above setup (the test rig, a pair of columns and the actuator) and the load application system remained the same for all the tests. The only modification between different tests concerned the replacement of the fuses. The basic problem during the assembly of beam fuses were the structural imperfections (initial out-of-plane imperfections of the plates, weldings, transportation, residual deformations transferred from one test to the next). Gaps were formed at the contact area between the columns and the receptacle plates, for this reason filler plates were used. Additionally, the surface of the plates in some cases was not plane which was solved by tightening the bolts.

3.1 Loading Procedures and Experimental measured data

Regarding the loading procedures of the experiments, the cyclic loading protocol was defined in terms of the European ECCS – procedures [8]. The yield displacement of the specimens is the reference value that defines the loading protocol. The testing procedure was done in displacement control. The displacement was fed from the computer to the controller, which then displaced the hydraulic actuator to match the demand. The imposed displacement increases linearly, with constant velocity 1,5mm/sec and a short pause 5sec between the cycles of the same amplitude and 10sec when the amplitude increases. The cyclic loading protocol was followed by constant amplitude cycles of 5% until fracture.

Figure 4 shows the cyclic loading protocol, with increasing inter-storey drift, and the correspondent horizontal displacement imposed by the actuator to the frame. The data measured during the experimental investigations were the following:

- Displacement of the cylinder piston with an electronic displacement transducer.
- Hydraulic load applied by the cylinder, with the load cell at the end of the piston.
- Differential displacement of beam ends–shear displacement vertical to the beams with electronic displacement transducers (LVDTs) with a maximum capacity of ±50mm.

No of cycles	Frame Displ. (mm)	Interstorey drift (%)
3	2,55	yield
3	5,10	0,15
3	8,50	0,25
3	17,00	0,50
3	25,50	0.75
3	34,00	1.00
3	51,00	1.50
3	68,00	2.00
3	85,00	2.50
3	102,00	3.00
3	119,00	3.50
3	136,00	4.00
3	153,00	4.50
Until failure	170,00	5.00

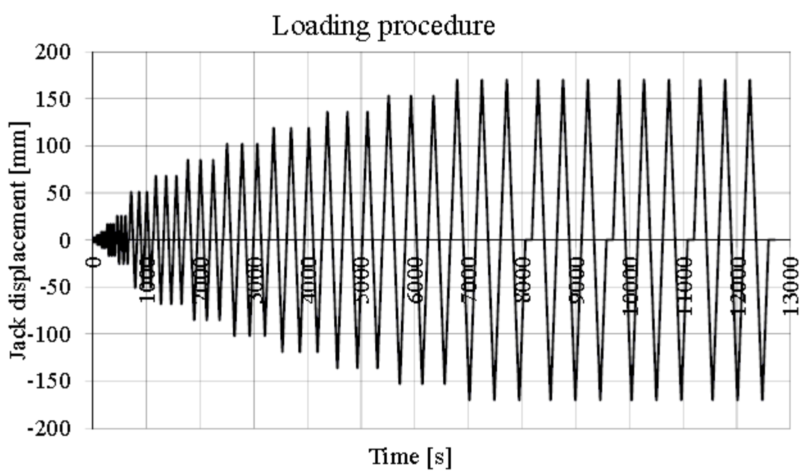
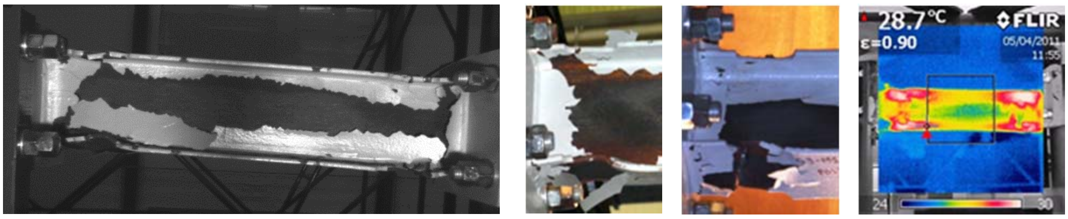


Figure 4: Applied loading protocol

3.2 Test results

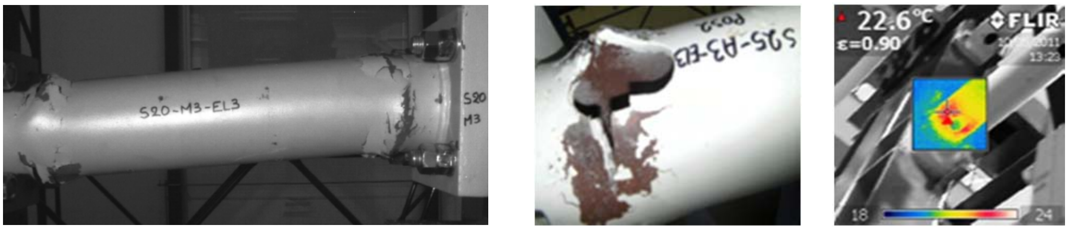
All test specimens showed good plastic deformation capacity. It is remarkable that the system's resistance kept increasing after the initial and during the subsequent plastifications, mostly due to hardening, without losing its stability. Plastic deformations took place within the fuse devices only, while the columns remained elastic and undamaged until the completion of the last test. The design and stiffening of the test rig proved to be successful since they remained almost perfectly rigid. The time required for replacing one fuse device was approximately 60 minutes. At the end of the test, depending on the condition of the fuses, some fuses were cut and removed and others – completely damaged – were instantly dispatched. During the tests, the receptacle and the filler plates did not have any significant damage and were reused.

Yielding of the beam specimens started at the RBS area. Ductile fractures were observed at the curve of the RBS and finally the height of the beam shrank at the same position, as the deformation of the specimens got larger. After attainment of maximum strength, the load degraded gradually with distortion of the RBS. All frames reached an interstorey drift between 2 and 4%. Generally, for the hollow sections it was observed that the load degraded more gradually compared to the IPE sections due to their torsional stiffness and additional resistance provided by the webs. Specifically, the CHS sections behaved even better as the plastification was distributed along the section circumference. The hinge formation during the experiments is clearly shown at the photos taken by an infrared camera, higher temperature values are observed at the RBS. Figure 5 shows photos of the deformed beam fuses.



a) IPE beams

b) SHS beams



c) CHS beams

Figure 5: Photos of the deformed fuseis beams

In some cases, during the first cycles the behaviour of the system was affected significantly by the construction imperfections of the surfaces between the columns and the receptacle plates. This phenomenon diminished at later cycles. The hysteretic diagrams have pinching in the response due to the slippage of the bolts and the construction imperfections.

The force–drift diagrams for all tests are presented in Figure 6.

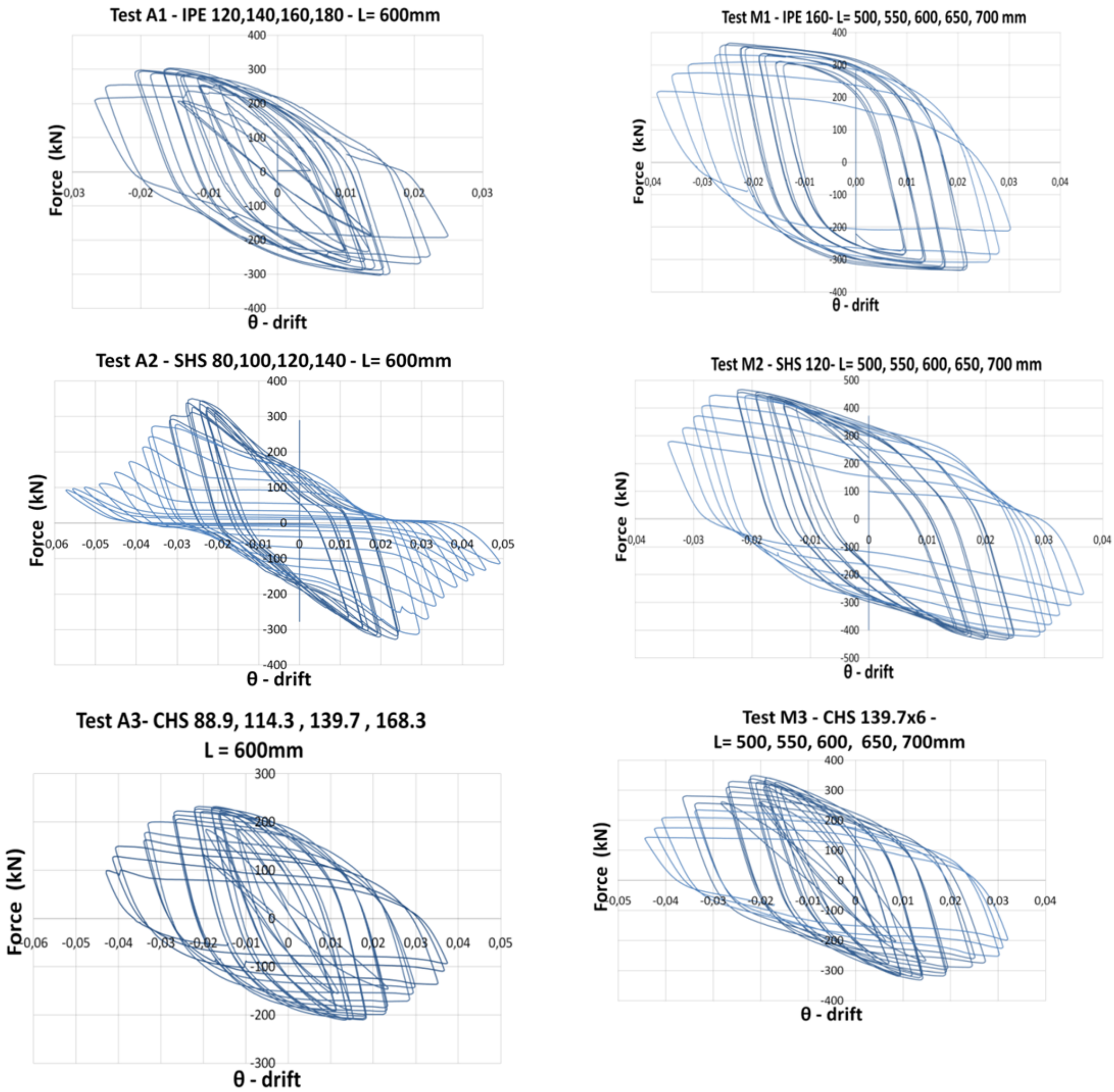


Figure 6: Hysteretic response of tested frames

3.3 Development and calibration of simple models

The commercial software SAP2000 was used and provided simple engineering models able to simulate the behavior of frames with fuses accurately. The FUSEIS elements were represented by appropriate beam FE-elements. Rigid zones were provided from column centres to column faces to exclude non-existent beam flexibilities. Further on, the net length of the beam fuses was subdivided to 5 zones that represent the full sections (ends – middle) and the RBS sections (Figure 7). In this manner, the true system flexibility and strength was accounted for. The remaining structural elements were simulated as usual. Beam-to-column joints were represented as rigid in accordance to the connection detailing of the test.

The non-linear lateral behaviour of the 2D test frames was evaluated by incremental pushover analyses (Figure 7). The target displacement corresponds to the maximum displacement (170mm) of the experimental tests. Since the ductile elements are the FUSEIS beams, potential plastic hinges were inserted at the ends of the beam RBS-sections. Table 2 summarizes the non-linear hinge parameters that comprised the most suitable approach, for each type of fuse section.

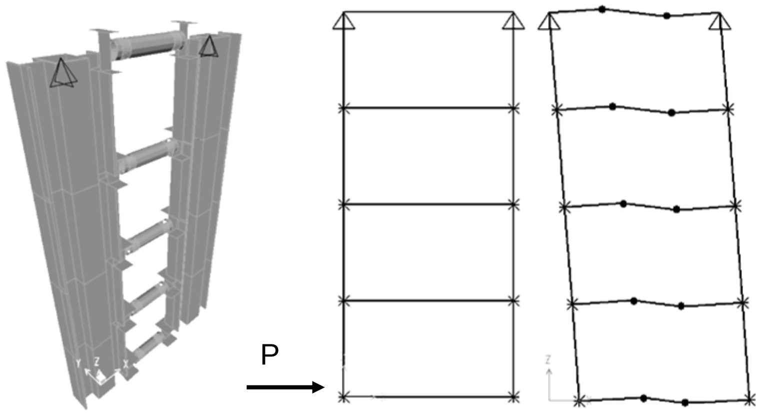


Figure 7: Pushover loading and hinge formation

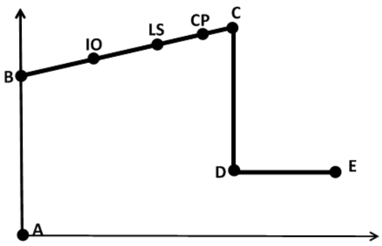
HINGE PROPERTIES (α_{pl} =shape factor)

	IPE		SHS		CHS	
Point	**M/SF**	**Rot./SF**	**M/SF**	**Rot./SF**	**M/SF**	**Rot./SF**
E-	-0,6	-45	-0,4	-30	-0,2	-30
D-	-0,6	-40	-0,4	-25	-0,2	-25
C-	- α_{pl}	-40	- α_{pl}	-25	- α_{pl}	-25
B-	1	0	-0,6	0	-1	0
A	0	0	0	0	0	0
B	1	0	0,6	0	1	0
C	α_{pl}	40	α_{pl}	25	α_{pl}	25
D	0,6	40	0,4	25	0,2	25
E	0,6	45	0,4	30	0,2	30

ACCEPTANCE CRITERIA

	IPE	SHS	CHS
IO	15	5	6
LS	25	12	10
CP	35	18	16

SF are scale factors corresponding to the moment and rotation yield values



M–θ Relation

Table 2: Non-linear hinge parameters for IPE, SHS, CHS

4 LINEAR STATIC ANALYSIS AND DESIGN OF FRAMES WITH FUSEIS 1-1

Based on the understanding gained from the full scale tests on frames with FUSEIS 1-1 2D building frames with fuses were designed and analyzed using the general purpose software Code SAP 2000. In addition to conventional design, non-linear static analyses were carried out to estimate the behaviour factor of the system. Non-linear analyses based on the response of the devices which have been experimentally investigated.

Several building frames with fuses were designed according to the provisions of EC 3 and EC 8. Additional rules given in the relevant Design Guide were applied to ensure that yielding, takes place in the FUSEIS beams prior to any yielding or failure elsewhere. The FUSEIS beams, able to dissipate energy by the formation of plastic bending mechanisms, were designed to resist the forces of the most unfavourable seismic combination.

Since, this system module is used to ensure lateral stiffness and seismic energy dissipation for building structures, the frames were dimensioned to cover the Seismic zones corresponding to the Greek earthquake intensity conditions 0.16g, 0.24g,0.36g. In order to reflect the findings obtained during the experimental program the cross sections used were similar to the ones tested (IPE, SHS, CHS) and were reduced near the ends. These models provided an optimal approximation of the behaviour of a steel building.

A typical 2D building frame, part of a five-storey composite building, was used for all the cases examined. The composite building consisted of similar frames at 8m axial distance which was the effective width for both the vertical loads and the lateral mass during earthquake loading. The bays of the main frame were 6.0 m. The beams were composite and the thickness of the slab was 15cm. The effective width of the composite beams was equal to 1.5m in accordance with EC2.

The FUSEIS system consisted of two closely positioned vertical hollow strong columns, jointed together with five horizontal beams in a tight arrangement. The center line distance of the columns was 2m. The simulation concept for the fuses was the same as described in Section 3.3. The column bases were pinned to limit yielding at the foundation level. In order to introduce partial fixity conditions between the composite beams and the columns, rotational springs were assigned at the composite beams' ends according to EC 3 part 1.8, §6.3 and EC 4 part 1 (Annex A).

Vertical loading was equal for all storeys and masses were lumped at the joints. Steel grade of the non-dissipative structural members was S 355 and for the dissipative elements (the fuses) S 235 to eliminate the risk of possible overstrength of the dissipative elements. A preliminary value of the behavior factor q = 5 was employed that is discussed later.

The described layout was followed for all nine frames. All models were designed according to the provisions of EC 3 for ULS and SLS. Limitations on 2nd order effects according to EC8 were also taken into account. In all cases the value of the interstorey drift sensitivity coefficient θ was between 0,1 and 0,2 indicating that a frame with FUSEIS system is flexible. The relevant seismic action effects are then multiplied by a magnification factor 1/(1 - θ). Interstorey drifts were limited to 0,0075 (buildings having ductile non-structural elements) - EC 8. This was the crucial condition for the determination of the controlling structural members (FUSEIS beams and columns).

Design was based on multi modal response spectrum analysis, using a linear-elastic model of the structure and a design spectrum. The analysis showed that the first mode of vibration activated approximately 75% of the mass, while few more modes were needed to reach 90% as required by Codes. Figure 8 presents the building frame with SHS fuses for PGA=0,36g .

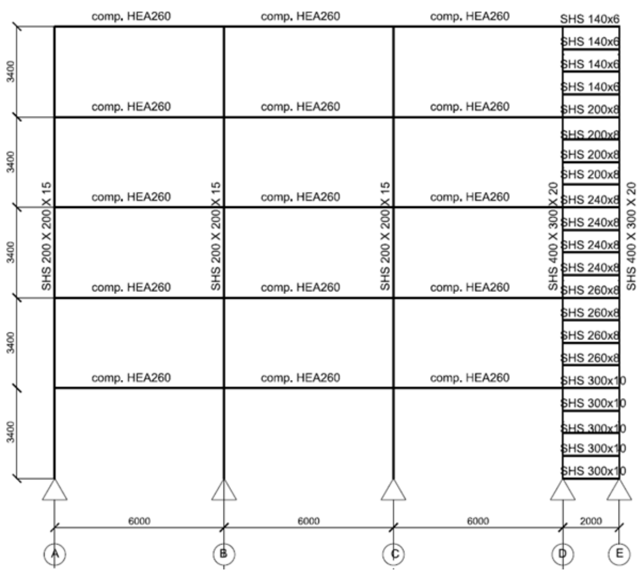


Figure 8: Typical 2D building frame

5 NON- LINEAR STATIC ANALYSIS (PUSHOVER) AND EVALUATION OF THE BEHAVIOUR FACTOR

The structural models used for elastic analysis were extended to include the response of structural elements beyond the elastic state via a non-linear static analysis (Pushover). The principal objective of this investigation was to estimate the behaviour factor q.

The analysis was carried out under conditions of constant gravity loads and monotonically increasing lateral loads. The results of the analysis according to the fundamental mode of vibration, 1st mode, are presented hereafter. The analysis was based on the assumption that the mode shape remains unchanged after the structure yields, P –Delta effects were also taken into account.

In pushover analysis, the behaviour of the structure is characterized by a capacity curve that represents the relationship between base shear force and top displacement and the demand curve for the design earthquake which was based on ATC-40. The performance point is defined as the intersection of the demand curve with the capacity curve (Figure 9).

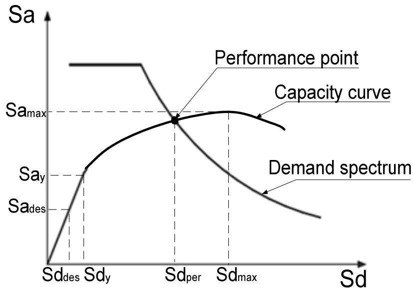
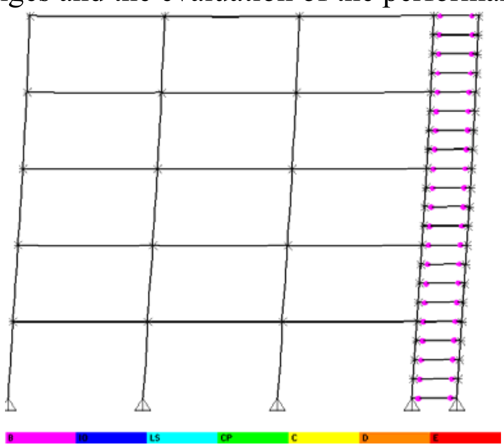


Figure 9: Definition of the performance point

In the implementation of pushover analysis, correct modeling of the hinges is crucial. The model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities. Non-linear hinge elements were assigned at the ends of the reduced parts and their properties were derived from the calibrated models of the tests (Table 2). At the start of the calculations, potential plastic hinges were also inserted at the ends of the composite beams, the columns and the system columns to check if they also behave inelastic during the seismic event. These analyses showed that at the performance point plastic hinges formed only in the reduced parts of the fuses so in subsequent analyses potential hinges were introduced only in the fuses. The distribution of plastic hinges and the evaluation of the performance point are given in Figure 10.



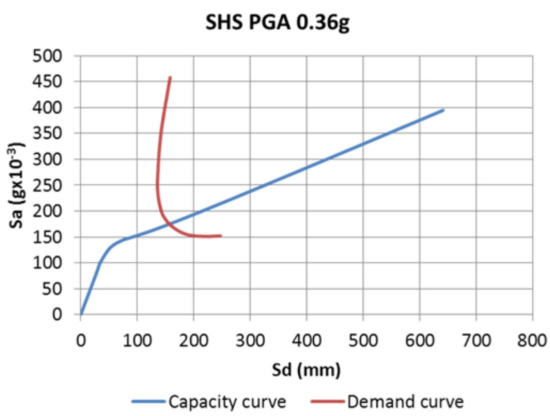


Figure 10: Hinges at the Performance Point and Evaluation of performance point

As expected the weak beam strong column concept was fulfilled for all frames studied.

Besides the assessment of the structural performance of the building frames, pushover analysis also offered the possibility to estimate their behaviour factor. Due to the flexibility of the system $T_1 \geq T_c$ the 'equal displacement rule' was applied. Figure 11 shows the typical pushover response curve for the evaluation of behaviour factor.

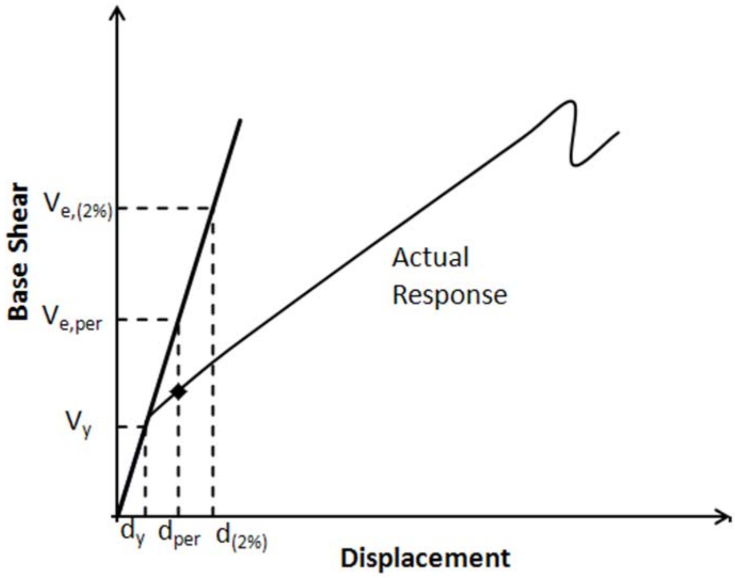


Figure 11: Typical pushover response curve for evaluation of behaviour factors

The behaviour factor, q, accounts for the inherent ductility and overstrength of a structure and may be generally expressed in the following form taking into account the above two components:

$$q = q_\mu \cdot \Omega \tag{1}$$

The structure ductility, q_μ, is defined in terms of the Elastic Base Reaction that corresponds to 2% drift (V_e) to the Idealised Yield Strength - First hinge (V_y) (Figure 11), as following:

$$q_\mu = \frac{V_e}{V_y} \tag{2}$$

The ID equal to 2% was determined experimentally and corresponds to the maximum force reached.

The structural overstrength is defined as the ratio of the Idealised Yield Strength - First hinge V_y to the Design Strength (V_d), as following:

$$\Omega = \frac{V_y}{V_d} \tag{3}$$

The Design Strength (V_d) was based on the fundamental vibration mode which had the largest participation to the vibrating mass and was determined from $V_d = \eta \cdot M \cdot S_d(T)$ where n is the modal participating mass ratio, M is the total mass and $S_d(T)$ is the spectral acceleration that derives from the design spectra for the fundamental mode.

The calculated q factors for each type of cross section are given in Table 3. The system's behaviour is characterised by high values of ductility, its overstrength is dependent on the intensity of the design earthquake. All q values are above 5, the value that was initially considered for the design. However, when high values of q are employed the interstorey drift sensitivity coefficient θ increases significantly due to 2nd order effects. Therefore, a maximum value of q equal to 5 is proposed for the design of buildings with FUSEIS system.

FUSEIS BEAMS	0,16g			0,24g			0,36g		
	q_μ	Ω	q	q_μ	Ω	q	q_μ	Ω	q
IPE	3.31	2.99	9.91	3.31	1.97	6.51	3.81	1.26	4.79
SHS	3.97	2.26	8.99	3.97	1.51	6.01	5.79	1.09	6.31
CHS	3.18	3.52	11.19	3.18	2.31	7.35	3.82	1.31	5.00

Table 3: Non-linear hinge parameters for IPE, SHS, CHS

The calculated q factors are summarized in Figure 12.

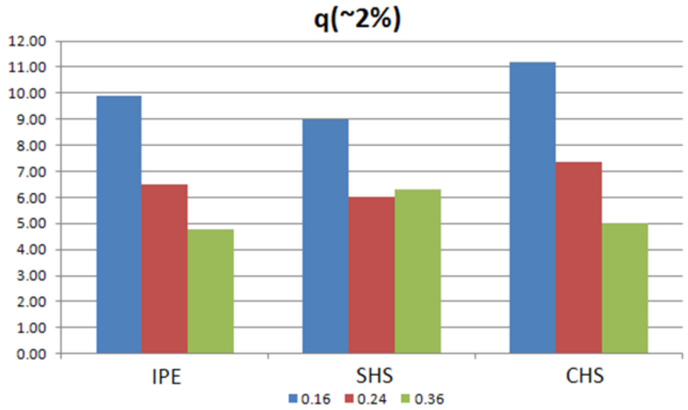


Figure 12: Evaluation of q factors

6 CONCLUSIONS

The above study illustrates the successful application of the innovative FUSEIS system. The following observations are worth noting:

a) The system resists lateral loads as a vertical Vierendeel beam.

b) Inelastic deformations are strictly limited to the dissipative elements preventing the spreading of damage into rest of the structural members (slab, beams, columns). The devices and the frames with the devices have a very good behaviour: strong, stiff, large capacity of energy absorption.

c) It may be designed as more flexible/rigid depending on the section types and their distribution between floor levels. The number of stories and supporting weight strongly affects the required sections and geometry. The seismic resistance of a building may be obtained by appropriate provision of a number of FUSEIS systems in the relevant directions.

d) It consists an architecturally versatile solution for the lateral stability of building structures compared to the braced frames as they can be positioned in small areas of the building and do not interrupt the architectural plan. They can also constitute visible parts of the building indicating its seismic resistant system

e) Sequential plastification may be allowed by appropriate selection of the sections of the dissipative elements.

f) The dissipative elements are easily replaceable if they are damaged after a strong seismic event, since they are small and are not part of the gravity loading resistant system. The assembling and disassembling is easy from a practical point of view.

g) The proposed q-factor for buildings with FUSEIS1-1 is 5.

h) Code relevant design rules for the seismic design of frames with dissipative FUSEIS and practical recommendations on the selection of the appropriate fuses as a function of the most important parameters and member verifications have been formulated and are included in a Design Guide.

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REFERENCES

[1] Vayas, Ph. Karydakis D. Dimakogianni, G. Dougka, C. A. Castiglioni, A. Kanyilmaz et al. Dissipative devices for seismic resistant steel frames - *The FUSEIS Project, Final Report. Research Programme of the Research Fund for Coal and Steel*, 2012.

[2] Vayas, Ph. Karydakis D. Dimakogianni, G. Dougka, C. A. Castiglioni, A. Kanyilmaz et al. Dissipative devices for seismic resistant steel frames - *The FUSEIS Project, Design Guide. Research Programme of the Research Fund for Coal and Steel*, 2012.

[3] Castiglioni, C. A., Calado, L., Kanyilmaz A. Experimental analysis of seismic resistant composite steel frames with dissipative devices. *Journal of Construction Steel Research*, 2012.

[4] EN1993-1-1, Eurocode 3: Design of steel structures - *Part 1-1: General rules and rules for buildings. Brussels.* Comitee Europeen de Normalisation (CEN), 2003.

[5] EN1998-1-1, Eurocode 8: Design of structures for earthquake resistance – *Part 1-1: General rules, seismic actions and rules for buildings.* Brussels: Comitee Europeen de Normalisation (CEN), 2003.

[6] FEMA – 350: *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*, 2000.

[7] EN1998-3, Eurocode 8: *Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings*. Brussels: Comitee Europeen de Normalisation (CEN), 2005.

[8] ECCS document: *Recommended testing procedure for assessing the behaviour of structural steel elements under cyclic loads.* Technical committee 1: structural safety and loadings, Technical working group 1.3: seismic design, 1986.

[9] Calado, L., Castiglioni, C. A. *Steel beam-to-column connections under low-cycle fatigue: Experimental and numerical research*. In: Proceedings of XI WCEE; 1996.

[10] SAP2000, CSI Computers and Structures Inc., www.csiberkeley.com

[11] FEMA – 356: Prestandard and Commentary for the seismic rehabilitation of Buildings, 2000.