Experimental Study to Validate an Improved Approach to Design Acceleration-Sensitive Nonstructural Components

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Abstract. Nonstructural components in buildings can be subjected to very large acceleration and deformation demands during earthquakes. This is particularly true for flexible components that are tuned or nearly tuned to one of the modal frequencies of the supporting structure. To control the seismic demands in these situations, the authors have proposed a new design approach in which the connections between the structure and the nonstructural element are designed and detailed to experience nonlinearities to limit force and deformation demands. This paper summarizes an experimental campaign sponsored by the Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA) aimed at validating the proposed design approach. The research project involved subjecting 14 different specimens representing nonstructural elements, with masses ranging from approximately 200 kg to 800 kg, to severe floor motions recorded during the 1989 Loma Prieta and the 1994 Northridge earthquakes in three instrumented buildings in California. A total of 45 individual tests were carried out. The tests were conducted at the EQUALS laboratory shake table at the University of Bristol. Mass and stiffness were carefully selected in each specimen to have vibration periods that resulted in both non-tuned and tuned components to modal frequencies of the supporting structure. Furthermore, some of the tuned tests involved components tuned to the fundamental mode while others were tuned to the second mode of vibration of the supporting structure to examine possible differences. Lateral strength and primary energy dissipation were provided by two steel plates with rotations restrained at both ends and loaded out-of-plane. The tests demonstrated how the proposed approach greatly reduces force and acceleration demands while also reducing lateral displacement demands. Furthermore, tests also demonstrated that the proposed approach reduces the response sensitivity to the period ratio of the nonstructural element to that of the supporting structure leading to a reduction in seismic demands uncertainty.

Keywords: Nonstructural components, Shake table tests, New design approach, Recorded narrowband motions, Nonlinearity.

1. INTRODUCTION

Recent seismic events have highlighted the damage vulnerability of nonstructural components. This has been observed even in seismic events with low or moderate intensities; 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 or no structural damage during an earthquake. Notable examples of the aforementioned seismic performance are the Sylmar County Hospital in the aftermath of the 1994 Northridge earthquake [Naeim 2004] and the Santiago and Concepcion airports during the 2010 Maule earthquake in Chile, which sustained very little structural damage but massive nonstructural damage [Fierro et al. 2011; Miranda et al. 2012]. Similar observations have been made after recent earthquakes in New Zealand [Dhakal 2010; Ferner et al. 2014]

To this end, the engineering community in countries with modern seismic codes has shifted its attention to the development of robust methodologies for the evaluation of acceleration demands, that are imposed on nonstructural components, during an earthquake. An accurate absolute acceleration demand assessment could lead to an effective design strategy of nonstructural components. Recently, Kazantzi et al. [2020b] conducted a numerical study that evaluated the elastic/inelastic floor acceleration spectra for single-degree-of-freedom (SDF) secondary systems with linear/bilinear non-degrading hysteretic behavior. The study identified important characteristics of floor spectra of actual recorded motions as related to nonstructural components performance:

i. The building amplifies and filters ground motions leading to floor motions characterized by narrowband spectra. Therefore, acceleration demands imposed to nonstructural components can be strongly amplified if the component's fundamental period (T_p) is close to the building's fundamental, or higher mode, period (T_{bldg}); i.e. $\tau_m = T_p / T_{bldg} = 1$. This amplification, which is quantified as the ratio of the peak component acceleration (PCA) to the peak floor acceleration (PFA), can approach on average values close to eight and close to five for such tuned components with damping ratios of 2% and 5%, respectively.

ii. Allowing for some inelasticity to occur, either in the support or the bracing of the nonstructural component installed above ground on buildings, can substantially reduce acceleration demands with reductions much larger than those that occur from ground motions recorded on rock or firm soils which are characterized by wideband spectra; a conclusion that was initially introduced by Miranda et al. [2018]. This suggests that the well-known strategy of allowing nonlinearities to take place in structures during moderate and strong earthquakes can be even more effective in reducing force and deformation demands when used for nonstructural components if the nonlinearity occurs between the structure and the nonstructural element.

iii. Benefits of using an inelastic support system to protect nonstructural components from damage was later carefully quantified in Kazantzi et al. [2020a]; Kazantzi et al. [2020b] who conducted statistical studies of strength reduction factors and inelastic floor spectral ordinates by using motions recorded in instrumented buildings in California. They showed that allowing nonlinearity to occur in the component's support system results in acceleration demands that are much less sensitive to changes in the period of the component relative to the modal periods of the supporting structure (τ_m ratio), since even small level of nonlinearity lead to inelastic floor spectra that are substantially flatter compared to their elastic counterparts.

As part of a project sponsored by the Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA), an experimental testing program was undertaken to confirm the aforementioned numerical observations. The experimental testing program involved dynamic tests conducted at the shake table facility of the University of Bristol. The main short and long term objectives of the testing program were: (a) to demonstrate the -undesirable- potential for large amplified acceleration demands for components tuned to periods of modes of vibration strongly contributing to the seismic response of the supporting structure; (b) to demonstrate the conceptual validity of using nonlinear ductile steel fuses for decoupling and significantly reducing these acceleration demands; and (c) to show that the proposed approach leads to a reliable and inexpensive solution for the protection of acceleration sensitive nonstructural elements.

2. DESCRIPTION OF THE EXPERIMENTAL PROGRAM

2.1 Description of the Test Setup

The experimental testing program was conducted on the shake table within the Earthquake and Large Structures (EQUALS) laboratory at the University of Bristol. The shaking table at the EQUALS LABORATORY has a 3m by 3m platform supported by eight hydraulic actuators. The table can carry up to 15 tonnes and depending on the loading can reach acceleration levels up to about 5g with peak displacements of ±150mm, making it ideal for testing specimens to destruction.

Fig. 1 shows an overview of the test setup and its components. The setup mainly comprises of a mass-carrying rigid steel carriage that is resting on two cylindrical rollers; meant to represent the lateral movement of a non-structural component. The rollers rests on two track rails that are securely bolted to the shake table adapter plate. Four L-shaped guide sliders, installed on the carriage, are set in close contact with the grooves of the track rails to enforce unidirectional movement of the carriage and to block out-of-plane displacement and potential uplift during shaking.

The carriage is connected to the top end of two fuse plates (i.e., the deformable yielding element between the mass of the nonstructural element and the supporting structure) using a rigid assembly with double 20mm pins and shrink-fitted ball joints. Fig. 2b shows the typical geometry of the deformable fuse plates. The fuse plate comprises of three regions forming a dog-bone shape. The bottom region is clamped using pre-tensioned high-strength M16 bolts to a rigid block that is securely fastened to the shake table (i.e., fixed end). Similarly, the top zone is also clamped to the rigid pin assembly (i.e., free end). The middle zone is the deformable zone, which has a varying height, width and thickness, i.e., h, b and t, respectively in order to adjust the strength and stiffness of the lateral bracing of the carriage simulating the nonstructural component. Essentially, the carriage and fuse plates represent a single-degree-of-freedom (SDF) system as demonstrated in Fig. 2a based on the deformed configuration during a test.

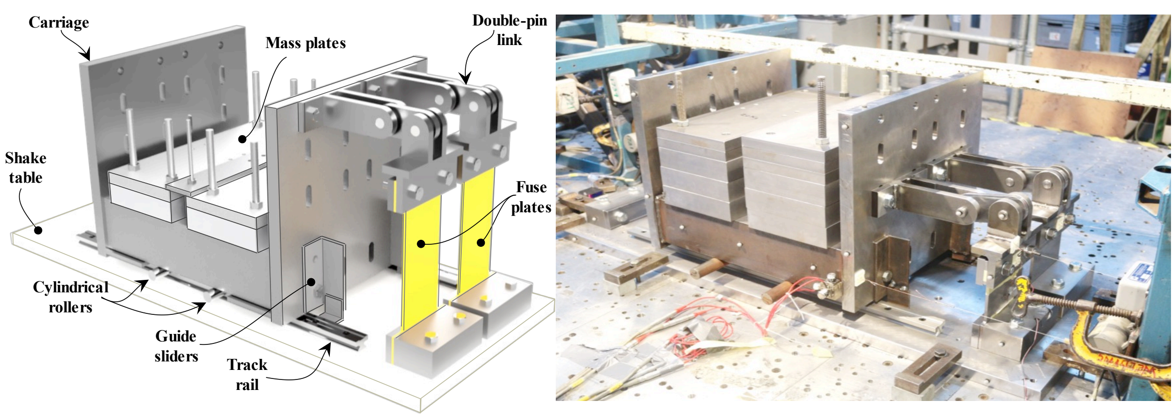
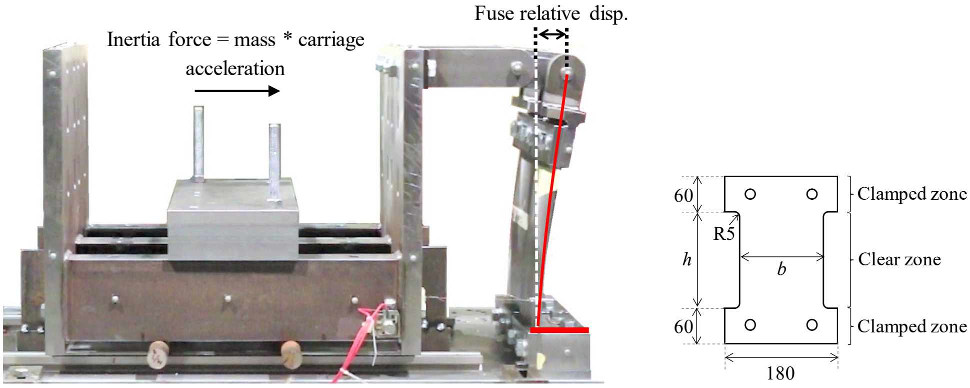


Fig. 1 Test setup overview



(a) (b)

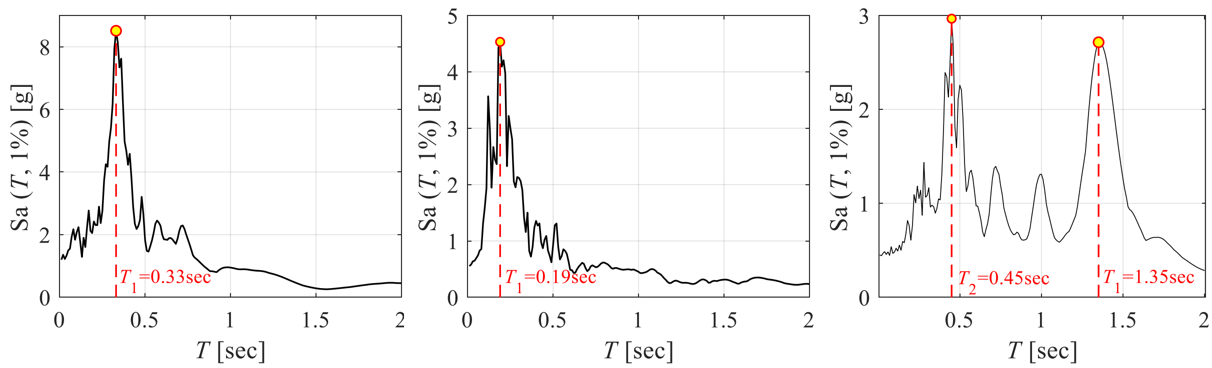
Fig. 2 Test setup overview and typical fuse dimensions

2.2 INPUT FLOOR MOTIONS

Unlike most prior shake table tests of nonstructural elements which typically have made use of artificial (synthetic) motions, this testing program made exclusive use of floor motions recorded in instrumented buildings in California to represent realistic severe motions that nonstructural components can be subjected to. Three recorded floor motions, labeled as FM14, FM21 and FM93 were selected to be used as input motions for the uniaxial shake table tests. These floor motions were recorded during the 1989 Loma Prieta and the 1994 Northridge earthquakes in three instrumented buildings in the United States [Kazantzi et al. 2022]. The details of each record are summarized in Table 1. For the first two cases, the floor acceleration spectra are characterized by large amplifications at or close to the fundamental period of the instrumented building (see Fig. 3a and 3b, i.e. T_1=0.33sec and T_1=0.25sec for FM14 and FM21, respectively) whereas the third case has its maximum floor acceleration spectral ordinate at the second modal period of the supporting structure (see Fig. 3c, i.e. T_1=0.45sec).

Table 1. Summary of floor records

Record ID	Earthquake event	Building type	Building height	T_{bldg} [sec]	Recorded floor	Duration [sec]	PFA [g]
FM14	1989 Loma Prieta	RC shear walls	4-story	0.33	Roof	40	1.20
FM21	1989 Loma Prieta	RC tilt-up walls	2-story	0.19	Roof	60	0.58
FM93	1994 Northridge	Steel frames	6-story	0.45	Roof	60	0.45



(a)FM14 (b) FM21 (c) FM93

Fig. 3 Acceleration response spectra for the three employed unscaled ground motion records.

2.3 Test Matrix

The carriage mass and fuse plate dimensions were modified in order to achieve specific target tuned periods of vibration and lateral strength on each test. As summarized in Table 2, the experimental program involved a total of 45 individual tests conducted on 14 specimens (i.e., 14 different combinations of fuse plate sizes and masses). As shown in this table in some cases, the target period was selected to be perfectly tuned to that of the modal period of interest of the supporting structure where the motion was recorded but some of the tests involved slightly shorter periods or slightly longer periods to investigate the sensitivity to the uncertainty in the period ratio, $\tau_m = T_p / T_{bldg}$. Each different specimen was then subjected to a single specific recorded floor motion scaled by different factors.

Table 2. Testing matrix and measured dynamic properties

Specimen ID	Fuse dimensions [b x b x t in mm]	Mass [kg]	Record ID	Scaling [%]	T_{target} [sec]	T_{actual} [sec]	ζ [%]
A1-01	150 x 150 x 8	202	FM21	50, 100, 150	0.15	0.15	1.18
A1-02		331		50, 100	0.19	0.20	2.96
A1-03		570		50, 100, 150	0.22	0.26	5.46
A1-04	200 x 180 x 6	202	FM14	20, 50, 100, 150	0.26	0.26	2.99
A1-05		357		20, 50, 100, 150	0.33	0.35	3.16
A1-06		454		20, 50, 100, 150	0.39	0.40	5.73
A2-01	80 x 38 x 10	806	FM14	30, 80, 150	0.26	0.28	1.11
A2-02	150 x 150 x 5	199		30, 50, 100	0.33	0.32	3.88
A2-04		199		40, 80, 150	0.33	0.32	4.52
A2-05	80 x 38 x 8	806	FM93	50, 75, 100, 150	0.36	0.39	1.66
A2-06	260 x 180 x 5	228		75, 100, 120, 140	0.45	0.47	3.81
A2-08		219		50, 140	0.45	0.46	2.61
A3-01	95 x 150 x 6	648	FM14	40, 80, 150	0.27	0.28	3.98
A3-02		806		40, 80, 150	0.31	0.32	1.07

2.4 Instrumentation

An instrumentation scheme was developed to capture accelerations and displacements at various locations. The instrumentation layout is shown in Fig. 4 which includes a combination of accelerometers, string potentiometers and a wireless displacement tracking system. In particular, two accelerometers are employed to measure the shake table and carriage absolute accelerations in the three orthogonal directions X, Y and Z. Eight string potentiometers are used to measure the in-plane (i.e., X-direction) displacements of the carriage and at the top end of each fuse plate relative to the table. For measurement redundancy, and to track possible uplift or twisting of the carriage, a wireless system with 12 light-emitting diodes (LED) targets was also employed to track absolute displacements of the shake table, carriage and fuse plate in the three-dimensional space.

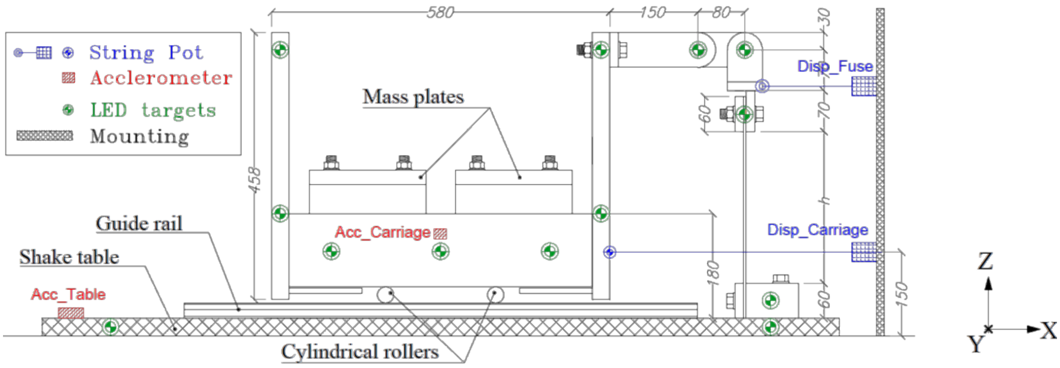


Fig. 4 Instrumentation layout

2.5 Testing Procedure

For each specimen, and prior to running the main test (i.e., shaking with recorded floor motions), a free vibration test was conducted in order to identify the system's dynamic properties; that is, its fundamental period of vibration (T_{actual}) and its equivalent viscous damping coefficient (ζ). Fig. 5 shows a sample of the decaying motion of the carriage displacement in one of the characterization tests as tracked by the wireless tracking system. As demonstrated in the figure, the system period is obtained as the difference in time between two consecutive decaying cycles' peaks or as the difference in time to complete a certain number of cycles divided by the number of cycles. The equivalent viscous damping coefficient is obtained based on the linear regression fit slope of the logarithm of the decayed carriage motion. When necessary, the mass was fine-tuned by slightly increasing it or slightly decreasing it in an iterative process to get closer to the target period (T_{target}). The values of the target and achieved (measured) periods and inferred equivalent damping ratios are summarized in Table 2. Typical inferred damping ratios ranged between 1 to 5 %. Damping in the test set up was mainly due to the interaction/friction between the carriage and the guiding rails, cylindrical rollers and pins. As such, higher damping values are generally observed in specimens with large mass due to the increased contact between the carriage, the rollers and the guide rails (see Table 2). In all tests, carriage twisting, and uplift was negligible. Out-of-plane accelerations were practically zero and vertical accelerations were below 0.1g.

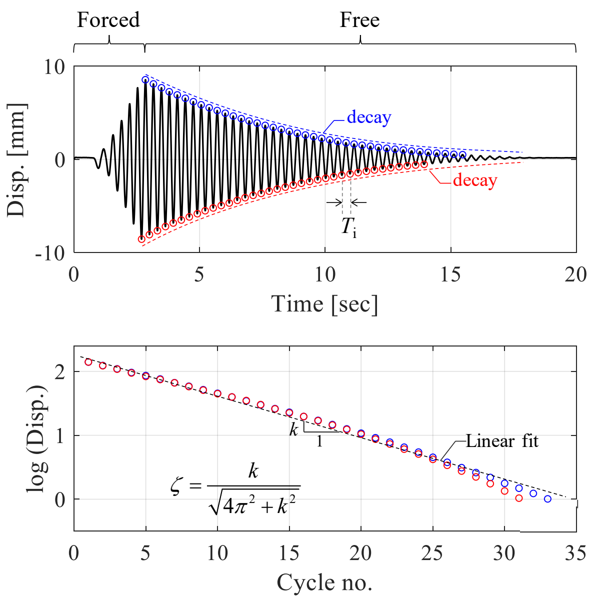


Fig. 5 Illustration of system period and damping coefficient inferred from the free vibration tests

2.6 MEASURED QUANTITIES

In addition to baseline correction, a second-order low-pass Butterworth filter was applied to all recorded signals (accelerations and displacements) with a frequency cutoff at 10Hz. The chosen filter type and parameters were efficient in removing noise from recorded signals without compromising the amplitude or main frequency content characteristics of the signals. This is demonstrated in Fig. 6 which shows a comparison of the acceleration history and Fourier amplitude spectrum (FFT) of the original and filtered time series.

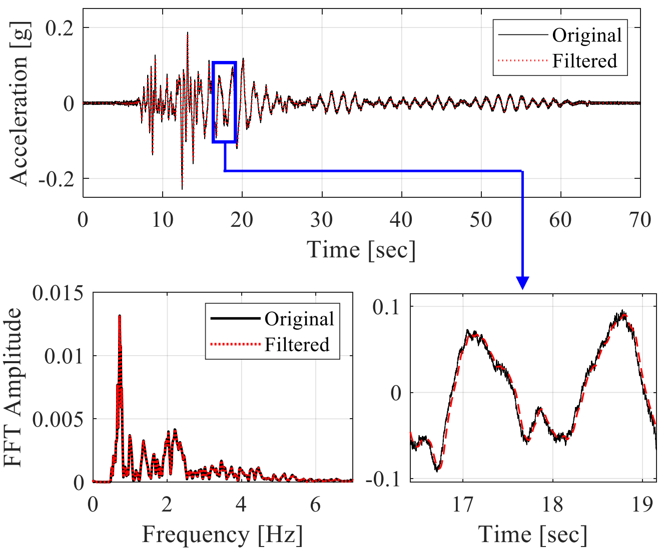


Fig. 6 Illustration of signal filtering using a 2nd-order low-pass Butterworth filter

3. TEST RESULTS

Figure 7a-c shows the inertia force versus the relative displacement fuse plate response of specimen A3-02 when subjected to the FM14 floor motion for three different intensity scaling factors. In the same figure, the elastic stiffness (dashed red line) and the plastic force (dashed blue line) are superimposed for reference. The elastic stiffness was deduced from the force-displacement curves, using data fitting, and validated using the period measured during the characterization tests. The plastic force is computed using Equation (1), where Z is the fuse plate plastic modulus about the axis of bending, f_y is the plate material's measured yield

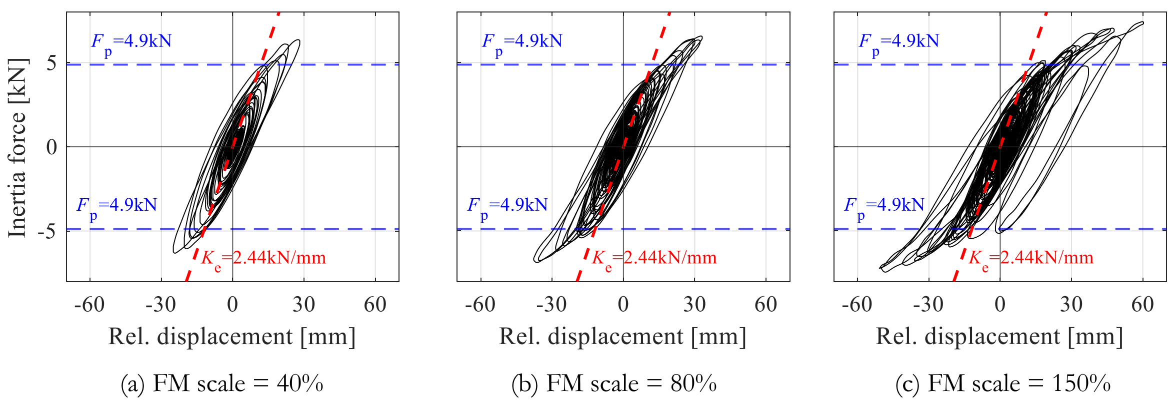


Fig. 7 Summary of force-displacement response for specimen A3-02

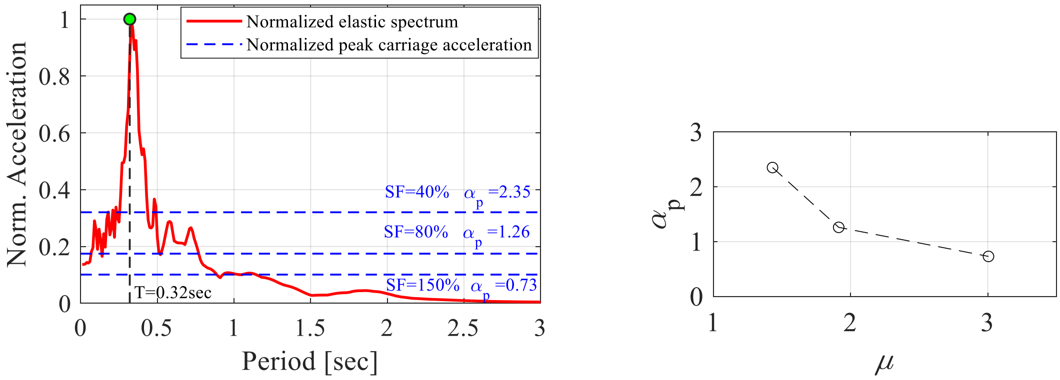


Fig. 8 Summary results for specimen A3-02

stress based on coupon testing and h_{eff} is the effective plate height which is taken as the plate clear length of the plate plus 140mm (distance to top pin center). Based on the computed K_e and F_p, the yield displacement of this plate was 20mm. This yield displacement was exceeded during the shaking at 40%, 80% and 150% scale factor as shown in Figure 7a-c, with peak displacements of 28, 38 and 59mm, respectively. This resulted in ductility ratios, μ, (ratio of peak to yield displacement of the carriage) that ranges from 1.5 to 3.

$$F_p = \frac{Z \cdot f_y}{h_{eff}} \tag{1}$$

A summary of the reductions in peak acceleration (and force) measured on the carriage on all specimens as a function of the level of nonlinearity is shown in Figure 9. It can be seen that, consistent with the analytical studies [Kazantzi et al. 2022], by allowing even relatively small levels of nonlinearity−such as ductility demands of two−significant reductions in force are achieved. The largest reductions are obtained for specimens whose period of vibration was perfectly tuned to that of the supporting structure where the record was obtained. This was true whether the carriage was tuned to the fundamental mode of the supporting structure or tuned to the second mode. Figure 8 shows the reductions in acceleration demands on the carriage as the level of ductility demand was increased. In the left-hand figure, the floor spectrum shown in red corresponds to the elastic acceleration ordinates normalized by the peak ordinate, whereas the figure superimposes -in blue dashed lines- the measured peak accelerations of the carriage, normalized by the peak ordinate of corresponding elastic spectrum; represented by the factor α_p. The extend of the reduction in acceleration demands with respect to increasing ductility in specimen A3-02 is plotted on the right-hand side of Figure 8. The figure demonstrated that by roughly doubling the ductility demand (from μ=1.5 to μ=3), the acceleration amplification is reduced by about two thirds.

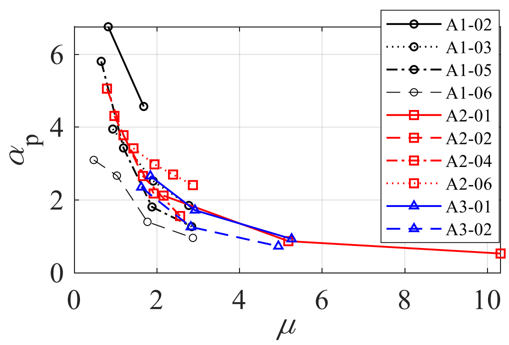


Fig. 9 Measured reductions in force demands as a function of ductility demand

4. SUMMARY AND CONCLUSIONS

This paper has summarized an experimental program that was undertaken to evaluate a novel approach to the design of nonstructural elements in which the bracing elements between the structure and the nonstructural element are designed and detailed to experience nonlinearities during moderate and strong earthquakes. The tests were conducted at the EQUALS laboratory shake table at the University of Bristol. The research project was sponsored by the Seismology and Earthquake Engineering Research Infrastructure Alliance for Europe (SERA).

The experimental program involved the uniaxial shake table testing of 14 different specimens representing nonstructural elements, with masses ranging from approximately 200 kg to 800 kg. Rather than using synthetic motions developed to match a certain floor spectrum specified in a seismic code or in a loading standard, specimens were subjected to motions recorded at the roof level of three instrumented buildings in California, thus the input motions that were used represent realistic intensities, frequency content and durations that nonstructural components can be subjected to. The mass, lateral stiffness and lateral strength of each specimen was modified to obtain specific periods of vibration to obtain perfectly tuned or nearly tuned components relative to the modal period of interest of the supporting structure. Eleven specimen had periods of vibration equal or close to the fundamental period of vibration of the building where the record was obtained, while three of the specimens had periods of vibration equal or close to the period of vibration of the second mode of building where the record was obtained, whereas the lateral strength was selected to obtain specific relative strengths compared to those required to maintain the component elastic. Each specimen was carefully instrumented with accelerometers, string potentiometers and optical (wireless) displacement tracking devices to measure absolute motions of the carriage and motions relative to those occurring in the shake table.

The tests demonstrated how by allowing nonlinearity to occur in the bracing elements, results in the forces acting on the nonstructural element being significantly reduced with respect to those that would occur if the bracing and component were to remain elastic. In particular, consistent with analytical studies conducted by the authors and reported elsewhere prior to the testing program, even small levels of nonlinearity such as a ductility ratio of two, are sufficient to produce significant reductions in the accelerations and the lateral forces acting on nonstructural elements. Reductions are larger for perfectly tuned components than those in nearly tuned components. Therefore, tests also validated that the proposed approach reduces the response sensitivity to the period ratio of the nonstructural element to that of the supporting structure leading to a reduction in seismic demands uncertainty.

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