Numerical and Experimental Results of Project FUSEIS (Seismic Resistant Composite Steel Frames)

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SUMMARY:
The project Fuseis carried out with the financial grant of the Research Fund for Coal and Steel of the European Union (RFCS-CT-2008-00032), aims at developing two innovative types of seismic resistant steel frames with dissipative fuses. In case of strong earthquakes damage will concentrate only in the fuses, which will be easily and inexpensively replaced. This paper presents the results of experimental and numerical investigations on the behaviour of overall frames with one of the two types of fuses under cyclic and earthquake loading.

Keywords: Energy Dissipative Devices, Steel Structures, Seismic Design

1. INTRODUCTION

Experience shows that earthquakes lead frequently to damages in large extent with consequent very high repair costs (Engelhardt and Sabol, 1997). After recent seismic events (mainly Northridge 1994 and Kobe 1995), some new solutions for moment resisting frame connections have been implemented introducing weakened areas near the beam ends where plasticity is concentrated. An example to these solutions is the Reduced Beam Section (RBS), introduced by Plumier (1990). Experiments carried out by Pachoumis et al. (2010) and by Yu and Uang (2001) showed that with this solution it is possible to successfully dissipate energy and concentrate plasticity, however simple and cost effective reparability of the damaged parts of the structure still remained as a problem. It is therefore advisable to develop structural systems that are simple to be repaired, i.e. to introduce the reparability as a new property.
Furthermore, in reality "steel structures" rarely exist by themselves (eventually just in the case of industrial buildings). Most often, in the case of high-rise buildings, housing, as well as commercial buildings, the steel beams support reinforced concrete slabs and usually behave as composite members. In this case, damage to the steel members results in damage in the reinforced concrete slabs and in the finishes, so that repair works will be increased together with the related costs.
Other engineering disciplines (e.g. mechanical, electrical and automobile or aircraft engineering) show that the best way of repairment is the complete replacement of a damaged part. Such a strategy could be also envisaged in civil engineering, especially for buildings in seismic areas that are susceptible to damage as described above. Like bumpers in cars that absorb the crash energy and are replaced afterwards, innovative devices will be developed that dissipate energy, protect the overall structure and may be disassembled and replaced after a strong earthquake.
Significant progress has been achieved in the research of dissipative and easy-replaceable fuses for braced steel frame structures. Chan et al. (2009), Li and Li (2007) and Bruneau et al. (2010) carried

out experiments on eccentrically braced frame systems with different link geometries. Plumier et al. (2004) developed innovative dissipative systems for concentric braced frames, comprising pinned and "U" type connections. Oh et al. (2009) developed an innovative fuse based on a damper system assembled at the bottom flange of the beam at the beam-to-column connection for moment resisting steel frames.
The research project FUSEIS (carried out with the financial grant of the Research Fund for Coal and Steel of the European Commission , grant agreement no: RFSR-CT-2008-00032) focuses on MR Steel frames. Two types of devices are studied, one applying to shear walls (presented elsewhere), the other to beam-to-column connections (presented in this paper).

2. FULL SCALE TESTS

Within the Fuseis project, for each type of device, two types of tests are performed:

a) Component tests for the characterization of the device behaviour,
b) Tests on a full scale frame

Component tests are implemented in the structural engineering laboratory of Istituto Superior Tecnico University in Lisbon, to characterize the performance of the fuse devices in terms of moment and rotation capacities (Calado et. al. 2012)
The test specimen consists of a beam-to-column connection, with a concrete slab and of the fuse devices as schematically shown in figure 1.

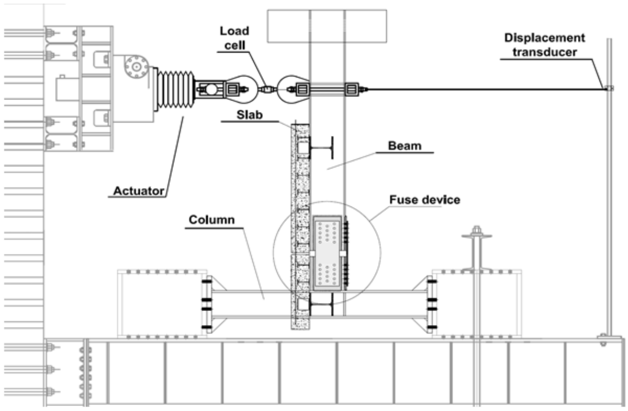

Figure 1 Scheme of the bolted fuse device

In order to simulate a more realistic case and evaluate the response of the fuses and the global behavior of the structure, a full scale composite steel frame with fuse devices is tested in the structural engineering laboratory of Politecnico di Milano. Four symmetric cyclic full scale tests as well as four seismic cyclic tests were carried out in order to evaluate the performance of a composite steel frame with fuse devices in terms of moment rotation behavior of the joints, global energy dissipation, storey drift and frame stability. Hereafter the results of the full scale tests are presented in detail.

2.1. Test Set up

The test set up shown in figure 2 represents a two dimensional portion of a storey of a composite steel multistory building. The fuse devices are named as fuse 1, fuse 2 and fuse 3 respectively.
All the steel parts are provided by the Greek steel production company Sidenor S.A, industrial partner of the FUSEIS project, and assembled in the structural engineering laboratory of Politecnico di Milano. The frame in S275 steel consists of HEB240 columns, IPE300 beams, and a 150 mm thick

C30/37 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.

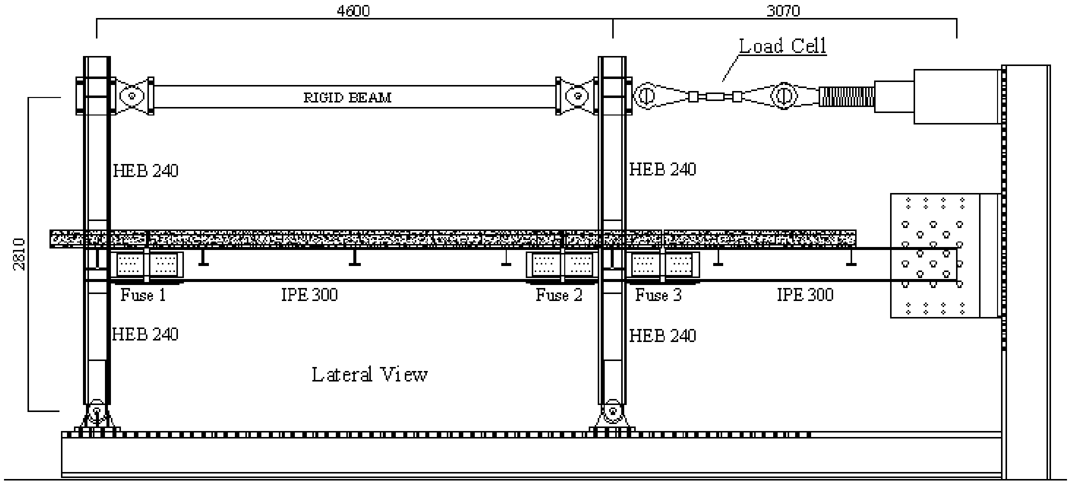

Figure 2 Test Equipment

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. The design of the composite slab is made according to Eurocode 4.

The bases of the columns 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 no 3 is restrained only against the vertical displacement, but free to slide in horizontal direction (figure 2).

The horizontal constraint between two top joints of the frame is provided with a rigid beam which is connected to the column top joints with pins (figure 2).

The out of plane stability of the frame is achieved with transversal elements providing a pinned joint free to slide longitudinally on the reaction frame of the laboratory (figure 3).

Figure 3 Transversal elements supporting the structure in out of plane direction

The fuse devices are obtained by means of steel plates connected 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. To connect the steel plates to the beams, high strength friction grip (HSFG) bolts are used. The bolts are tightened according to provisions given in UNI EN 14399-2:2005. The part of the beam near the connection is reinforced with steel plates welded to the web and to the flanges. 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 4 and 5.

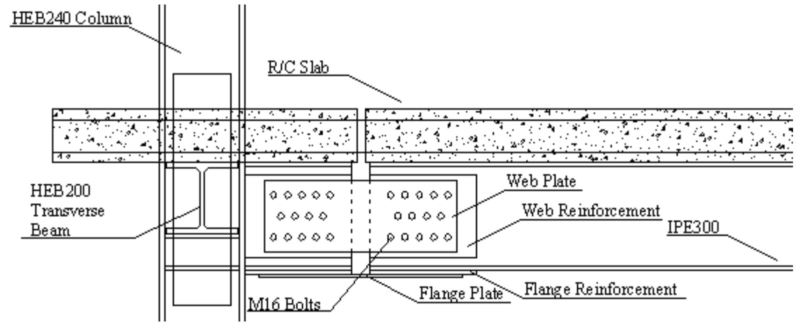

Figure 4 Exterior Beam-Column Joint Detail

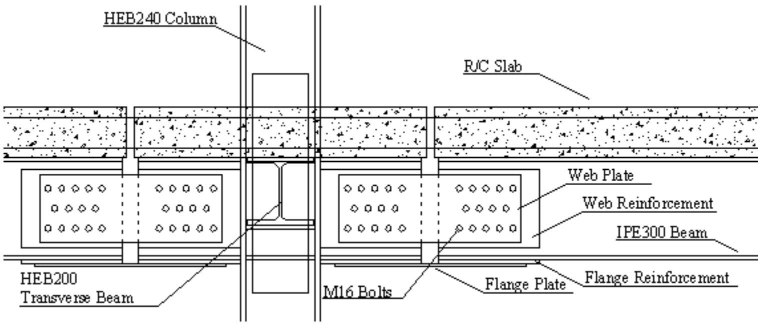

Figure 5 Interior Beam-Column Joint Detail

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, though 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.

The slab reinforcement is designed according to the provisions given in Eurocode 8 (Annex J), which provides rules for the possible mechanisms that describe the force transfer mechanism between the concrete slab and steel column for positive and negative bending. Longitudinal reinforcement consists of Ø20/100 bars on the upper level, and Ø16/200 + Ø12/200 bars on the lower level. The transverse reinforcement consists of Ø12/72 bars near the fuse section and Ø10/72 bars in the rest of the slab. Moreover, additional steel reinforcement is positioned in the area of the “gap” in order to guarantee elastic behavior of the steel beam as well as that center of rotation of the device remains between the two steel layers. This additional top and bottom steel reinforcement also ensures the diaphragm action to take place in real structures. Length of the additional reinforcement bars is such that adhesive bond with concrete is fully developed.

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.

In addition to the seismic design, also a serviceability limit state design is performed to define the dimension of the structural elements and the amount of the additional reinforcement in the gap section. Since the floor acts as a composite section made by an IPE300 and a concrete slab, under gravity loading the serviceability levels are satisfied without difficulty.

2.2 Specimens and loading histories

Four symmetric cyclic tests are implemented on the steel-composite frame with four different fuses having the same web plates but with several flange plate thicknesses. The geometrical properties of the flange and web plates used for each type of fuse are shown in figure 6. The variable parameters (flange plate height and thickness) are shown in table 1.

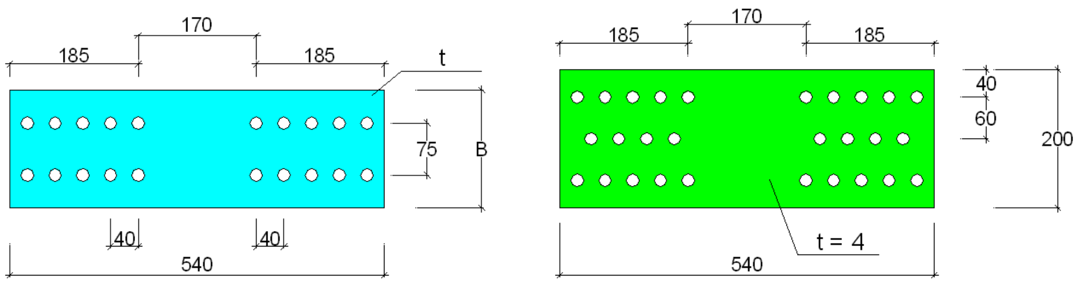

Figure 6 Geometrical Properties of Flange plate (on the left), Web plate (on the right)

Specimen	B (mm)	t (mm)	Area of cross section (mm^2)
Flange Plate A	120	10	1200
Flange Plate B	170	10	1700
Flange Plate C	150	12	1800
Flange Plate D	140	8	1120

Table 1 Changing Parameters of the flange plates used in the fuse devices

Each test is performed until complete failure of the fuse flange plate, whichever fails first. After each test, the fuse plates are replaced by new ones and the next test is carried out. The fuses are tested in the following order: D, A, B, and C, for minimizing possible damage effects in the test frame specimen.
To force the position of the plastic hinge to remain within the fuse and to avoid that damage spreads to the non-dissipative zones, fuse elements have to be designed to be weaker than the adjacent members. In order to parameterize this quantity, a testing parameter α was introduced, which relates the resistance capacity of the fuse with the plastic resistance of the cross-section of the composite beam. This value is defined as the capacity ratio of the fuse device and is given by Eqn.1.

$$\alpha = M_{max,fuse} / M_{pl,beam} \tag{1}$$

where $M_{max,fuse}$ is the maximum moment developed by the fuse device and $M_{pl,beam}$ is the plastic resistant moment of the non-reinforced area of the composite cross-section of the beam. The capacity ratios of the fuse devices are listed in table 2. The positive and negative values of α refer to the capacity ratios of the fuse device under sagging and hogging bending respectively.

Table 2 Capacity factors of the fuse devices

Specimen	α^+	α^-
A	0.53	0.36
B	0.65	0.41
C	0.67	0.46
D	0.52	0.33

The frame is subjected to cyclic (quasi static) loading by means of an actuator with a maximum loading capacity of 1000 kN, and a constant loading velocity of 21 mm/min. The cyclic loading protocol used in the tests that has been set according to the provisions given in ECCS is summarized in table 3.
The tests are considered satisfactory when a drift causing at least a 35 mrad rotation in the fuse devices can be obtained without significant inelastic deformation on the structural elements and a significant damage on the reinforced concrete slab. The global displacement at the fuse level is

measured with a displacement transducer attached to the right end support of the structure. To measure the displacements and rotations throughout the structure, a total number of 42 displacement transducers are used.

Table 3 Cyclic Loading Protocol

Load Step	Peak displacement at the fuse level	Number of cycles
1	±2.5 mm	1
2	±5 mm	1
3	±7.5 mm	1
4	±10 mm	1
5	±15 mm	3
6	±20 mm	3
7	±25 mm	3
8	±30 mm	3
9	±35 mm	3
10	±40 mm	3
11	±45 mm	3
12	±50 mm	2
13	±55 mm	2

The load is applied in symmetric cycles in -x and +x direction. Therefore in each cycle, in the beam and eventually in the fuse element, a positive and negative bending are observed. During the tests, a maximum rotation of 42 mrad in the fuse element was observed. In addition to these four symmetric cyclic tests, other four cyclic tests were carried out with "seismic" loading histories as described in section 5.

3. SYMMETRIC CYCLIC TEST RESULTS

The overall cyclic behaviour of the structure is characterized by plastic deformations that take place in the flange and web plates in the fuse devices. Measurements of relative rotations and displacements in the vicinity of beam-to-column connection showed that the columns and beams remained elastic with no evidence of plastic deformation or local buckling. The beam-to-column connections, which have larger moment capacity than the fuse parts, remained almost perfectly rigid. The deformations in the steel reinforcement did not go beyond the elastic range, as expected. The maximum relative displacement between the slab and the beam was 0.5 mm, which means the composite action between the reinforced concrete slab and the steel beam has been satisfactorily achieved. Both rotations and moments are computed at the mid-section of the fuse.

The maximum rotation observed in the fuse devices is 42 mrad and after all the tests implemented, there was not any significant damage in the concrete slab. Fuse types A, B and C were capable of developing the minimum ductility requirements of Eurocode 8, achieving a rotation capacity of at least 35 mrad with a strength degradation less than 20%. Fuse type D, which is composed of the thinnest plates, had even larger rotations, however its strength degradation values were above the 20% limit that is stated in Eurocode 8. Therefore it can be pointed out that the thickness of the plates in the fuse device D were too small to achieve the desired ductility, while the other devices gave satisfactory results.

The overall joint behavior is characterized by the inelastic deformation taking place in the fuse devices. In the fuses, two types of deformation are observed. Every cycle includes positive (sagging) and negative (hogging) bending moments in the fuse devices (figure 8). During sagging, both flange and web plates stayed under tension, whereas under hogging both plates stayed under compression and they buckled. The composite beam and the columns, the moment capacity of which was greater than that of the fuse, remain elastic with no evidence of plastic deformations or local buckling of the flanges.

In general, the results showed that fuses with higher values of capacity ratios (α) provide higher performance levels in terms of stiffness, resistance, ductility and dissipated energy. Nevertheless, fuses with values of α close to unity and, therefore, whose strength is similar to that of the composite beam, induce more damage and thus fail to concentrate plasticity within the fuse section. This behaviour contradicts one of the underlying concepts of the fuses, and, therefore, the value of α should be limited by an upper bound, in order to prevent that plasticity spreads into the irreplaceable parts.

Figure 7 Hogging (on the left) and sagging (on the right) behaviour of the fuse device

Although in full scale tests only α values in the range $0.52 \leq \alpha^+ \leq 0.67$ and $0.33 \leq \alpha^- \leq 0.46$ were considered, in the component tests α values in the range $0.45 \leq \alpha^+ \leq 0.71$ and $0.27 \leq \alpha^- \leq 0.50$ were studied. Furthermore a parametric study based on numerical analyses was carried out in order to define the optimal values of this parameter. Based on the achieved results, it can be suggested that, in order to obtain the best performance of the fuse device in terms of capacity and energy dissipation, α values should be assumed in the range:

$$0.60 \leq \alpha^+ \leq 0.75$$
$$0.40 \leq \alpha^- \leq 0.50$$

One of the main advantages of the “FUSEIS” design approach is the reparability. After the seismic event, the damage is concentrated in the fuses, the rest of the structure remaining elastic, without having any plastic deformation. During the experimental activities, it is seen that the damaged fuse plates can be easily replaced. After each test, the replacement of three fuse devices is accomplished by two workers in about 1.5 hour.

4. NUMERICAL ANALYSIS

For the design of the test specimens, a refined numerical analysis was carried out at Politecnico di Milano, using Abaqus software. Such a detailed model, however, can be used only for specific research applications, but is not feasible to be used for engineering purposes. Therefore a simplified nonlinear model is developed using the commercial software package Sap2000.

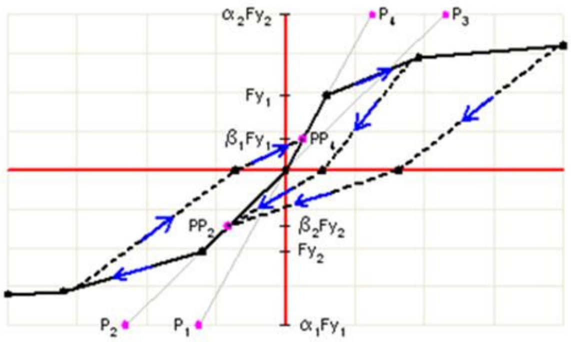

Figure 9: Multilinear Plastic Pivot Link Behaviour

In order to obtain the non-linear response of the fuses, they are modeled as multi-linear plastic link elements with a length equal to the plastic hinge length, which may be assumed to be the same as the free length of the fuse. As shown in figure 9, the hysteresis type should be the one provided by the

Pivot model. This behaviour is defined only for the rotational degree of freedom of the link with respect to the major axis of inertia. The remaining degrees of freedom are modeled as linear.
The input of the monotonic moment-rotation diagram was calibrated on the component test results, by means of a fiber model with the stress-strain relationships defined for the materials, using the software package Perform 3D.

4.1 Analysis Results

As expected, the inelastic deformations only occur in the fuse elements of the frame while the composite beam and column elements deform within their elastic range. This can be understood examining the moment rotation diagrams of inelastic composite beam and fuse elements shown in figure 10: The fuse element deforms beyond its yield limit and contributes to the energy dissipation in the frame with a maximum rotation of 45 mrad.

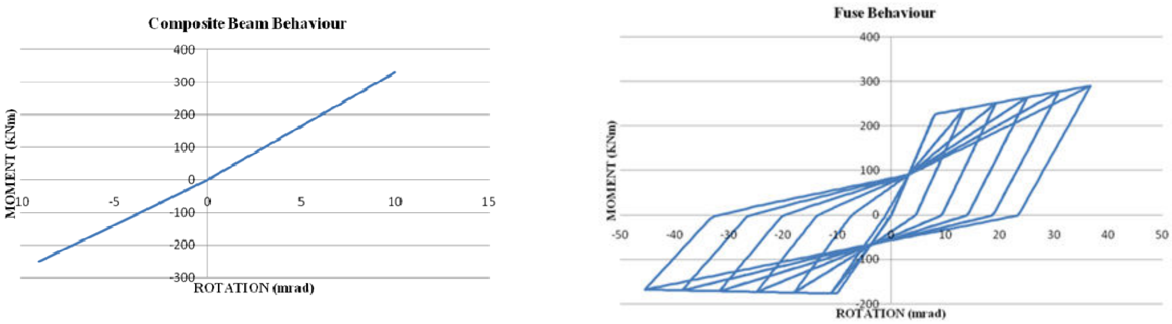

Figure 10: Moment-Rotation diagrams of the composite beam (on the left) and the fuse device (on the right) under horizontal cyclic loading

The conformity of numerical and experimental results in terms of maximum plastic moment capacity and the maximum rotation that the device undergo can be observed in figure 11 that shows a comparison of the numerical versus experimental response of the test frame. The differences in the initial stiffness and amplitude of the hysteresis plots between experimental and numerical results are due to inelastic effects (mainly slippage) occurring in the connections between the fuse device and the beams, that were not accounted for in the numerical model.

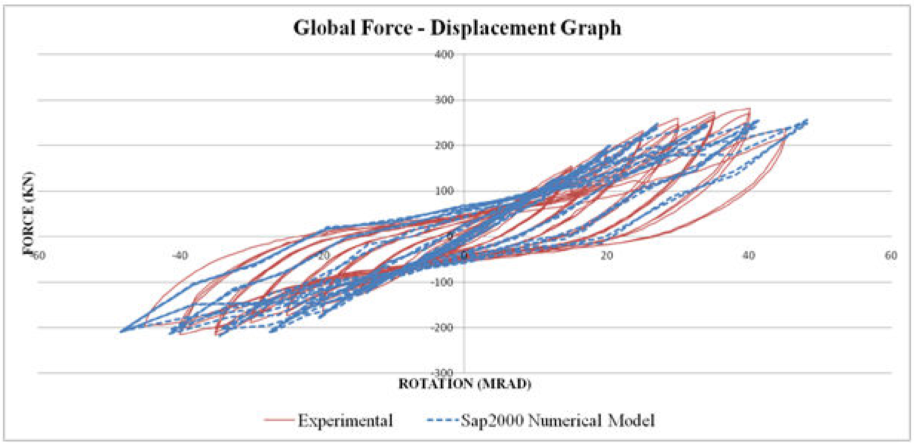

Figure 11: Calibration of the numerical model

5. SEISMIC CYCLIC TESTS

Another set of four tests was carried out to observe the dissipative behaviour of fuse devices under seismic displacements. In order to simulate seismic displacements on the test frame, a numerical model of a 5 storey building is developed using the calibrated parameters, and nonlinear time history analyses are carried out under three horizontal earthquake excitations (Kobe 1995, Chile 2011, and New Zealand 2011). The maximum inter-storey displacements obtained as a result of the nonlinear time history analysis are experimentally applied to the top joint of the column of the test frame with fuse devices (in the same loading configuration as for symmetric cyclic tests). As a result, force displacement diagrams are obtained from each seismic cyclic test carried out. From figure 12 which shows the global force-displacement diagram of the seismic cyclic tests using fuse type A, it can be seen that also under inter-storey displacements which are the numerical representatives of real seismic events, the fuse devices show a very good performance in terms of dissipation capacity and ductility.

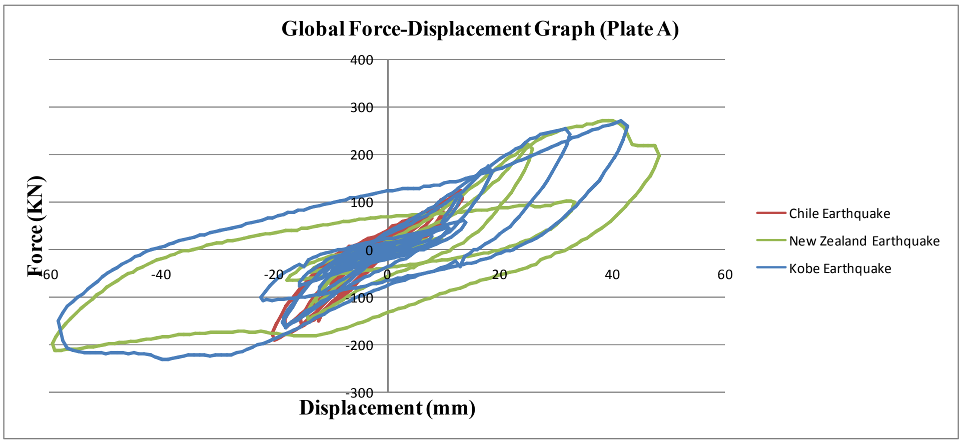

Figure 12

5. CONCLUSIONS

In this study, the results of the experimental analysis carried out on one of the two types of innovative fuse devices that were studied within the FUSEIS research project, namely the one that can be applied to beam-to-column connections of moment resisting frames are presented. The ability of the fuse devices to dissipate energy and reduce the horizontal earthquake forces in steel frames is investigated. The possible damage that the main structural elements of a moment resisting steel frame would suffer during a strong earthquake is aimed to be concentrated in the fuse devices. While in the conventional moment resisting steel frames, the beams and their connections –the elements that resist gravity loading and are difficult to repair- must be repaired after a strong earthquake, in the innovative type seismic resistant steel frames with dissipative fuses, the repair work, if needed, will be limited only to the replacement of the fuses.

The structures with bolted fuse devices during the four full scale cyclic tests and the subsequent four seismic tests carried out at Politecnico di Milano showed very good performance in terms of ductility, stiffness, energy dissipation and resistance. Thanks to the concentration of the inelastic behaviour only in the fuse devices, the irreplaceable parts (beams, columns and concrete slab) did not suffer any significant damage, and remained elastic as intended.

Buckling of the fuse plates under hogging bending proved to govern the hysteretic behaviour of the fuse devices. Besides the fuses showed a stable hysteretic behaviour, buckling induced a loss of strength under hogging bending, which did not allow the plates to explore their full plastic capacities.

Moreover, the tests proved that repair work can be achieved very easily and efficiently, thanks to the concentration of the damage only in the fuse devices.

Experiments have shown that the capacity ratio α is one of the most influencing design parameters, determining to what extent non-linear behaviour is demanded from the fuse (meeting the exchangeability requirement), also controlling the non-linear behaviour exhibited by the assembly. In this way, fuses with higher capacity ratio values also present higher bending resistance and higher energy dissipation capacities. However, the capacity ratio should be upper bounded, since fuses with high values of α lead to increased deterioration of the irreplaceable parts, which is undesired.

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PERFORM 3D Nonlinear Analysis and Performance Assessment for 3D Structures