The use of ductile steel fuses for the seismic protection of acceleration sensitive non-structural components: Numerical and Experimental verification

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1. ABSTRACT

Recent seismic events have showcased the vulnerability of non-structural components to even low- or moderate-intensity earthquakes that occur far more frequently than design-basis ones. Thus, community-critical buildings, such as hospitals, telecommunication facilities, or fire stations, often face lengthy functionality disruptions despite having suffered little structural damage during an earthquake. This paper summarises the numerical, and corroborating experimental, studies that were undertaken as part of the NSFUSE project at the University of Bristol's shake-table facility. The primary focus was to investigate the concept validity of using ductile steel fuses for protecting acceleration-sensitive non-structural components in the aftermath of earthquakes. The objective was to offer a reliable and inexpensive solution, via replaceable sacrificial elements, for the protection of such components. The experimental program involved a series of planar shake table experiments. These were conducted using narrow-band floor acceleration input signals that were recorded in instrumented buildings through the California Strong Motion Instrumentation Program during three different earthquake events. By changing the mass of the carriage-like test specimen, as well as the fuse height and its cross section, different component-to-building period ratios (tuned and slightly detuned cases) along with yield strength levels were investigated. For each test, the input signals were incrementally scaled, if needed, to induce

different ductility demands. The tests provided insight into the seismic performance of non-structural components that are mounted on a structure and the benefits of allowing controlled yielding to occur in the attachments of non-structural components that are tuned or nearly tuned to one of the primary modal periods of the supporting structure.

2. INTRODUCTION

Past earthquake events (e.g., 1994 Northridge earthquake) have highlighted that non-structural components could be particularly vulnerable to seismic-induced damages even in developed countries, in which the buildings are designed according to modern seismic code provisions. Notably, non-structural damages have been observed not only under strong earthquake shakings, with intensities close to the design ones, but also in the aftermath of moderate or even low seismic events that are likely to affect more than once the primary structural system during its lifetime. Of much interest to seismic resilience, are those damages that occur to the non-structural components of the so-called community critical facilities (e.g., hospitals, fire stations) since, as per the societal demand, the latter should not only maintain their structural integrity but also remain functional following earthquakes that could impose structural and non-structural damages to other structures of ordinary importance.

To this end, lately, the engineering community has shifted its attention on the development of robust methodologies for the evaluation of the acceleration demands that are imposed to the non-structural components−located at different floor levels along the height of the supporting building−during an earthquake. Relatively recent, Kazantzi *et al* [1], on the basis of a numerical study that involved floor motions that were recorded during earthquakes on instrumented buildings in the United States (US), have showcased two important attributes with reference to the performance of the non-structural components, these being:

(a) The acceleration demands imposed to the non-structural components could be significantly amplified if the component has its fundamental period at or close to the supporting building' predominant modal period (fundamental or any other higher mode),

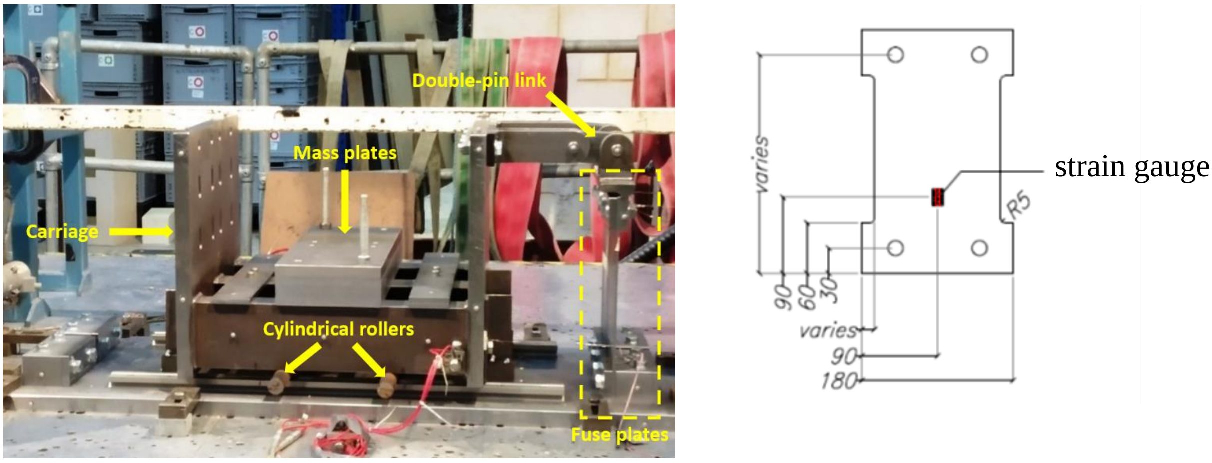
(b) Allowing for inelasticity to take place either in the support or in the bracing of the non-structural component could reduce the peak acceleration demands on the component.

Allowing for inelasticity to reduce seismic demands, is a well-known concept in earthquake engineering that is reflected in modern seismic codes in the capacity design approach. The extension of such concept to non-structural components was initially introduced by Miranda *et al* [2] and further expanded in Kazantzi *et al* [3]. Interestingly, the latest revised version of Eurocode 8-Part 4 [4], that is currently under public enquiry, offers three different design methods for verifying the seismic integrity of non-structural industrial equipment, of which one exploits the dissipative design concept. Practically speaking, this concept can be materialised by inserting, in the interface of the non-structural component and the supporting structural element of the primary structure (usually the slab) steel fuse-like parts in the element's anchorage system. Those fuses should be engineered so as at certain levels of seismic intensity to yield in a ductile manner. A detailed comparison of the three alternative non-structural element design methodologies of Eurocode 8-Part 4 [4] is offered in Kazantzi *et al* [5].

This paper summarises the findings of the NSFUSE experimental study that was undertaken at the shake-table facility of the University of Bristol during the SERA Project, to investigate the conceptual validity of using ductile steel fuses for protecting different kind of acceleration-sensitive non-structural components during earthquakes that are not equipped with isolators or sustaining seismic forces via sliding or rocking. The fuse concept is of particular interest to those acceleration-sensitive non-structural components in order to remain functional and transmit lower forces to the supporting structure during an earthquake event. The overall objective of this research study was to demonstrate that, by inserting sacrificial steel fuses in the non-structural anchorage system one can attain an efficient mechanism for the protection of critical non-structural elements.

3. TEST SPECIMEN

The test specimen featured a carriage-like configuration, as illustrated in Fig. 1a. The carriage, that was supposed to simulate a non-structural component, is essentially a Single-Degree-of-Freedom (SDF) system. The carriage was allowed to move on two rollers. Two steel plates that acted as fuses were attached at one end of the carriage (see Fig. 1b). The fuses essentially act as cantilevers to provide resistance to the sliding of the carriage. The lower fixed end support per each fuse was materialised via one clamp plate (15mm thick) and a rigid block, that was mounted on the shake table with two M16 bolts. At the upper end of each fuse plate, two clamp plates (15mm thick) were employed to attach them via fillet welds to a plate assembly that was connected, by means of another plate equipped with two ball joints for nesting two ø20 pins per fuse, to the carriage (see Fig. 1a). This configuration allows for a nearly unrestrained rotation at the top end of the fuses. As such, the flexural stiffness of the fuses is reduced (and hence the stiffness of the overall configuration) by a factor of about 4, thereby allowing for a wide range of periods to be captured with relatively low masses and low fuse plate heights. The instrumentation layout is illustrated in Fig. 2, and includes a combination of accelerometers, string potentiometers as well as a wireless displacement tracking system.



(a) (b)

Fig. 1 (a) Side view of the actual carriage test specimen configuration loaded with steel plates and (b) generic fuse plate configuration

Having set the test specimen for targeting different vibration periods, the carriage was loaded with different masses. The period of the test specimen was also further adjusted by moderately modifying the fuse heights (spanning between 80mm up to 260mm), the thicknesses (ranging from 5 to 10mm) and the cross sections (i.e., rectangular or bog-bone). These modifications allowed us to consider different component/building period ratios (tuned and slightly detuned cases) along with yield strength levels.

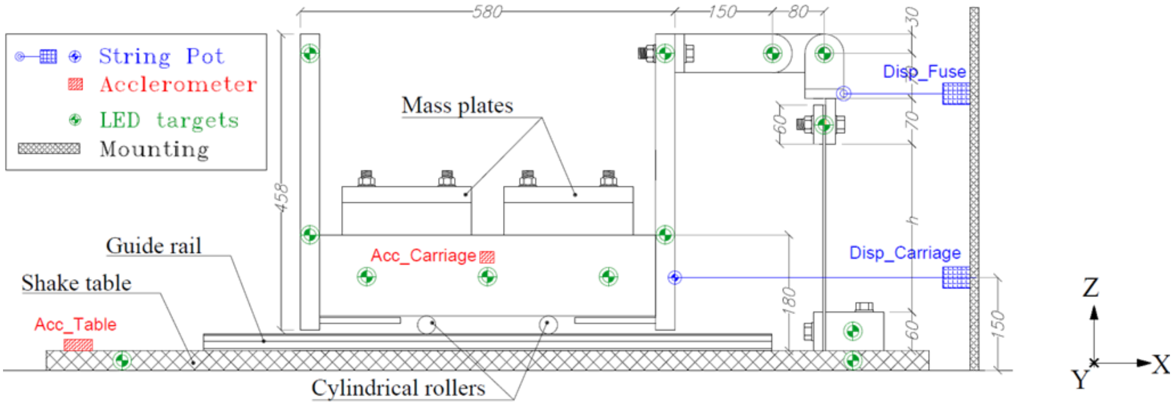


Fig. 2 Instrumentation layout

4. ANALYTICAL PREDICTIONS AND NUMERICAL MODELS

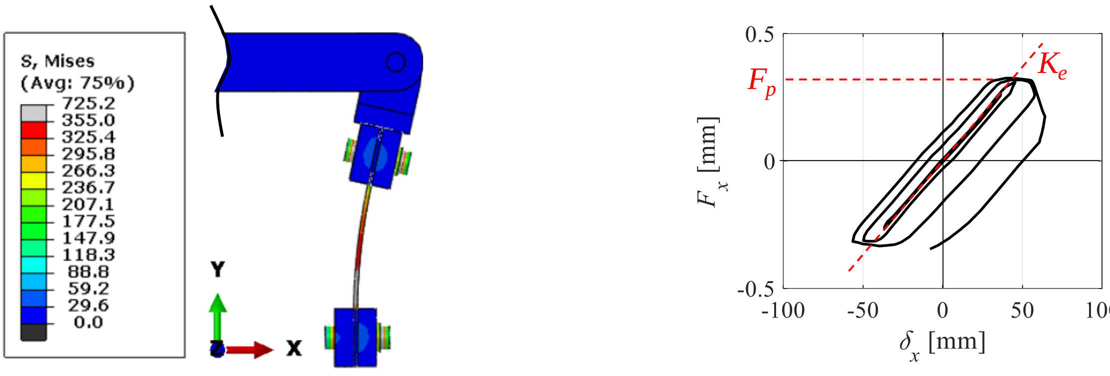
A representative finite element numerical model of the sliding platform test setup was developed in the commercial software ABAQUS/CAE [6]. The objective of this model was to provide pre-test predictions of the stiffness and strength of the cantilever steel fuse plates under cyclic loading. The full model was meshed using 8-node linear hexahedral elements with reduced integration (C3D8R). All the model plate parts were assigned a nonlinear material model with combined kinematic/isotropic cyclic hardening; representative of S355J2 steel grade. The adopted parameters for the material model were based on the values recommended by de Castro e Sousa et al. [7]. The bolts were assigned a similar model with parameters representative of high strength steel grade SHV 10.9. Rigid parts were assigned an elastic material model.

Fig. 3a illustrates the typical deformation profile and the von Mises stress contours of a ductile steel fuse. Fig. 3b shows the expected nonlinear response of the fuse (lateral force versus lateral displacement) under symmetric cyclic loading. The simulated responses were then used to deduce analytical expressions for the plastic strength (F_p) and the elastic lateral stiffness (K_e), as demonstrated in Fig. 3b. In particular, the following expressions were developed:

$$F_p = Z \cdot f_y / d_{\text{eff}} \tag{1}$$

$$K_e = 3E / \sum_{i=1}^{i=4} \frac{L_i^3}{I_i} \tag{2}$$

where, Z is the plastic section modulus of the fuse plate with respect to its weak axis, d_{eff} is the effective depth of the cantilever fuse plate (taken as h+160mm, where h is the clear height of the fuse plate), E is the Young's modulus of the steel, I_i and L_i are the cross-sectional moment of inertia with respect to the weak axis of the plate and length of region i, respectively. Note that, as per Eq. (2), the elastic stiffness is deduced by breaking down the deformed cantilever into four regions to compute the equivalent L^3/I term.



(a) (b)

Fig. 3 Simulated response of the nonlinear steel fuse plate [contour stresses in MPa]

5. TEST SETUP

The earthquake simulator tests featured one-dimensional shaking. For this purpose, narrow-band floor acceleration input signals were selected, that were recorded in instrumented buildings through the California Strong Motion Instrumentation Program during three different earthquake events.

For each test, the input signals were incrementally scaled up or down, to induce different ductility levels to the steel fuses. The experimental campaign shed light on the benefits of detuning the non-structural component period from resonance with the period of the supporting structure by means of a controlled yielding element.

The relative displacements between the test specimen and the shake table were recorded via three string potentiometers, of which two were attached to the carriage and one was attached to one of the two fuses connected to the carriage in each test, as well as via a vision system with three cameras (see Fig. 2). Three three-dimensional accelerometers measured the achieved acceleration histories of the shake-table. Six additional three-dimensional accelerometers located in the South-West (SW) and North-East (NE) of the carriage measured the absolute acceleration demands of the carriage. Furthermore, a strain gauge was installed in one of the two steel fuses employed in each test to measure the uniaxial strain demands during earthquake shaking (see Fig. 1b).

The period and damping ratio of a test specimen was inferred by free vibration tests. It was found that a smartphone on top of each specimen was sufficient to accurately characterise the dynamic properties of the SDF system.

6. TEST RESULTS

The conducted test series (i.e., in total 45 tests) demonstrated that, the controlled yielding steel fuse concept to reduce the acceleration demands imposed to the non-structural components holds and the fuses at all cases were able to develop the intended ductile yielding mechanism.

Fig. 4 to Fig. 7, present test results undertaken for “Test No1”. Test No1 refers to a series of shake table experiments utilising Fuse #3 (i.e., a rectangular plate with a thickness of 5mm and a height of 260mm). The input floor motion was the GM93 floor signal, which corresponds to a floor motion that was recorded during the 1994 Northridge earthquake event at the roof of a 6-storey commercial building (Station No. 24370). It belongs to a group of records that were used in prior numerical studies undertaken by three members of this research team (e.g. [1,3,8]) and it is a characteristic example of a floor motion that has large acceleration amplifications at periods tuned to a higher vibration mode of the primary structure. The motion was incrementally scaled utilising four scaling factors, namely 0.75, 1.00 (unscaled case), 1.20 and 1.40 motion. Fig. 4 presents the elastic floor acceleration spectrum (for a component damping level equal to β_{comp}= 1%) for this floor motion. From this figure it can be inferred that, the computed floor spectrum has two peaks, the first signifying the tuning to the second mode period (T_{comp}= 0.45sec) of the building and the second to its fundamental period. Notably, for a non-structural component with a damping level equal to 1% and a vibration period equal to 0.45sec (i.e., equal to the second mode period of the supporting structure where this floor motion was recorded) the acceleration demand at the non-structural component reached a value as high as 3g.

The total mass of the carriage (mass of the carriage plus additional masses) to get an SDF component with a period of vibration equal to 0.45sec was 228kg. This mass resulted in an actual vibration period for the carriage equal to 0.47sec (very close to the valley on the right, next to the first spectrum peak, see Fig. 4) and a damping ratio of 3.81%. It should be noted that the damping ratios measured for other specimens of this experimental campaign varied between approximately 1% to 5.7%. Although, at least for the time being, the knowledge level with regards to the non-structural component damping ratios remains rather incomplete, the considered range is deemed quite representative [8].

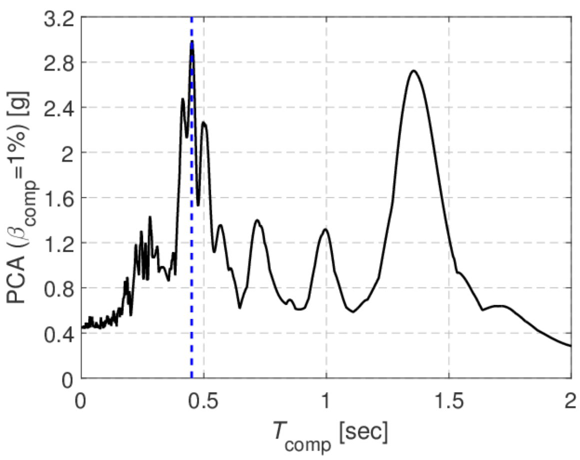


Fig. 4 Elastic floor acceleration spectrum for GM93 evaluated for a component damping level, β_{comp} = 1%. PCA denotes the Peak Component Acceleration

For the damping ratio that was measured during the free vibration test (β_{comp} = 3.81%) and using the acceleration signal that was recorded by the accelerometer located on the table (in the X-direction), the floor acceleration spectra illustrated in Fig. 5 could be evaluated for the 0.75/1.00/1.20/1.40-times scaled (Figs. 5a-d) input motions.

Figs. 6a-d illustrate the component acceleration (CA) histories along with the shake-table achieved floor acceleration (FA) histories for the four scaling factors that were used to scale up or down the initial floor acceleration motion and consequently induce different nonlinearity levels to the steel fuses. Referring to Fig. 6a (i.e., scaling factor = 0.75), the fuse response was nearly elastic, since, the recorded peak component acceleration (PCA

= 1.12g) was approximately equal to the evaluated elastic PCA (see Fig. 5a). Same findings hold true for the case presented in Fig. 6b (i.e., scaling factor = 1.00). The fuse experienced modest inelastic response, since the recorded inelastic PCA equals 1.31g and the elastic PCA at the actual component period (see Fig. 5b) differ by no more than 5%. For the two aforementioned cases the (inelastic) component acceleration amplification factor α_p, evaluated as the ratio of recorded PCA over the imposed PFA, were found equal to 3.37 and 3.07, respectively. The steel fuse experienced higher inelastic demands when the same record was amplified by a factor of 1.20. In this case, the recorded inelastic PCA equals 1.41g (see Fig. 6c) and the elastic PCA at the actual component period (see Fig. 5c) was approximately equal to 1.65g. By further amplifying the record utilising a factor of 1.40, the recorded inelastic PCA was found equal to 1.45g (see Fig. 6d) whereas the elastic PCA at the actual component period (see Fig. 5d) was approximately equal to 1.92g. For the latter two cases the (inelastic) component acceleration amplification factors α_p were found equal to 2.78 and 2.45. In brief, the results demonstrate that the steel fuses reduce the rate at which the acceleration demands imparted at the component level increase.

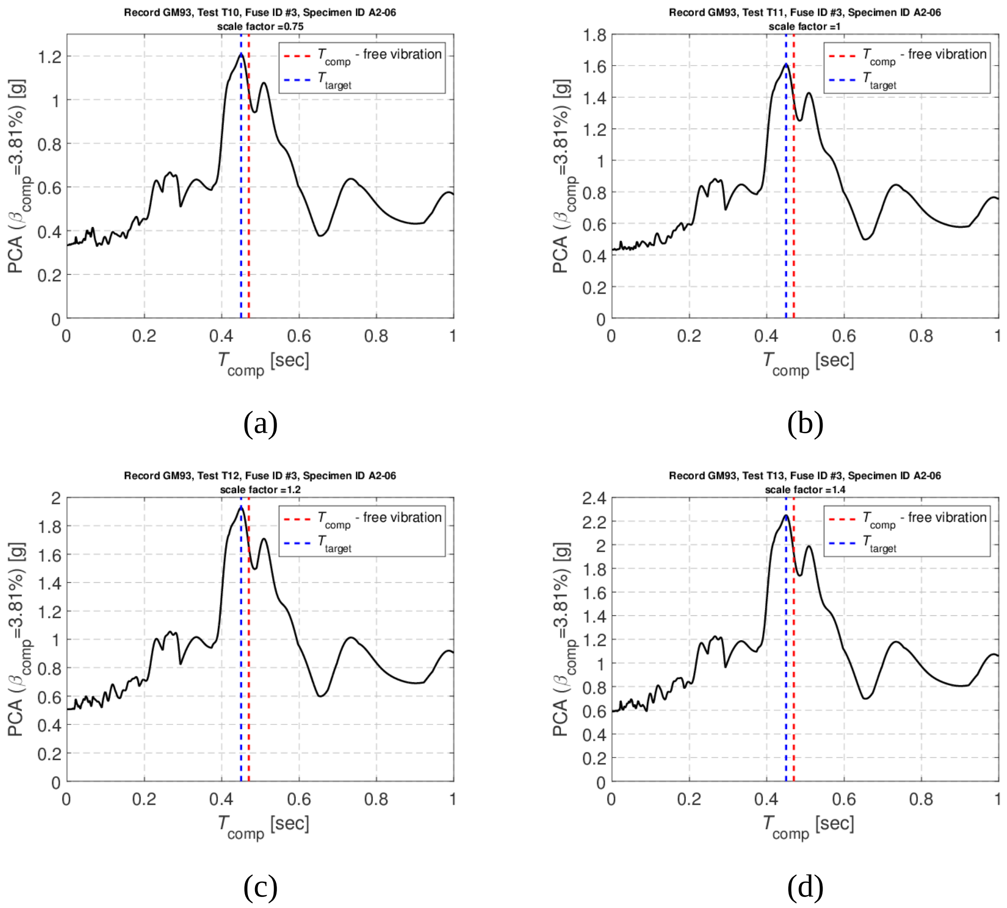


Fig. 5 Elastic floor acceleration spectra for GM93 (Test No1) evaluated for a component damping level, β_{comp} = 3.81% using the acceleration history recorded on the shake-table for (a) the 0.75-times (b) the 1.0-times (unscaled), (c) the 1.2-times and (b) the 1.4-times scaled input motion. The blue dashed line depicts the second mode period of the supporting structure and the red dashed line the actual period of the carriage as evaluated from the free vibration testing

For the signal scaled up by 40% the evaluated α_p (=2.45), is the lowest estimated for this test series. This essentially means that the addition of the yielding fuses to the system (even for moderate ductility levels as could be inferred from Fig. 7 for all tested specimens) resulted in the acceleration demands being reduced compared to the acceleration demands that would have been recorded if the carriage was left to behave purely elastically. This reduction is clearly manifested as the level of the inelasticity induced to the fuses is increased, although the benefit from increasing the ductility beyond certain levels is bounded. Apparently, even for low utilised fuse ductility levels the reductions attained at the component accelerations are notably high, since the introduction of the fuses diminishes the resonance effect that characterises the narrow-band floor spectra; a property that is not usually manifested in ground motion spectra of ordinary seismic records (i.e., far field records recorded on firm soils). This design strategy is clearly advantageous and substantially more controlled compared to a design strategy that relies on the inelastic response of the supporting building for mitigating the floor accelerations and consequently the accelerations imparted at the components.

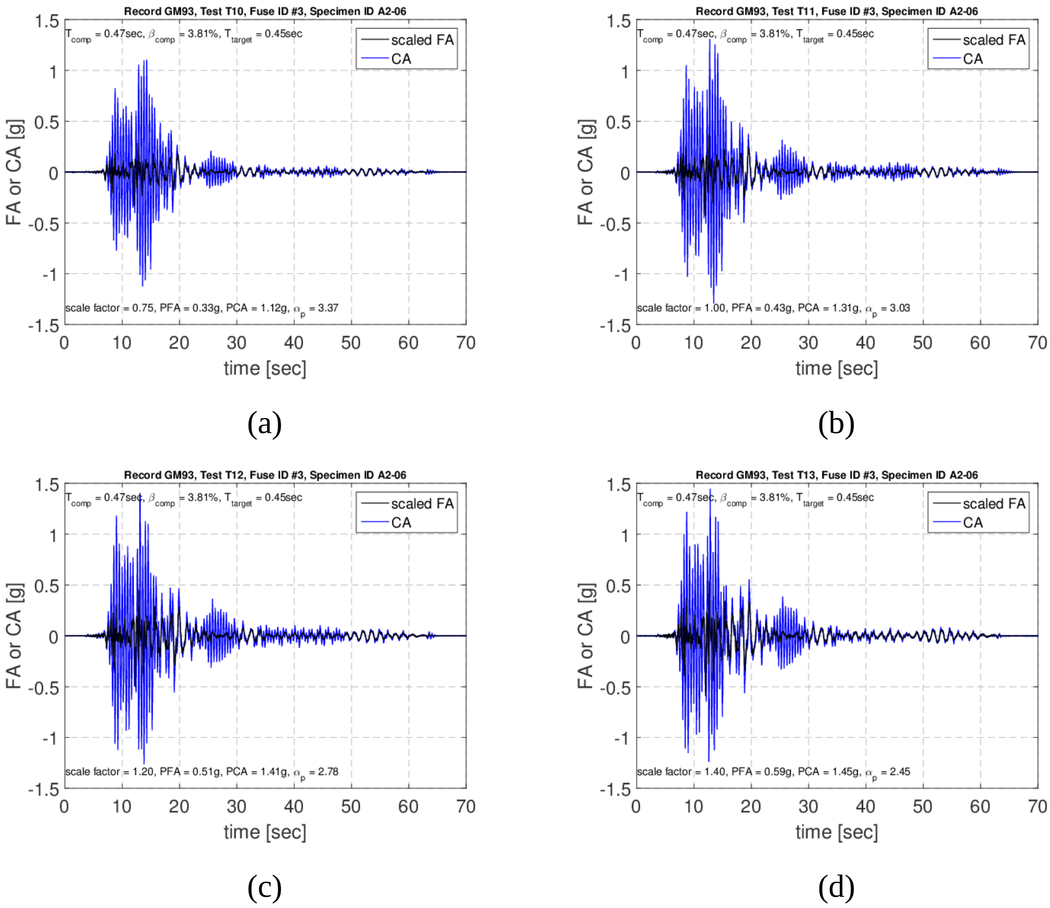


Fig. 6 Floor acceleration (FA) and component acceleration (CA) histories for GM93 (Test No1) and (a) 0.75, (b) 1.00, (c) 1.20, (d) 1.40 scaling factors

7. CONCLUSIONS

The NSFUSE test series explored the concept of using a simple yielding connector to reduce acceleration amplifications for non-structural components attached to buildings. The acquired data and preliminary results suggest that the above concept is promising. The results demonstrate that the protective design of non-structural components is doable, subject to the condition that a fuse of certified ductility and strength is provided by the pertinent manufacturer.

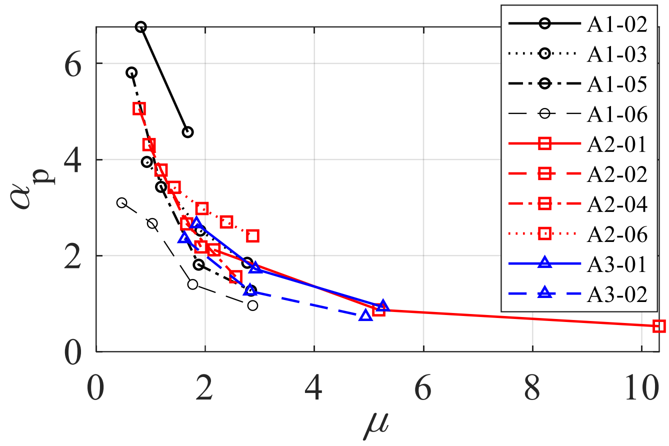


Fig. 7 Measured reductions in α_p as a function of ductility demand for the tested specimens

8. ACKNOWLEDGEMENTS

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Πλάστιμοι μεταλλικοί σύνδεσμοι για την αντισεισμική προστασία μη δομικών στοιχείων ευαίσθητων σε επιταχύνσεις: Αριθμητική και πειραματική διερεύνηση

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ΠΕΡΙΛΗΨΗ

Πρόσφατα σεισμικά γεγονότα κατέδειξαν την τρωτότητα των μη δομικών στοιχείων σε σεισμούς μικρής ή μέσης έντασης, οι οποίοι συμβαίνουν με σημαντικά μεγαλύτερη συχνότητα από εκείνους που θεωρούνται κατά το σχεδιασμό των κατασκευών. Ως εκ τούτου, κρίσιμες για το κοινωνικό σύνολο κτιριακές υποδομές συχνά χάνουν τη λειτουργικότητα τους για μεγάλα χρονικά διαστήματα παρά το γεγονός ότι έχουν υποστεί μόνο μικρές δομικές βλάβες. Η παρούσα δημοσίευση συνοψίζει τα αριθμητικά και πειραματικά αποτελέσματα ερευνητικής εργασίας που διενεργήθηκε στο πλαίσιο του προγράμματος NSFUSE στη σεισμική τράπεζα του University of Bristol. Η έρευνα επικεντρώθηκε στη διερεύνηση της θεωρητικής υπόθεσης ότι πλάστιμοι μεταλλικοί σύνδεσμοι που λειτουργούν ως δικλείδες ασφαλείας μπορούν να χρησιμοποιηθούν για την προστασία μη δομικών στοιχείων ευπαθών σε επιταχύνσεις. Απώτερος στόχος της εν λόγω μελέτης ήταν να προταθούν αξιόπιστα, οικονομικά και εύκολα αντικαταστάσιμα σε περίπτωση βλάβης στοιχεία, για την προστασία του μη δομικού κτιριακού εξοπλισμού. Για το λόγο αυτό πραγματοποιήθηκε μία σειρά πειραμάτων σε σεισμική τράπεζα με τη χρήση τριών επιταχυνσιογραφημάτων που αφορούν καταγραφές σεισμικών επιταχύνσεων ορόφων σε ενοργανομένα κτίρια στην Καλιφόρνια των ΗΠΑ. Μεταβάλλοντας τις ιδιότητες της πειραματικής διάταξης, διερευνήθηκε ένα φάσμα λόγων περιόδου μη δομικού στοιχείου-κτιρίου καθώς και διαφορετικά επίπεδα διαρροής. Τα πειράματα παρείχαν πληροφορίες για τη σεισμική συμπεριφορά των μη δομικών στοιχείων και τα πλεονεκτήματα που απορρέουν από τη χρήση ελεγχόμενα διαρρεόντων μεταλλικών συνδέσμων σε μη δομικά στοιχεία που βρίσκονται σε συντονισμό με μία από τις βασικές ιδιοπεριόδους του κτιρίου που τα περιέχει.