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SEISMIC BEHAVIOUR OF INNOVATIVE ENERGY DISSIPATION SYSTEMS FUSEIS 1-2

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Abstract. *Modern seismic codes allow for inelastic deformations in dissipative zones during design earthquakes, accepting damage to a certain extent in the relevant structural parts. As past experience shows, repair works are needed after strong earthquakes, either less or larger than the design one. Structural systems that are easily repairable-replaceable, while maintaining the benefits of high ductility, are therefore beneficial in seismic regions.*

The scope of European Research Program "FUSEIS" was the development and study of two such innovative systems. The first, FUSEIS 1, consists of a pair of strong columns jointed together by fuses forming a shear resistance wall whereas in the second system, FUSEIS 2, the devices are made by introducing a discontinuity on the composite beams of a moment resisting frame. The main advantage of the "FUSEIS" compared to conventional systems is that inelastic deformations are strictly concentrated in controlled zones. The dissipative elements can be positioned in small areas of the building and do not interrupt the architectural plan as braced do. Another advantage of the system is the possibility of easy installation and removal within the structure.

In order to enhance the stiffness, strength and energy dissipation capacity of FUSEIS 1 system, several fuses are provided within each storey. Depending on the geometry of the fuse two cases were examined: FUSEIS 1-1 and FUSEIS 1-2. In FUSEIS 1-1system the dissipative fuses are beams. In FUSEIS 1-2 the fuses consist of beams cut in the middle and connected by short pins. The dissipative zones in this case are formed within the pins while the other parts of the structure remain elastic and protected.

This study focuses on the research carried out to evaluate the behavior of the FUSEIS 1-2 system and includes the results of the full scale tests conducted in NTUA and application examples of the system on real building frames.

1 INTRODUCTION

Opposite to concrete building frames, the lateral stability and seismic resistance of steel buildings may be obtained by a large variety of structural systems. Indeed, due to the monolithic nature of concrete, beam – to – column joints are generally rigid so that concrete buildings constitute 3D - frames with or without shear walls. This is not the case in steel buildings, where the designer has the freedom to form the connections as rigid or flexible and put additional bracing systems or shear walls.

The conventional systems applied worldwide to multi- storey steel buildings, such as moment resisting frames, concentric and eccentric braced frames and frames stabilized by steel or composite shear walls are difficult to repair when damaged after strong earthquakes. Moment resisting frames are ductile but usually flexible. Concentric braced frames are stiffer but less ductile due to buckling of braces. The properties of eccentric braced frames are something between the other two types.

In the frame of the EU supported project FUSEIS two innovative seismic resistant systems, FUSEIS 1 and FUSEIS 2, were developed [1]. In both systems inelastic deformations, and accordingly possible damage after a strong seismic event, concentrate in small easily repairable-replaceable fuses. The system dissipates energy in the fuses maintaining the benefits of high ductility. Easy mounting-dismounting of the fuses within the structure allows for a quick replacement in case of damage. The optimal dissipative fuse consists an economical solution that offers easiness in fabricating and combines the maximum energy dissipation in cyclic behavior with the minimum number of exchangeable part.

This paper presents the second alternative of the FUSEIS 1 system, named FUSEIS 1-2 where the fuses are small pins bolted to the structure. The investigation on FUSEIS1-1 and application examples will be presented in detail in other papers. FUSEIS 2 is presented in [1, 2, 3]. The overall system and a design guide are given in [1, 2].

2 DESCRIPTION OF FUSEIS 1-2

The proposed system is developed to combine the advantages of moment resisting frames in respect to ductility and architectural transparency and those of braced frames in respect to stiffness. Unlike braced frames, the proposed system does not obstruct the bays of the building (Figure 1). In this system, specific seismic elements are inserted in a steel frame that resist the seismic action and provide stiffness, strength and ductility. The seismic resistant system consists of closely spaced strong columns, rigidly connected to multiple beams that run from column to column (FUSEIS 1-1) or alternatively are interrupted and connected by short pins (FUSEIS 1-2). In order to enhance the stiffness, strength and energy dissipation capacity of the system, several beams are provided within each storey.

Under seismic loading the system behaves similarly to a vertical Vierendeel beam. It is important to note, that the fuses do not generally carry vertical loading. Lateral stability of a building may be provided by this system only by appropriate provision of a number of such systems in the relevant directions (Figure 1). The rigidity of the system module varies depending on the selection of element sections, column axial distance and number of intermediate beams (e.g. four to five fuses may be inserted per storey for a typical floor height). Pin sections may vary between floors, following the increase of storey shear from the top to the base of the building, or within the floor, either in respect to their cross-section dimensions or to their length resulting in sequential plasticization. In addition one fuse is provided between columns just above foundation level to minimize the moment transfer to the foundation and enable the realisation of simple pinned bases for FUSEIS columns.

Floor beam to FUSEIS-column connections are simple, while floor beam-to-column connections in the building frame may be formed as simple, semi-rigid or rigid. The formation of semi-rigid or rigid connections provides lateral stiffness of the building frame so that lateral forces are shared between the building frame and the FUSEIS system.

Open or hollow sections may be employed at the FUSEIS columns. In case of hollow sections the connection to the beams is more complex and a T-section can be welded on to ease the connections.

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 connections give the possibility of an easy replacement after a very strong seismic event.

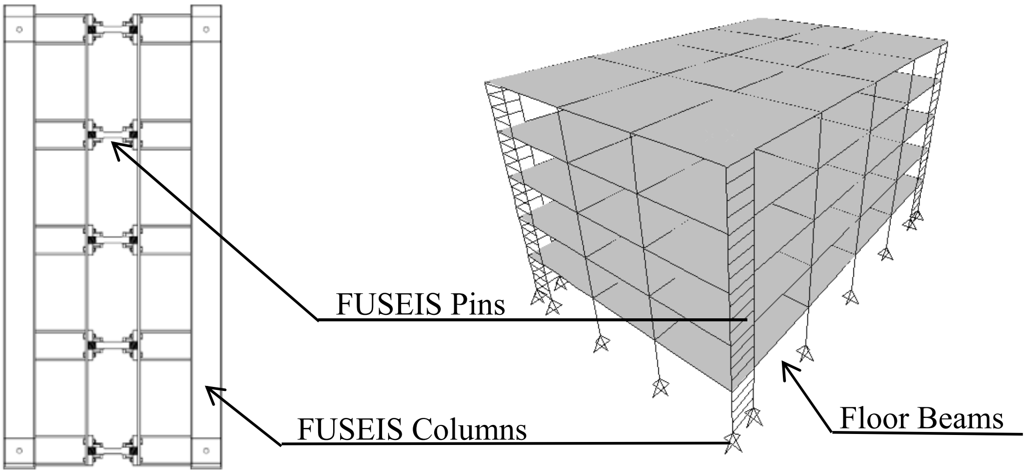


Figure 1: FUSEIS 1-2 system in a building

In FUSEIS1-2, the dissipative zones are the pins that act like fuses. Compared to the other proposed system FUSEIS1-1 their main difference is that the beams are not continuous between two columns. They are cut and connected with the pins (Figure 2). Under strong seismic motion, plastic hinges will develop at the pins that will dissipate a large amount of input energy. The replacement of the fuses is very easy, since it is restricted to the pin. Aiming to lead the plastic hinge formation away from the contact area between the face plate of the receptacles and the pins, the pins diameter is reduced in the middle (similar to the RBS sections). In order to keep the contact area away from the end of the plates, ensuring triaxial stress conditions, pin's diameter decrease starts away from the plate's face and the edges of the plate hole are smoothed (Figure 3). The pins may be circular if the receptacle beams are hollow or rectangular if the receptacle beams are I or H (Figure 2). By appropriate detailing of the pin ends, screwed with inverse directions, their length may be adjusted (Figure 3).

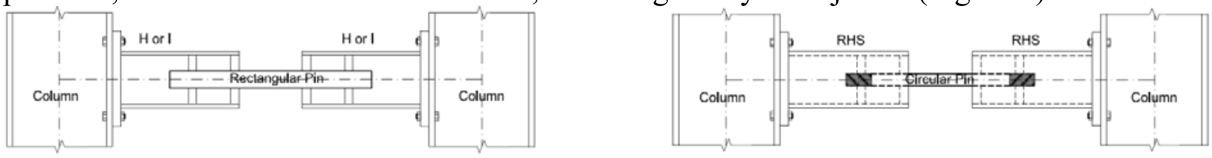


Figure 2: Fuseis 1-2

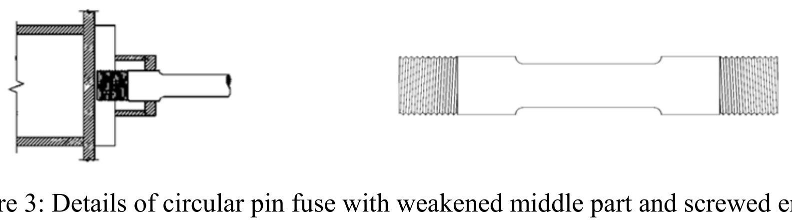


Figure 3: Details of circular pin fuse with weakened middle part and screwed ends

3 FULL SCALE TESTS ON FRAMES WITH FUSEIS 1-2

Numerical and experimental investigations were carried out to study the response of FUSEIS1-2 to cyclic loading. Experimental investigations on individual devices, i.e. on single pins, were performed in the Institute of Steel Construction of Aachen University (RWTH). Based on the understanding gained from these investigations, the necessity to study a real building's performance under a seismic event arose. Two full scale tests on frames with FUSEIS1-2 were conducted in the Steel Structures Laboratory of NTUA.

It is evident that the seismic response of a frame with FUSEIS1-2 depends mainly on the stiffness and strength of the pins. If the same pins are used within a storey all pins are expected to yield at the same time. To achieve a controlled yielding of the pins and a better behaviour of the frame under cyclic loading, the specimens should preferably yield gradually. During the full scale tests, this was realized with the two following ways:

- By using pins with the same cross section but with varying lengths, or
- By using pins with the same length but varying cross sections.

The experimental setup included a resistance space frame – test rig (Figure 4), a computer controlled hydraulic cylinder and the test frame.

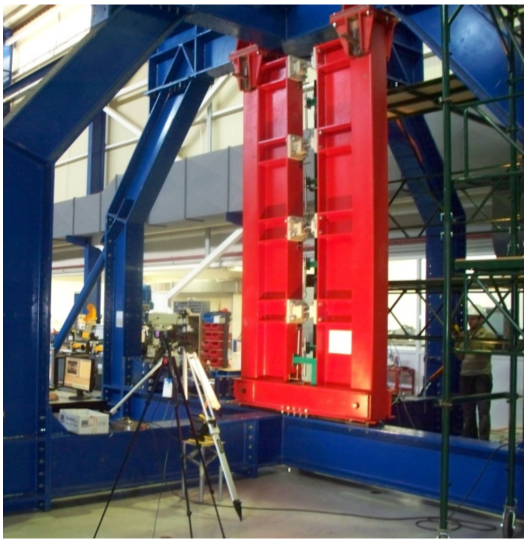


Figure 4: Test Rig

The test frame consisted of two closely spaced strong columns rigidly connected to five fuses as shown in Figure 5. The dimensions of the structural elements corresponded to a real building frame and were defined 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 1,50m, the length (l) of the pins 400mm. The columns of the test frame were pin-jointed at the top and bottom connections and were stiffened by adding stiffening T-sections on their inner side to remain elastic. The fuse consisted of a 400mm pin D60 and two receptacle SHS beams. The pins had a weakened part in the middle aiming to lead the plastic hinge formation within this part. To facilitate mounting and dismounting of the fuse, threads were cut at pin ends in inverse directions to fasten them to the receptacles.

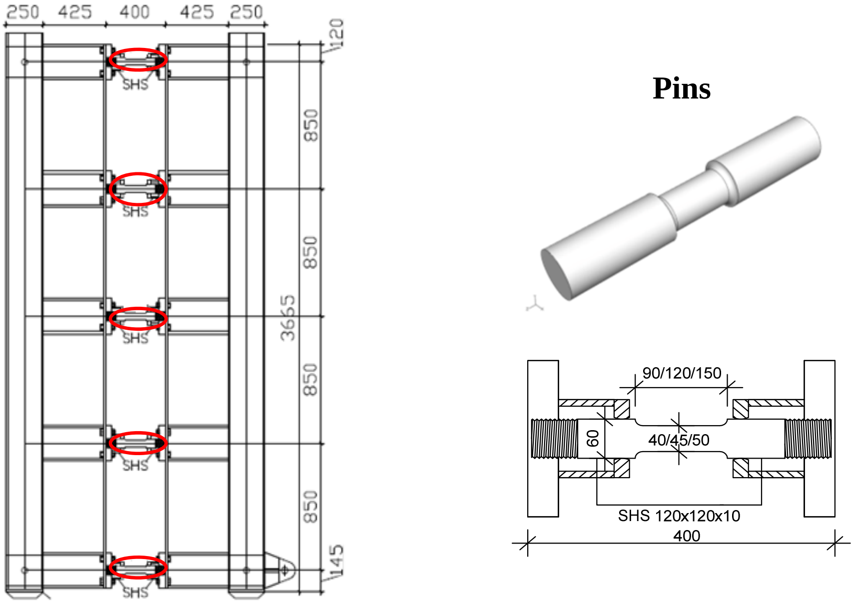


Figure 5: Test Frame & Beam specimens

Based on initial analytical investigations on a variety of reduced sections, three different lengths 90, 120, 150 mm with the same diameter D45 (Test M4) and three different sections with diameters D40, 45, 50 with the same length 120mm of the reduced part (Test M5) were examined. In all cases the diameter's decrease didn't exceed the 30% of the full section based on the provisions of FEMA350 [6] and EC8 [7] for the design of Reduced Beam Sections (RBS).

The actuator was a computer controlled hydraulic cylinder with a maximum displacement capacity of +/-250mm, more than the target displacement of 5% inter-storey drift (=170 mm). It was pin-jointed to the bottom of the column in order to transfer only horizontal forces to the frame and set to the middle course position at the beginning of each test.

In order to determine the material properties of the fuses, material tests were conducted. The range of the determined yielding stresses was usual for steel grade S235.

3.1 Loading Procedures and Experimental measured data

The test procedure was based on the relevant ECCS-recommendation [8]. The cyclic tests were carried out deformation controlled with increasing amplitudes as shown in Figure 6. Starting with an applied displacement of 2.55mm in the axis of the jack the loading was increased up to 170 mm which corresponds to an interstorey drift 5 %.

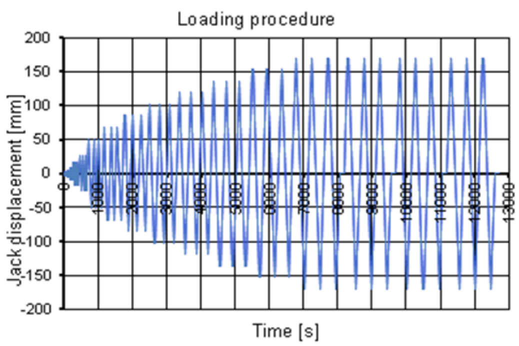


Figure 6: Applied loading protocol

The procedure was continued with amplitudes of 170 mm until total failure. The velocity during the cyclic tests was 1.5 mm/s with a short pause 5sec between the cycles of the same amplitude and 10sec when the amplitude increased.

The data measured during the experimental investigations were: the displacement of the cylinder piston with an electronic displacement transducer, the hydraulic load applied by the cylinder (with the load cell at the end of the piston) and the differential vertical displacement between pin–ends by means of electronic displacement transducers (LVDTs) with a maximum capacity of ±50mm. Special bases were constructed for this purpose. They were bolted to the end plates of the receptacles.

3.2 Test results

All test specimens showed good plastic deformation capacity. It is remarkable that the system's resistance kept increasing after first yield and subsequent plastifications of the pins, due to development of a catenary action in the pins and strain hardening. Plastic deformations took place within the fuse devices, the pins, only while the columns remained elastic and undamaged (Figure 7). The time required for replacing one fuse device was approximately 60 minutes. At the end of the test, the damaged pins were instantly dispatched.

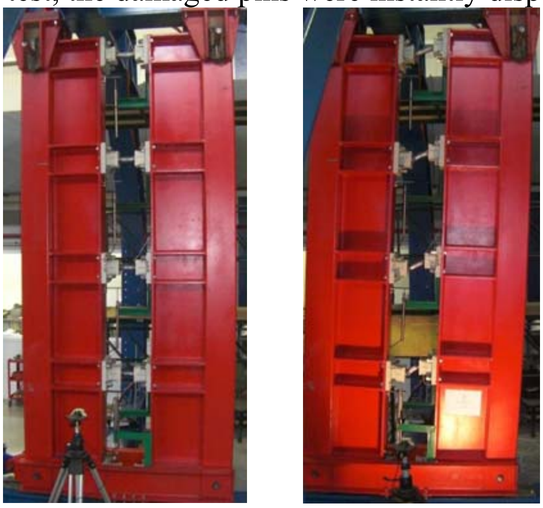
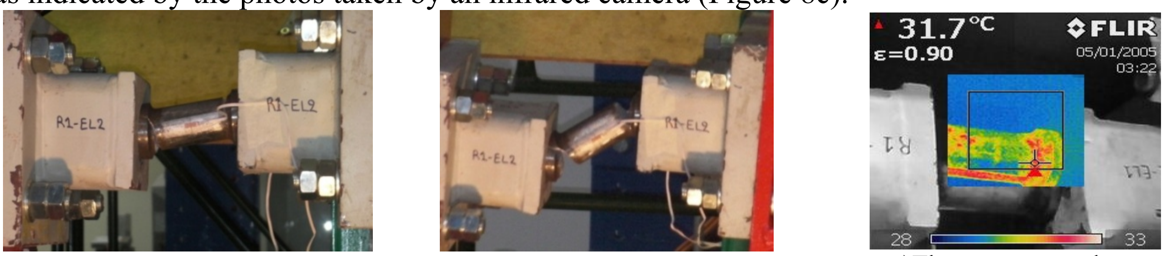


Figure 7: Frame with pin with varying lengths at its initial state and at the end of test

The pin specimen first acted as a beam in flexure, then the resisting mechanism changed to a tension field action and plastic hinges were generated under large deformations. This corresponds to the observed overall behaviors of both tests. At the beginning of the test and during several cycles the measured load was increasing. When the first crack formed at the ends of the weakened part of the pin (Figure 8a) the pins fractured (Figure 8b) and as a result the load dropped. This means that there was local stress concentration at the ends of the weakened pins as indicated by the photos taken by an infrared camera (Figure 8c).

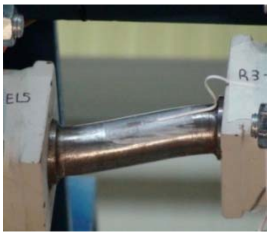


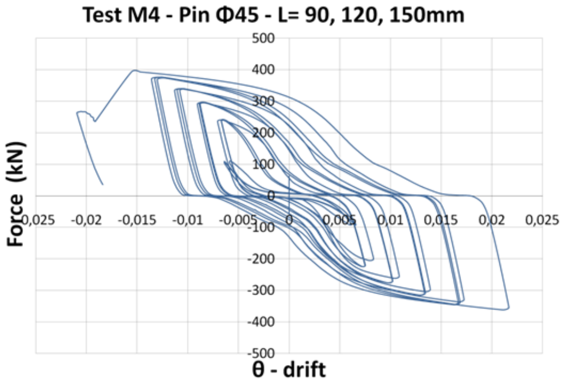
a)First crack b) Fracture c)Thermocamera photo

Figure 8: Specimen photos during the test

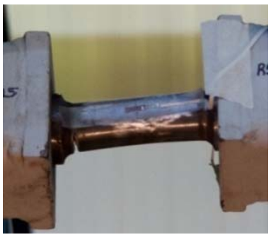
Figure 9 shows the hysteretic diagrams of the tests and photos of the deformed pins during the test. The hysteresis loops have a pinching due to the gap formed between the pin and the plate, as a result of the extensive plastic deformation of the pin and the Poisson effect along the pin circumference. This pinching of the hysteretic curve is accompanied by a substantial drop in the initial stiffness due to the release of the tension field developed in the previous load excursion. Generally, it should be noted that the shorter pins failed at smaller drifts than those of the longer.

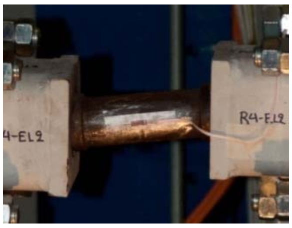

Short pin

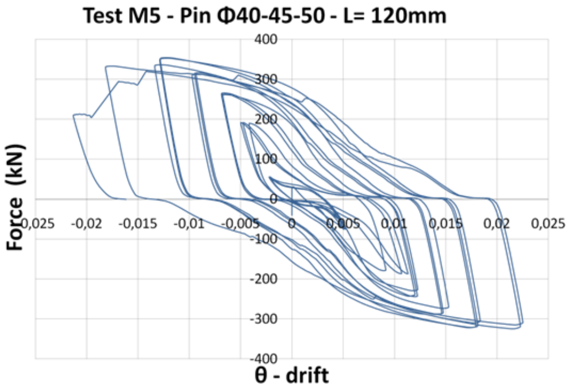

Long Pin



Cyclic Test M4 – Varying lengths


Thin pin


Thick Pin



Cyclic Test M5 – Varying diameters

Figure 9: Force–drift diagrams and photos of the deformed pins

3.3 Low cycle fatigue considerations

Preliminary analyses and experiments on pin fuses, showed that pins can sustain a limited number of cycles in the post-elastic region. The number of cycles to be sustained is dictated by low-cycle fatigue considerations. Low cycle fatigue may be captured by definition of appropriate S-N lines, where S is expressed in terms of deformations rather than in terms of stresses. Such S-N lines may be written as:

$$\log N = -m \log \Delta\varphi \tag{1}$$

where Δφ is the pin rotation, N is the number of rotation cycles and m is the slope constant of the fatigue strength curves.

Using the Palgrem – Miner accumulation law the damage produced by a certain number of constant amplitude cycles is expressed by:

$$D_i = \frac{n_i}{N_{fi}} \tag{2}$$

where: n_i is the number of cycles at specific rotations and N_{fi} is corresponding number of cycles to failure.

For cycles of various amplitudes failure occurs when

$$D = \frac{n_1}{N_{f1}} + \frac{n_2}{N_{f2}} + ... \frac{n_i}{N_{fi}} \geq 1 \tag{3}$$

The S-N lines were derived by evaluation of the cyclic tests on individual devices performed at RWTH and on complete frames performed at NTUA. The test results indicated that the most appropriate value for the slope is m = 3 as proposed in EC3 [10] for high cycle fatigue. The resulting S-N lines are shown in Figure 10. It may be seen that individual devices provide more conservative results. However, the frame results may be considered more realistic due to the combined action of multiple beams. Therefore, following validated relation is proposed to verify low-cycle fatigue:

$$\log \Delta\varphi = -\frac{1}{3}(\log N)\text{-}0{,}30 \tag{4}$$

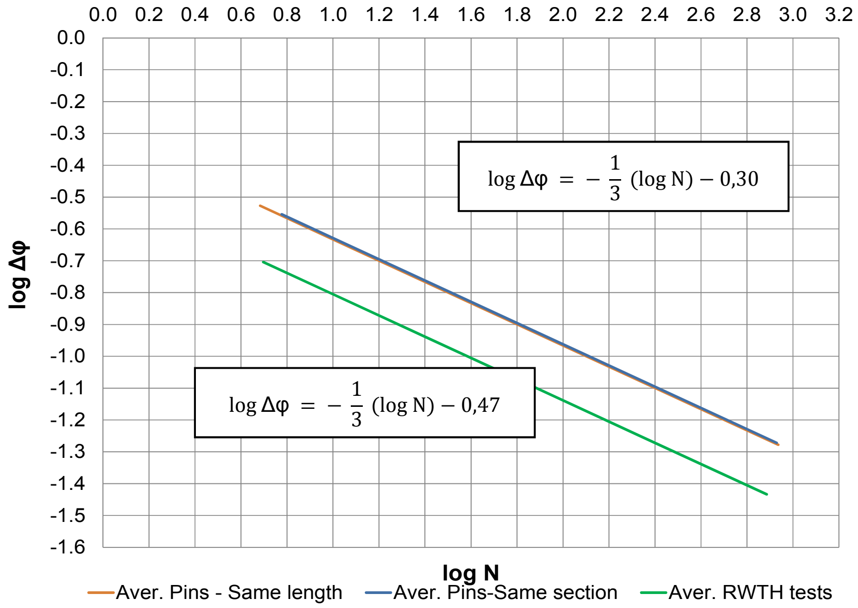


Figure 10: Low cycle fatigue diagrams

3.4 Development and calibration of simple models

The commercial software SAP2000 [11] 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 pin fuses was subdivided to 3 zones that represent the full section at the ends and the weakened section in the middle (Figure 11). In this manner, the true system flexibility and strength was accounted for. The columns and the receptacle beams were simulated as usual. Beam-to-column joints were represented as rigid in accordance to the connection detailing of the test.

Two different approaches were followed for the calibration of the tests and the evaluation of the non-linear lateral behaviour of the 2D test frames, a non-linear static (incremental pushover) and a non-linear dynamic (time history) analyses (Figure 12).

Regarding the incremental pushover analysis, the target displacement corresponds to the maximum displacement (170mm) of the experimental tests. Since the ductile elements are the

FUSEIS pins, potential plastic hinges were inserted at the ends of the weakened part of the pin. Initially, the relevant moment rotation curves (hinge property data) were based on the parameters obtained by the RWTH tests on the devices. These parameters were the starting point for the determination of the non-linear hinge properties of the FUSEIS1 devices. Five points labeled A, B, C, D, and E define the moment – rotation behavior of a plastic hinge [12].

In order to fit the analytical results to the experimental, several iterations were made changing the parameters of the hysteresis model. Table 1 summarizes the non-linear hinge parameters that comprised the most suitable approach.

The calibrated models predict with good accuracy the entire behavior of the frames with fuseis indicating that the proposed hinge properties can be applied to study the response of the fuses beyond the elastic state and to estimate expected plastic mechanisms and the distribution of damage.

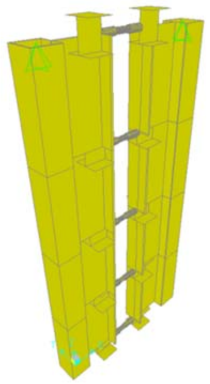


Figure 11: Sap model

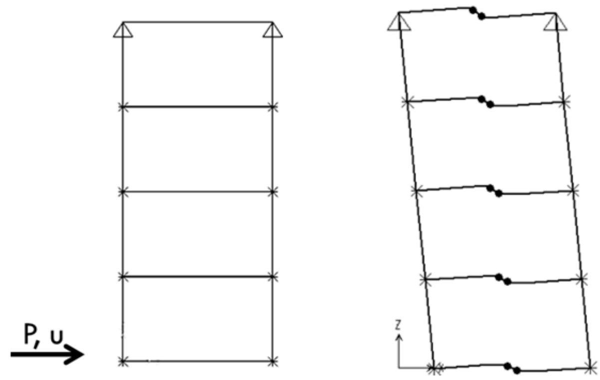
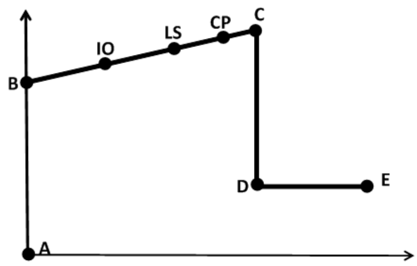


Figure 12: Non-linear static(P) & dynamic(u) loading and hinge formation

HINGE PROPERTIES		
Point	**M/SF**	**Rot./SF**
E-	-0,5	-150
D-	-0,5	-100
C-	-2,5	-100
B-	-2	0
A	0	0
B	2	0
C	2,5	100
D	0,5	100
E	0,5	150
ACCEPTANCE CRITERIA		
IO	30	
LS	45	
CP	60	



M–θ Relation

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

Table 1: Proposed Non-linear hinge parameters

4 LINEAR STATIC ANALYSIS AND DESIGN OF FRAMES WITH FUSEIS 1-2

In the current state of the art, a building frame with Fuseis 1-2 was analyzed with the use of the general purpose software Code SAP 2000. The examined frame was part of a five-storey composite building with 3,4m storey height and 5 system pins/storey with 2m axial distance of system columns. The geometry and the section properties are given in Figure 13. The systems were placed in the building plan at the outside bays of the frames in a similar manner to shear walls, coupled walls, etc. The beams were composite and the thickness of the slab was 15cm. The system columns were hollow strong columns, jointed together with five horizontal fuses in a tight arrangement. The effective width of the composite beams was calculated based on EC2. The general assumptions for the design of the building frame are given in Table 2.

The models were analyzed and dimensioned according to the provisions of EC3 and 8 and the relevant Design Guide. The Design Guide includes rules to ensure that yielding, takes place in the pins prior to any yielding or failure elsewhere.

The simulation concept for the pins was the same as described in Section 3.4, the reduced pin sections per storey for PGA=0.36g are also presented in Figure 13. The pins were connected to the system columns through receptacle beams SHS 240x240x20 forming a rigid joint to enable the Vierendeel action. The receptacle beams and the system columns were designed to have sufficient overstrength in order to achieve energy absorption only in the fuses.

The remaining structural elements were simulated as usual. The column bases were pinned to limit yielding at the foundation and thereby minimize damage to the columns. Rotational springs were assigned at the composite beams' ends to simulate the connection between the composite beams and the columns (EC 3 part 1.8, §6.3 and EC 4 part 1 - Annex A).

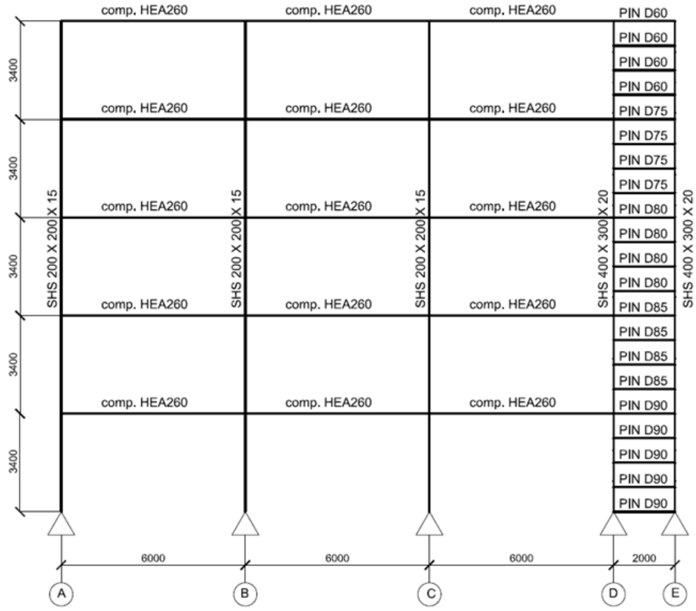


Figure 13: Typical 2D building frame

For the design of the frame the following conditions were fulfilled:

1. Serviceability Limit State: composite beam deflections
2. Ultimate Limit State: composite beam resistance ratio
3. Modal response spectrum analysis: The first mode of vibration activated approximately 75% of the mass
4. Seismic design- EC 8:
   - Limitation of interstorey drift to 0,0075 (buildings with ductile non-structural elements)
   - 2^{nd} order effects via linear buckling analysis, the interstorey drift sensitivity coefficient θ was lower than 0,1

- Design of FUSEIS pins for the most unfavourable seismic combination – Design Guide:
  a. Condition for bending moments
  b. Condition for axial forces
  c. Condition for shear forces
  d. Global dissipative behaviour of the structure: the maximum overstrength of the pins does not differ from the minimum value by more than 25%.
  e. Plastic Rotations Capability θ_p
- Capacity design of Fuseis columns and receptacle beams

Materials	
Concrete	C25/30,γ=25KN/m^3, E=31GPa
Reinforcement	B500C
Structural steel	S 235: Dissipative elements (FUSEIS) S 355: Non dissipative elements (beams - columns)
Vertical loads – calculated for 8m effective width	
Dead loads apart from self-weight –	2.00kN/m^2
Live loads – Q	2.00kN/m^2
Seismic loads	
Elastic response spectra	Type 1
Greek seismic zones	A=0.16g – 0.24g – 0.36g
Importance class	II
Ground type	B
Behaviour factor q (verified in Section 5 through nonlinear analysis)	3
Damping	5%
Factors of operating loads for seismic combinations	φ=1.00 (roof) φ=0.80 (storeys with correlated occupancies)

Table 2: Proposed Non-linear hinge parameters

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

Non-linear analyses were carried out to estimate the behavior factor of the system, the expected plastic mechanisms and the distribution of damage. The base of the analysis was the target displacement applied at the roof of the frame equal to 0,680m (inter-story drift 4%). The analysis was carried out under conditions of constant gravity loads and monotonically increasing lateral loads taking into account P –Delta effects.

Since the ductile elements are the FUSEIS pins, potential plastic hinges were inserted at the ends of their reduced parts. The nonlinear properties for the pin sections that derived from experimental and analytical investigations are given in Table 1. The rest of the structure - composite beams and columns - remained elastic. Figure 14 demonstrates the formation of plastic hinges within the reduced parts of the pins at the performance point.

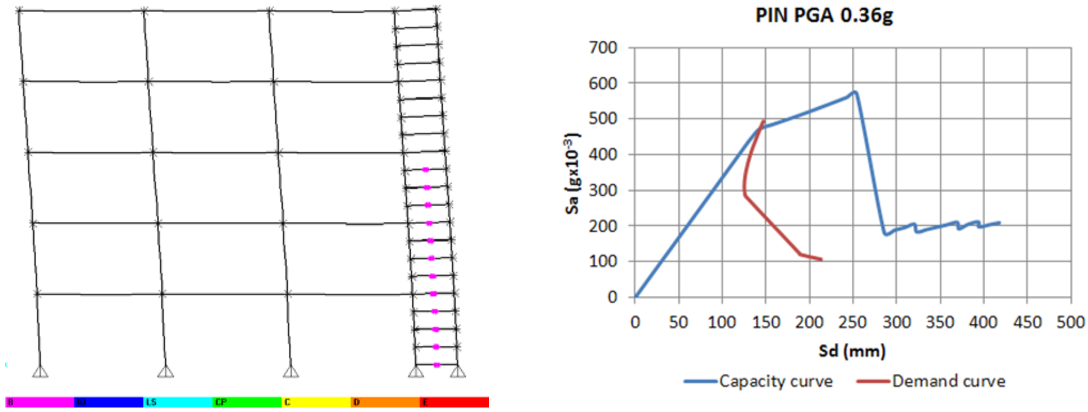


Figure 14: Hinges at the Performance Point and Evaluation of performance point

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 14: Hinges at the Performance Point and Evaluation of performance pointFigure 15 shows the typical pushover response curve for the evaluation of behaviour factor.

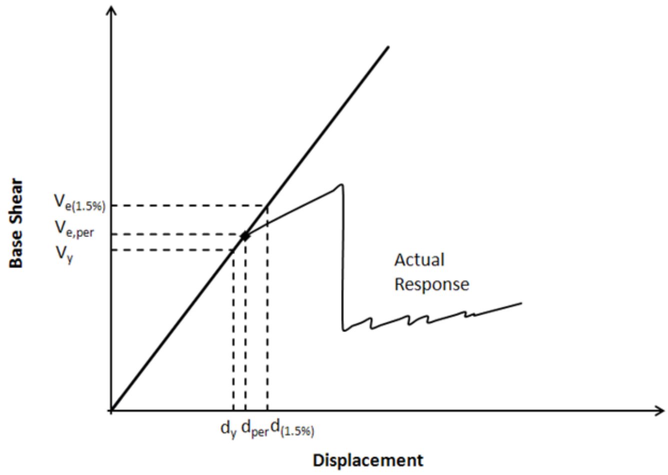


Figure 15: Typical pushover response curve for evaluation of behaviour factors

The behaviour factor, q, accounts for the inherent ductility and overstrength of a structure and may be generally expressed in the following form:

$$q = q_\mu \cdot \Omega \tag{5}$$

The structure ductility, q_μ, is defined in terms of the Elastic Base Reaction that corresponds to 1,5% drift (V_e) to the Idealised Yield Strength - First hinge (V_y) (Figure 15), as following:

$$q_\mu = \frac{V_e}{V_y} \tag{6}$$

The interstorey-drift (ID) adopted, 1,5%, was derived from the experimental results. At approximate this drift the maximum applied force was reached in the tests. This drift corresponds to plastic rotations of the pins of about 20%.

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:

$$\Omega = \frac{V_y}{V_d} \tag{7}$$

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 are given in Table 3 and Figure 16. The system exhibits high over-strength values and relatively low ductility, the q values range between 3 and 6. Conservatively, a maximum value equal to 3 is proposed for the design of buildings with FUSEIS 1-2 system.

0,16g			0,24g			0,36g		
q_μ	Ω	q	q_μ	Ω	q	q_μ	Ω	q
1.74	3.26	5.67	1.38	2.92	4.05	1.28	2.73	3.48

Table 3: Non-linear hinge parameters

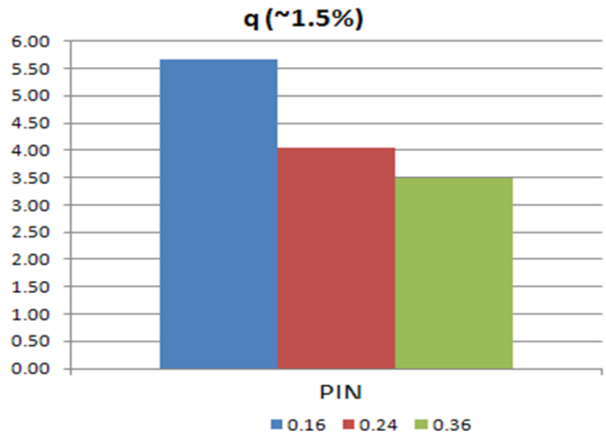


Figure 16: Evaluation of q factors

6 CONCLUSIONS

Innovative fuses systems may be applied to multi-storey steel buildings as an alternative to conventional seismic resistant systems. They combine ductility and architectural transparency with stiffness. Additionally, they offer the possibility of easy mounting and dismounting within the structure. The knowledge obtained is now sufficient to provide all the necessary data for their complete design.

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

The dissipative elements can be positioned in small areas of the building and do not interrupt the architectural plan as braced systems do. They can also be visible parts of the building indicating its seismic resistant system.

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

The investigations on FUSEIS 1-2 provided a good estimation of the behaviour factor of the system which is approximately 3.

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 after test is easy from a practical point of view: the time required for replacing one FUSEIS1 device is approximately 60 minutes.

The damage index of a building with pin fuses may be determined through the low cycle fatigue curve proposed.

The application of FUSEIS devices can provide a more accurate and less expensive design of a building. The steel quality of the dissipative elements can be controlled and thus their resistance can be calibrated avoiding excessive overstrength. The knowledge obtained is now sufficient to provide all the necessary data for their complete design.

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