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BENCHMARK TESTING AND PERFORMANCE COMPARISON OF SHAKING TABLES AND REACTION WALLS

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SUMMARY

Studies concluded in the framework of previous European Consortia of Earthquake Shaking Tables have compared the performances of the main European shaking tables (NTUA – Athens, Greece, EERC – Bristol, U.K., LNEC – Lisbon, Portugal, CEA – Saclay, France and Enel.Hydro, Seriate, Italy). This activity has induced an enhanced control at all the facilities and has highlighted the need to carry out further benchmark tests with extension to non-linear behaviour. In the meantime the new consortium ECOLEADER (European Consortium of Laboratories for Earthquake and Dynamic Experimental Research) was created and included the new reaction wall facility constructed at Ispra, Italy. Consequently this new benchmark activity was also extended to a parallel comparison between pseudo-dynamic testing on reaction walls and dynamic testing on shaking tables. Identical modular steel frames were built according to the capabilities of two different shaking tables (LNEC, Lisbon and NTUA, Athens) and to the necessary large scale for testing on the reaction wall at the JRC Ispra. Furthermore, two new partners showed their interest to be involved in this testing campaign and so the University of Trento (where a large reaction wall is recently available) and the University of Oxford (specialized in high speed on-line testing with substructuring) are also participating in this project. The 1-DoF steel specimen (3m long and high and 2.7m wide), easily transportable from lab to lab, was conceived at the University of Trento. It was designed to allow the insertion of two types of dissipation devices: one specially developed to insert a strong local non-linearity and another to insert a strain-rate sensitive effect. An 8250 Kg reinforced concrete mass is mounted on the top of the frame. In the present paper the design of the steel specimen and of the dissipative devices are discussed. The instrumentation plan and the test procedure are described. Finally, a critical analysis of the first results obtained and a performance comparison between pseudo-dynamic and dynamic facilities is presented. This action is part of the NEFOREEE project (New Fields of Research in Earthquake Engineering Experimentation), funded by the European Commission.

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INTRODUCTION

A concerted effort has been made, since the beginning of the nineties, by the main partners of the NEFOREEE project [1] to study the way in which shaking tables, and later reaction walls, really behaved. That is to say, the fidelity with which a specified input could actually be applied to a test-specimen, bearing in mind the mechanical and control imperfections.

Studies concluded in the framework of previous European Consortia of Earthquake Shaking Tables (ECOEST) have compared the performances of the main European shaking tables. The partners of these consortia were the Laboratory for Earthquake Engineering of the National Technical University of Athens, Greece, the Earthquake Engineering Research Centre of the University of Bristol, U.K., the Laboratorio Nacional de Engenharia Civil, in Lisbon, Portugal, the Commissariat a l'Energie Atomique, in Saclay, France and Enel.Hydro s.p.a., in Seriate, Italy.

This activity has induced an enhanced control at all the facilities and has highlighted the need to carry out further benchmark tests with extension to non-linear behaviour. In the meantime the new consortium ECOLEADER (European Consortium of Laboratories for Earthquake and Dynamic Experimental Research) was created by the former ECOEST partners, but then including also the new reaction wall facility constructed at the Ispra Joint Research Centre, in Italy.

Consequently this new benchmark activity was also extended to a parallel comparison between pseudo-dynamic testing on reaction walls and dynamic testing on shaking tables. Identical modular steel frames were built according to the capabilities of two different shaking tables (LNEC, Lisbon and NTUA, Athens) and to the necessary large scale for testing on the reaction wall at the JRC Ispra.

Furthermore, two new partners showed their interest to be involved in this testing campaign and so the University of Trento, Italy (where a large reaction wall is recently available) and the University of Oxford. U.K. (specialized in high speed on-line testing with substructuring) are also participating in this project.

In this paper part of a particular task of this new project is presented which deals with a rigorous comparison of the complementarity of shaking tables and reaction walls, by analysing their testing procedures relative with relative advantages and disadvantages.

A one-degree-of-freedom full-scale shear type inverted V-braced steel frame (3m long and high and 2.7m wide), easily transportable from lab to lab, was conceived at the University of Trento. It was designed to allow two types of dissipation devices to be inserted: one steel shear panel specially developed at the University of Kassel [2], Germany, to insert a strong local non-linearity; a viscous-elastic Jarrett device to insert a strain-rate sensitive effect [3]. An 8250Kg reinforced concrete mass was mounted on the top of the frame.

Two twin full-scale structures were constructed at the JRC Ispra. One structure is currently in preparation to be tested using the pseudo-dynamic method on the Ispra reaction wall. A second twin structure was sent by truck to the NTUA, in Athens, where it was already tested dynamically on the shaking table. Presently, this second structure is being dismounted and its steel components are being sent back to Ispra, together with the dissipation devices. After a complete revision, everything will be sent to the LNEC, in Lisbon, where the Athens test campaign will be repeated in order to allow also a comparison between the NTUA and the LNEC testing conditions. To perform the tests with the best possible similar structure, the large concrete top floor, cast and used in Athens, is being sent directly to Lisbon by boat.

Hereafter, the designs of the steel specimen and of the two dissipative devices are fully discussed. The instrumentation plan and the test procedure are described. The output of the tests performed at the NTUA shaking table is presented. Finally, a critical analysis of the results obtained is made. These results were obtained for the case of a full-scale specimen.

DESIGN OF A S.D.O.F. SHEAR-TYPE SPECIMEN

Design of main elements and passive energy dissipation devices

The main elements of the S.D.o.F. shear-type specimen in each direction were designed elastically on the basis of two earthquakes. The first one is the NS component of the 1940 El Centro earthquake while the second one shown in Fig. 1 is an artificial accelerogram matching Type 1 elastic response spectrum of Eurocode 8 (EC-8) [4]. It is illustrated in Fig. 2 and it derives from ground type A and 5 % viscous damping. Both are characterized by a peak ground acceleration of 0.5g. The choice of an artificial earthquake is justified by the will of exciting the S.D.o.F. shear-type specimen with the maximum force at the natural frequencies.

According to the design requirements the specimen has a longitudinal size of 300 cm; a transversal size of 275 cm; a column height of 450 cm; and an interstorey height of 300 cm.

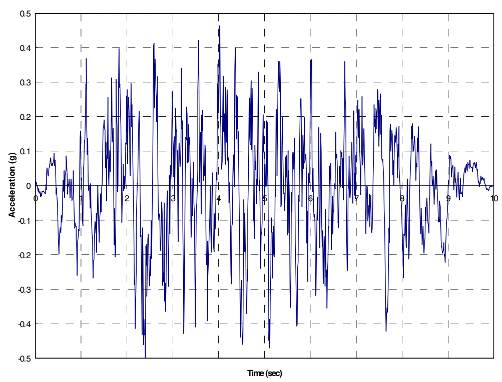

Fig. 1. Time history of an artificial accelerogram matching Type 1 EC-8 spectrum

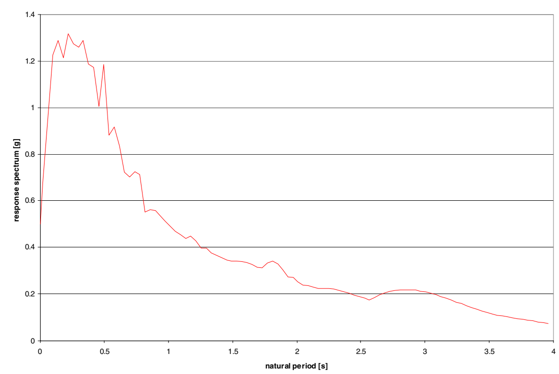

Fig. 2. Response spectrum of an artificial accelerogram matching Type 1 EC-8 spectrum

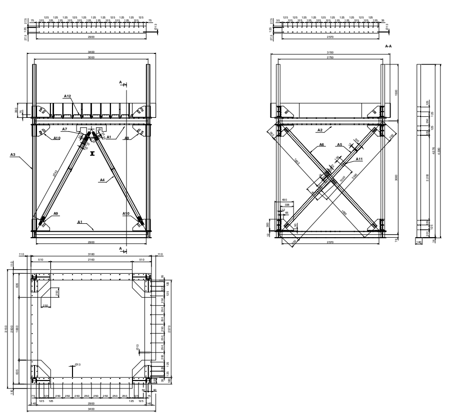

Fig. 3. Longitudinal, transversal view and plan of the S.D.o.F. specimen

Fig. 4. Longitudinal view of the S.D.o.F. specimen

Another steel-concrete composite slab can be added at a height of 600 cm and consequently, a second D.o.F. can be obtained simply using the elements with the same characteristics and details of the basic structure. One can easily observe the main characteristics of the specimen in Fig. 3 and a relevant global view in Fig. 4.

The choice of the steel sections of steel grade S355 is based on a compromise between flexibility and maximum admissible displacements tolerated by the shaking tables of the National Technical University of Athens and the National Laboratory of Civil Engineering of Lisbon; in particular, a structure with

fundamental frequency between 6 and 20 Hz was needed. Therefore, columns and beams were chosen to be HEB 100 and HEB 180 steel sections, respectively, while the inverted V and X braces were HEB 100 and UPN 100 sections, respectively. The connections between braces and devices are verified for maximum elastic forces that passive damper bear and are realized with bolts of grade 10.9. The connections between braces and bracing joints as well as bracing joints and beam and column elements are achieved with the same construction details, in order to prefabricate easily the structure. Sections and shapes of braces are chosen to allow: the crossing of the braces; the location of passive energy dissipation devices; and to grant the resistance to external forces. Inverted V braces with dampers in the upper position are located in the longitudinal direction; while X braces are chosen to grant stiffness to the frame in the transversal direction. Steel sections UPN 100 allow the crossing of braces in the central position. All selected steel sections satisfy Eurocode 3 checks [5].
The composite slab is limited in the bottom side by profiled steel sheetings and in the lateral sides by metal sheets as shown in Fig. 4. The thickness of the slab is taken to reach approximately the value of 10000 kg, corresponding to the maximum payload of the shaking table of the Technical University of Athens. The composite slab and the shear connectors satisfy Eurocode 4 checks [6].
The structure is equipped with energy dissipation devices in the longitudinal direction. In particular, two dampers have been designed. The first one is a steel shear panel device which was engineered by Schmidt and Dorka [2]; it is inserted into a square steel tube and dissipates energy by means of elastic-plastic shearing. Details of the damper are drawn in Fig. 5 while the relevant specimen is shown in Fig. 6.

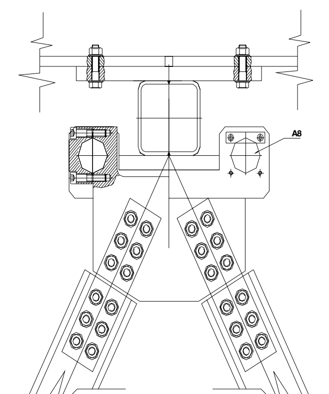

Fig. 5. Steel shear panel damper

Fig. 6. View of steel shear panel damper

The second one is the Jarret's energy dissipative device which relies heavily on the compression of a viscous-elastic silicone fluid. Extensive applications of this damper in braced structures have been made by Sorace and Terenzi [3]. The details of the damper are drawn in Fig. 7 while a view of the devices is reported in Fig. 8.

Preliminary numerical analyses
The S.D.o.F. shear-type specimen and the dampers with the features described in the previous subsection were modelled and analysed through the Sap2000NL computer programme [7]. The S.D.o.F. model was exploited to estimate the natural frequencies with different configurations of passive energy dissipation devices and to perform preliminary simulations.
In detail, the natural frequencies of the S.D.o.F. shear-type model of the specimen are summarized in Table 1. Three structures have been considered: i) Structure 1 without longitudinal braces; ii) Structure 2 which is endowed with longitudinal braces and shear panel dampers; iii) Structure 3 which is endowed with longitudinal braces and Jarret's viscous fluid dampers. Later on during physical testing, these

structures are labelled Specimen I, II and III, respectively. In particular, the first mode and the second mode define shear-type modes in the longitudinal and transversal direction, respectively; the third mode corresponds to a torsional mode; while the fourth one describes an axial mode in the vertical direction. Note that the fundamental frequency of Structure 3 depends on the elastic or inelastic stiffness of the Jarret's damper.

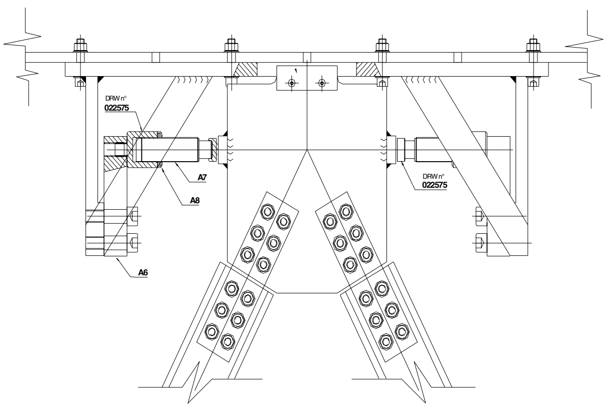

Fig. 7. Jarret's viscous fluid damper

Fig. 8. View of Jarret's viscous fluid damper

Table 1. Dynamic properties of the S.D.o.F. system

Frame	Mass [kg]	Damper	Frequency [Hz]			
			1st mode	2nd mode	3rd mode	4th mode
Frame with composite slab without braces (Specimen I)	9030	None	2.3	17.7	27.9	48.5
Frame with composite slab (Specimen II)	9943	Steel shear panel	7.8	18.4	25.6	39.7
Frame with composite slab (Specimen III)	9943	Jarret (initial/inelastic stiffness)	(8.4/3.4)	37.2	70.0	79.9

In order to perform simulations in the non-linear range, non-linear stress-strain relationships available in Sap2000NL programme have been exploited. In detail, the stress-strain relationship of the steel shear damper was reproduced through the Bouc-Wen model [8]. Therefore, the initial non linear stiffness k assumed equal to 27 kN/mm, the yield force equal to 40 kN, the ratio between the post yielding stiffness and the non linear stiffness equal to 0.2 and the exponent yielding equal to 2 were obtained through a fitting of experimental data.

The numerical model of the Jarret's damper was built by considering the device made up of a set of three elements: the first set is a pair of dampers; the second set is a pair of multi-linear elastic spring elements; the third set is a pair of gap elements, that are introduced to stop springs and dampers when the maximum stroke of the device is attained. The non-linear force-deformation relationship is defined by means of a piecewise linear curve. Again, the value of parameters was chosen to match experimental results.

Simulations of Structure 2 and 3 under stepped sine sweeps and earthquakes of different amplitudes were performed in order to highlight the behaviour of the Structures endowed with different dampers. For brevity, only the behaviour of dampers of Structure 2 and 3 subjected to the El Centro and to the artificial EC-8 accelerogram are illustrated in Figures 9-12.

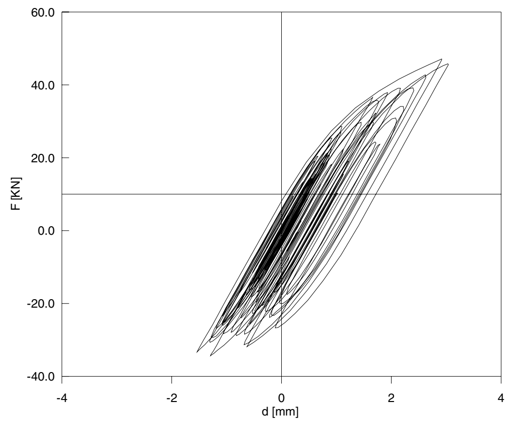

Fig. 9. Simulated hysteretic loops of steel damper subjected to the NS component of El Centro accelerogram with a 0.5g p.g.a.

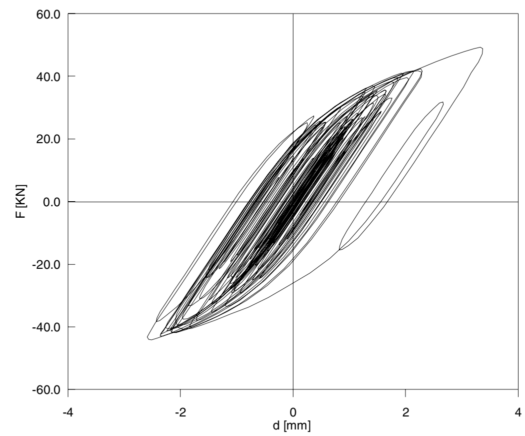

Fig. 10. Simulated hysteretic loops of steel damper subjected to the EC-8 artificial accelerogram with a 0.5g p.g.a.

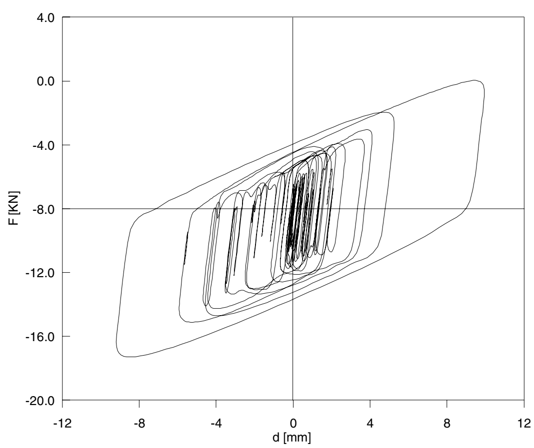

Fig. 11. Simulated hysteretic loops of Jarret's damper subjected to the El Centro accelerogram with a 0.5g p.g.a.

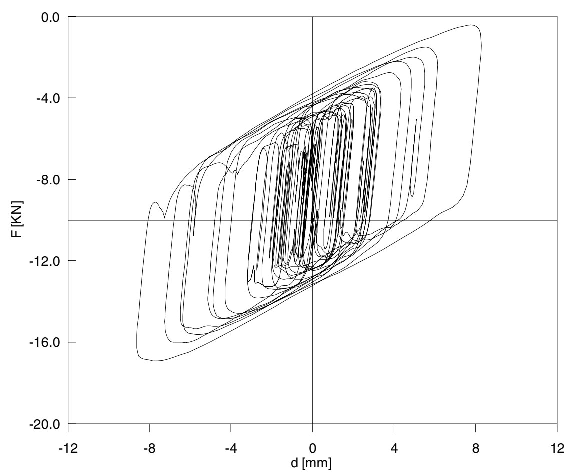

Fig. 12. Simulated hysteretic loops of Jarret's damper subjected to the EC-8 artificial accelerogram with a 0.5g p.g.a.

In particular, the time history of the response of the steel damper subjected to the El Centro earthquake and to the EC-8 artificial earthquake to a peak ground acceleration equal to 0.5 g is illustrated in Figure 9 and 10, respectively. Conversely, the response of the Jarret's damper in Structures 3 under the same excitations is shown in Figure 11 and 12, respectively. As expected, these simulations show clearly that the EC-8 artificial earthquake is more severe in terms of hysteretic cycles than the El Centro accelerogram at the same p.g.a. level. Therefore, the former accelerogram will be employed for the inelastic testing of dampers.

TEST PROCEDURE AND PRELIMINARY RESULTS

The primary test for each specimen was a uniaxial earthquake test using two different time histories in main X direction. These time histories were a modified component of El Centro earthquake (1940) and an artificial time history, which was generated to match the elastic response spectrum Type 1 of EC-8 [4] with peak ground acceleration of 0.50g, damping 5% and subsoil category A. Ramp sinusoidal excitation was performed as complementary test with test frequency 100% and 80% of natural frequency of each specimen. Prior the earthquake test, each specimen was tested under sine logarithmic sweep excitation along X direction in order its natural frequencies and their damping to be determined. The tests were carried out at the Laboratory for Earthquake Engineering of NTUA.

Sine logarithmic sweep

A sine logarithmic sweep signal has been applied. The frequency range is 1-35 Hz at a rate of one octave per minute. The tests were executed along global X axis with an amplitude of vibration of 0.50 m/sec^2. Since sine sweep rate is one octave per minute and the start frequency is 1 Hz, the exciting frequency versus time is given by the following expression:

$$f(\mathrm{Hz}) = 1.0 * 2^{\frac{\mathrm{TIME\ (sec)}}{60}}$$

These tests were performed in order to evaluate the natural frequencies and the damping of each specimen. The natural frequencies and the damping of each specimen are shown in Table 2. These frequencies were determined from acceleration time histories of the sine logarithmic sweep response of each specimen, while the damping ratios were calculated by using the half power bandwidth method [9].

Table 2. Measured natural frequencies and damping

Specimen	Frequency (Hz)	Damping (%)
I	2.600	3.02
II	8.574	5.30
III	8.000	17.0

Earthquake Tests

Uniaxial earthquake tests with two different time histories in the main X axis were performed. These time histories were a modified component of El Centro earthquake (1940) and an artificial time history, which was generated to match the elastic response spectrum Type 1 of EC-8 with peak ground acceleration equal to 0.50g, Subsoil Class A and damping 5%. In order to fit the available displacement capacity of the shaking table, both the El Centro accelerogram and the artificial time history were filtered with a high pass filter of 1Hz.

Additional to earthquake tests, a ramp sinusoidal excitation was performed with a test frequency 100% and 80% of the natural frequency of each specimen in order to achieve the resonant response.

The artificial acceleration time history and the modified El Centro earthquake are shown in Figures 13 and 14 respectively, while the corresponding elastic response spectrums are presented in Figures 15 and 16.

Specimen I

A uniaxial earthquake test with the El Centro time history in the main X axis was performed. Subsequently, tests were performed with the acceleration time history scaled to a peak table acceleration of approximately 0.025g, 0.10g and 0.20g.

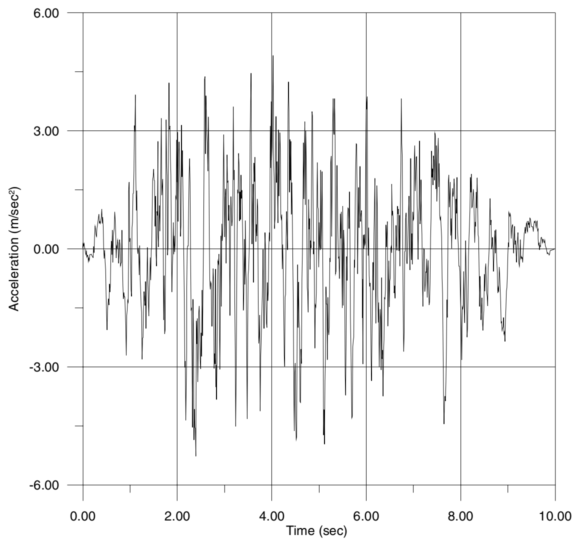

Fig. 13. Artificial time history.

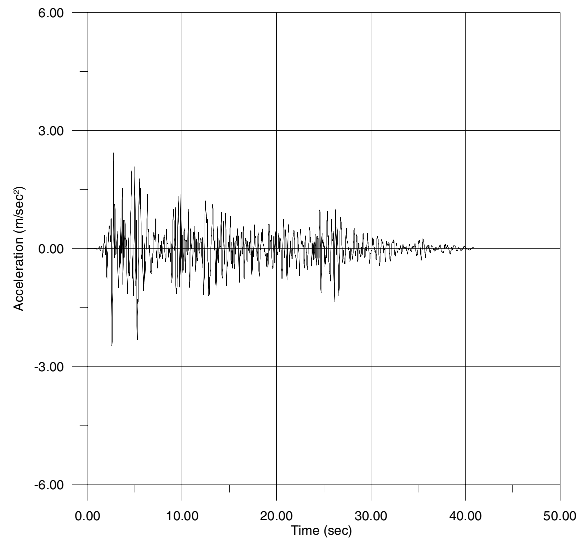

Fig. 14. Modified El Centro time history.

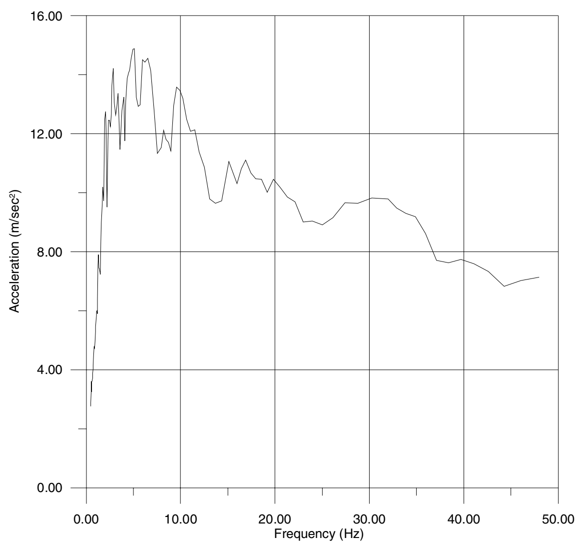

Fig. 15. Artificial time history elastic response spectrum with 5% damping.

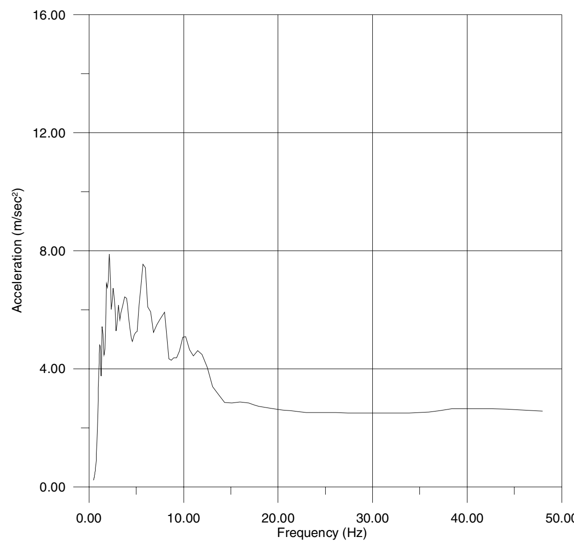

Fig. 16. Modified El Centro elastic response spectrum with 5% damping.

Specimen II

A uniaxial earthquake test with El Centro time history in X main axis was first carried out. Two tests were performed with the acceleration time history scaled to a peak table acceleration of approximately 0.10g and 0.20g. Then, a ramp sinusoidal excitation was carried out with a test frequency equal to the natural frequency of the specimen and the amplitude of excitation was 0.05g. Subsequent earthquake tests were performed using the artificial time history with the acceleration time history scaled to a peak table acceleration of approximately 0.10g, 0.20g, 0.30g, 0.40g, 0.50g, 0.60g, 0.70g and 0.80g. A ramp sinusoidal excitation was also carried out with a test frequency about 80% of the natural frequency of the specimen and amplitude of excitation 0.05g.

Specimen III

A uniaxial earthquake test with El Centro time history in the main X axis was first executed. Two tests were performed with the acceleration time history scaled to a peak table acceleration of approximately 0.10g and 0.20g. Then three uniaxial earthquake tests were performed using the artificial time history with the acceleration time history scaled to a peak table acceleration of approximately 0.10g, 0.20g and 0.30g. Two ramp sinusoidal excitations were followed with test frequency 100% and 80% of the natural frequency of the specimen. The amplitude of sinusoidal excitation was 0.10g for both tests. Subsequent earthquake tests were performed using the artificial time history. The acceleration time history was scaled to a peak table acceleration of approximately 0.40g, 0.50g, 0.60g, 0.70g, 0.80g and 0.90g.

Instrumentation

The instrumentation set-up that was used to measure the response of each specimen to earthquake tests is shown in Figures 17 and 18, respectively. Three accelerometers (A1X, A2X, A3X) were fixed on the top of the specimen in order to measure in and out of plane accelerations. Displacements at the top (D3, D4) and at the bottom (D1, D2) level of each specimen were measured relative to a stiff frame, which was fixed outside the shaking platform. Strain gauges (SG1, SG2) were also used in order to check the strain level at the end of steel columns. Laser sensors (L1, L2, L3) provided by the Joint Research Centre Laboratory were also mounted in order to measure vertical displacement. In specimens II and III, the shear force was measured directly on dissipation devices (LC), while two additional displacement transducers were mounted at these positions.

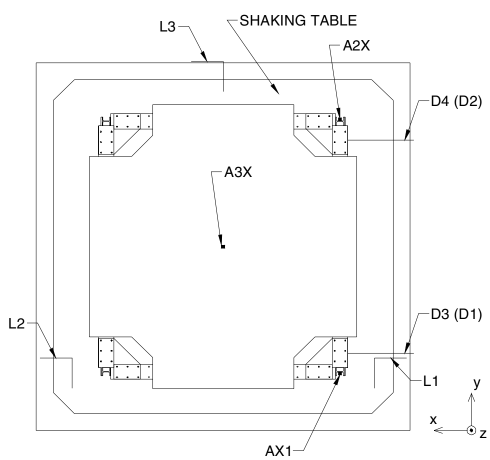

Fig. 17. Instrumentation set – up: plan view.

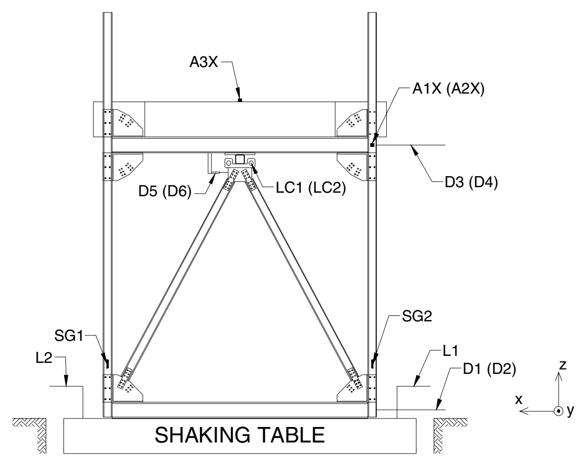

Fig. 18. Instrumentation set – up: lateral view

Hysteretic behaviour

The interaction diagrams of the shear force against relative top displacement of Specimen II and III under the artificial earthquake with peak acceleration scaled to 0.80g are presented in Figures 19 and 20 respectively, while in Figures 21 and 22 the hysteretic loop of dissipation devices are shown for the same earthquake test. Experimental responses presented in Figures 21 and 22 cannot be compared directly to the simulations presented in Figures 10 and 12 owing to the different level of peak ground acceleration.

Substructure technique

Besides the tests carried out at the Laboratory for Earthquake Engineering of NTUA other tests are in progress at the University of Oxford. Testing at Oxford is based on the use of the real-time substructure technique [10]. In this method, physical testing is performed only on the specific part(s) of the structure in which unpredictable non-linear or rate-dependent behaviour is expected – the physical substructure. The remainder, or numerical substructure, is modelled computationally. The interface forces and displacements between the two are provided by servo-hydraulic actuators. Since the response of each substructure has an immediate effect on the other, it is necessary for data to be passed between the two in the form of a real-time feedback loop. Running the test in real time allows an accurate simulation of the actual structural performance under earthquake loading, including resonance and rate-dependent behaviour in the physical substructure.

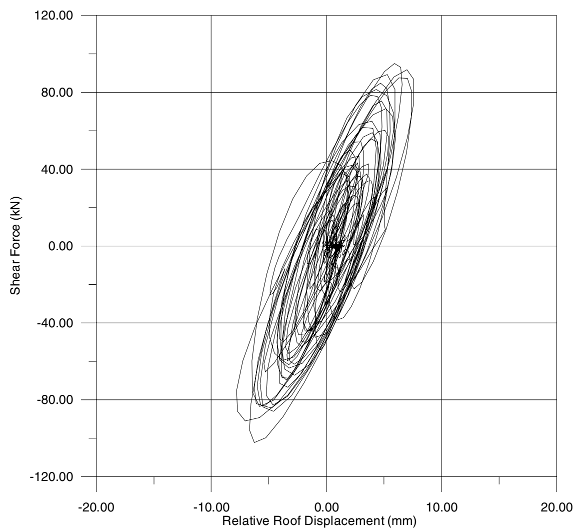

Fig. 19. Specimen II: Shear force – top displacement interaction diagram for the EC-8 artificial accelerogram with a 0.8g p.g.a.

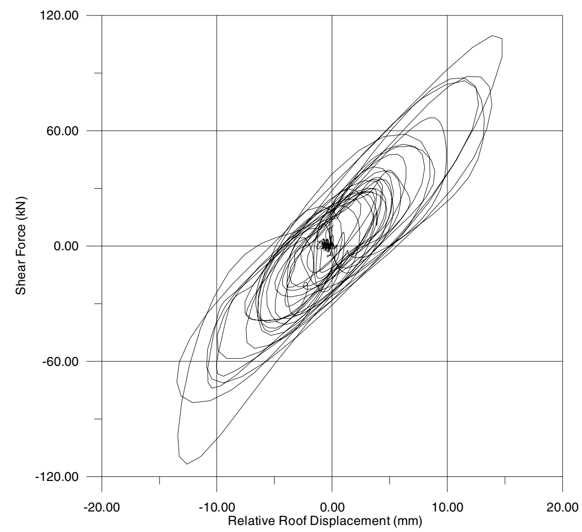

Fig. 20. Specimen III: Shear force – top displacement interaction diagram for the EC-8 artificial accelerogram with a 0.8g p.g.a.

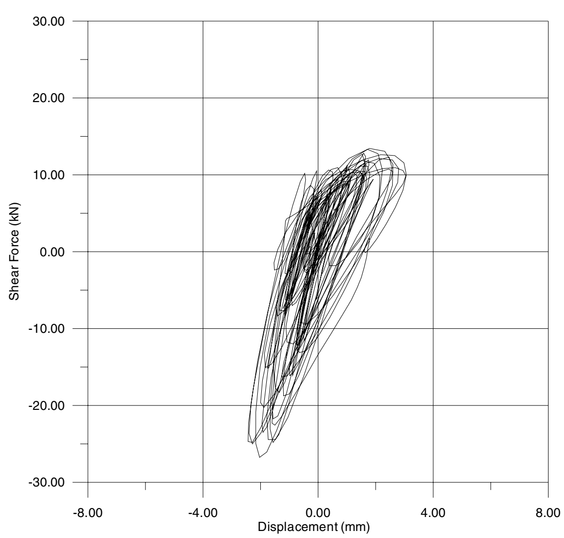

Fig. 21. Specimen II: shear force –displacement interaction diagram of a steel shear panel damper for the EC-8 artificial accelerogram with a 0.8g p.g.a.

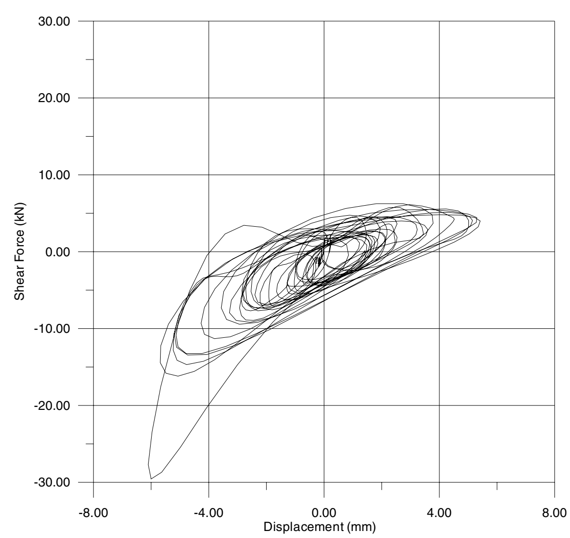

Fig. 22. Specimen III: Shear force –displacement interaction diagram of a Jarret's viscous fluid damper for the EC-8 artificial accelerogram with a 0.8g p.g.a.

In the current application, the physical substructure comprises only the non-linear device (yielding shear panel or Jarret damper) and its mounting. The remainder is simulated numerically. The physical substructure is mounted on a rigid support and connected to two 100 kN actuators aligned along the brace directions of the frame, as shown in Figure 23. To perform the test, the seismic input is first fed into the numerical model, and nodal displacements are determined. The relative displacements across the device are calculated and imposed by the actuators. The resisting forces thus generated are measured and fed back, along with the next seismic displacement increment, as input to the next timestep of the numerical model. This process repeats until the test concludes.

Figure23. The Oxford test set-up

CONCLUDING REMARKS AND FUTURE DEVELOPMENTS

The scope of the activity described in this paper is a reappraisal of the relative advantages and drawbacks of the two types of experimental facilities used for seismic testing, shaking table and reaction wall, and the circumstances under which their results can be compared. The benchmark structure and related tests campaign are very specific to illustrate the complementary of both testing approaches. The experimental procedures, boundary conditions and input excitation signal are thoroughly documented. In collaboration with several universities and European research institutions, a specific structure was designed to illustrate the drawbacks of the respective methodologies and to validate countermeasures leading to enhancements.
A single-degree-of-freedom shear-type benchmark structure was designed to exhibit a certain flexibility to be used in different laboratories for the comparison between the capabilities of shaking table and reaction wall tests. The material used is steel, because it allows specimens with good flexibility and easy connection details to be designed and to obtain the best modularity in order to be used in different laboratories. The braces with inverted V allows to work within a range of natural frequencies of interest for pseudodynamic and shaking table tests, reducing the maximum displacements during the tests as a good compromise between flexibility and mass requirement. Moreover the braces are designed in order to insert some passive-control devices, in this case a shear panel device or a Jarret's device were used.

Two twin benchmark structures were constructed. Details on the design of these specimens are given in the paper. One structure is currently in preparation to be tested using the pseudo-dynamic method on the ELSA reaction wall while the dynamic tests on the second structure were completed on the NTUA shaking table in Athens. The experimental results are presented here. After testing at NTUA, this structure will be dismounted and sent in Lisbon where the tests campaign will be repeated on the LNEC shaking table.
Furthermore high-speed substructuring tests are in preparation at Oxford University. In this case only the dissipation devices will be tested in laboratory and the rest parts of the structure will be modelled numerically. So the whole benchmark testing campaign will compare experimental results from two shaking tables (NTUA and LNEC) performing dynamic tests, from on reaction wall performing pseudodynamic tests (JRC) and from one high-speed on-line substructuring installation (Oxford). This action is part of the NEFOREEE project (New Fields of Research in Earthquake Engineering Experimentation), funded by the European Commission. The tests campaign will be completed by the end of 2004. The data will be mainly used to enhance the large-scale experimental facilities of the ECOLEADER partners.

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