

FREQUENCY AND DAMPING EVOLUTION DURING EXPERIMENTAL SEISMIC RESPONSE OF CIVIL ENGINEERING STRUCTURES

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SUMMARY: The results of the seismic tests on several reinforced-concrete shear walls and a four-storey frame are analysed in this paper. Each specimen was submitted to the action of a horizontal accelerogram, with successive growing amplitudes, using the pseudodynamic method. An analysis of the results allows knowing the evolution of the eigen frequency and damping ratio during the earthquakes thanks to an identification method working in the time domain. The method is formulated as a spatial model in which the stiffness and damping matrices are directly identified from the experimental displacements, velocities and restoring forces. The obtained matrices are then combined with the theoretical mass in order to obtain the eigen frequencies, damping ratios and modes. Those parameters have a great relevance for the design of this type of structures.

KEYWORDS: pseudodynamic test, seismic test, reinforced concrete, shear wall, framed structure, identification method, eigen frequency, damping ratio.

INTRODUCTION

Two different testing campaigns are described in this paper. In both cases the tests were performed by using the pseudodynamic (PsD) method at the ELSA (European Laboratory for Structural Assessment). The first one refers to 1-DoF shear walls, while the second one refers to a 4-storey frame.

The SAFE project included a series of seismic tests on 13 specimens of reinforced concrete shear walls. The differences among the specimens were in terms of the reinforcement, the theoretical mass or the presence of an additional normal load. The objective of the project was the generation of reference experimental data for the development and calibration of improved computational models for this type of structures.

The ICONS network project Topic 2 covers the subject of Assessment, Strengthening and Repair (ASR). Under this topic, a series of seismic tests were performed on a reinforced concrete 4-storey bare frame. The frame was built according to the construction practice in most of South European Mediterranean countries 40-50 years ago. The main objective of

these tests was to investigate possible seismic retrofit schemes for use in the upgrade of existing structures.

The complete analysis of the results of those projects is still under progress (Ile et al., Gonzalez et al, Pinto et al., 2000). This paper is intended to show some analyses based on an identification method of the frequency and damping ratio from the obtained experimental response.

PSEUDODYNAMIC TEST METHOD

Unlike in a shaking-table test, in a PsD test, a step-by-step integration of a discrete DoF equation is performed in a computer. Within this equation, a known matrix of mass is used whereas the non-linear restoring forces are experimentally obtained. To this purpose, at every step, the computed relative displacements are quasi statically imposed to the specimen and the existing forces at the actuators are then measured. These forces are subsequently fed back into the equation in order to compute the displacement at the next step.

The nature of the PsD method, by which realistic deformations are imposed to the specimen but at a very low rate, may lead to some advantages with respect to the shaking table. For example, much larger civil-engineering specimens can be tested or, particularly, a very high accuracy can be obtained in the imposition of the specified accelerogram and the measure of the response, which is mainly based on displacements transducers instead of accelerometers. Thus, a seismic response obtained from a PsD test is able to offer a good signal-to-noise ratio and becomes an excellent candidate for the application of identification techniques.

In order to obtain the PsD response of a structure, its equation of motion has to be modelled in a discrete-time fashion and in n_{DoF} discrete DoFs (Donea et al., 1996)

$$\mathbf{M}\mathbf{a}(n) + \mathbf{r}(n) = \mathbf{f}(n) \quad (1)$$

where $\mathbf{a}(n)$, $\mathbf{r}(n)$ and $\mathbf{f}(n)$ respectively represent the column vectors of accelerations, restoring forces and external forces at discrete time values $t = n T_S$ and \mathbf{M} is the mass matrix. In the case of an earthquake excitation, the external forces \mathbf{f} are computed as seismic equivalent inertial forces taking into account the ground acceleration and the mass of the structure.

As in a finite-element code, this equation is integrated step by step. The difference is that, in a PsD test, the restoring forces \mathbf{r} are not numerically modelled, but experimentally measured at every time step for the computed displacements. In order to do so, a servo hydraulic control system is coupled to the computer so that those displacements are quasi statically imposed to the specimen. For this reason, usually, the method is applied only to hysteretically damped structures, so that the restoring forces do not depend on the speed of deformation, which, for a PsD test, could be between 100 and 1000 times slower than reality.

IDENTIFICATION METHOD BASED ON A SPATIAL MODEL

The results of a PsD test may be used, among other applications, for the identification of the eigen modes of the tested structure. This type of parameter has a great significance in understanding the general behaviour or stating design rules. To this end, different time-invariant linear models can be used working in the time domain (Molina and Pegon, 1998, Molina et al., 1999). The advantage of working in the time domain is in the easiness to automate the identification process.

The results shown in this paper were obtained by using a spatial model. Within this model, the measured restoring forces $\mathbf{r}(n)$ and the corresponding displacements $\mathbf{d}(n)$ and velocities $\mathbf{v}(n)$ are assumed to be linked as

$$\mathbf{r}(n) = \mathbf{K}\mathbf{d}(n) + \mathbf{C}\mathbf{v}(n) \quad (2)$$

where \mathbf{K} and \mathbf{C} are respectively constant matrices of secant stiffness and viscous-equivalent damping. More precisely, the model can be formulated as

$$\begin{bmatrix} \mathbf{d}^T(n) & \mathbf{v}^T(n) & \mathbf{1} \end{bmatrix} \begin{bmatrix} \mathbf{K}^T \\ \mathbf{C}^T \\ \mathbf{o}^T \end{bmatrix} = \mathbf{r}^T(n) \quad (3)$$

where a constant force offset term \mathbf{o} has been added. Here, \mathbf{K} , \mathbf{C} and \mathbf{o} contain $2n_{\text{DoF}}^2 + n_{\text{DoF}}$ unknowns and the number of available equations is $N n_{\text{DoF}}$, so that, the required number of discrete-time data sets is

$$N \geq 2n_{\text{DoF}} + 1 \quad (4)$$

Once \mathbf{K} and \mathbf{C} have been estimated by a least squares solution, the complex eigen frequencies and mode shapes can be obtained by solving the generalised eigen value problem (Maia and Silva, 1997)

$$s \begin{bmatrix} \mathbf{C} & \mathbf{M} \\ \mathbf{M} & \mathbf{0} \end{bmatrix} \mathbf{w} + \begin{bmatrix} \mathbf{K} & \mathbf{0} \\ \mathbf{0} & -\mathbf{M} \end{bmatrix} \mathbf{w} = \mathbf{0} \quad (5)$$

where \mathbf{M} is the theoretical mass matrix. The conjugate couples of eigen values can be written as

$$s_i, s_i^* = \omega_i(-\zeta_i \pm j\sqrt{1-\zeta_i^2}) \quad j^2 = -1 \quad (6)$$

where ω_i is the associated natural frequency and ζ_i the damping ratio. The corresponding mode shape is also given by the first n_{DoF} rows of the associated eigen vector \mathbf{w}_i .

SAFE PROJECT SHEAR WALLS

Testing Set-up

The reinforced-concrete specimens consisted of a wall 3m long and 1.2m high with a thickness of 0.2m. In order to ease the load application, each wall was cast with 1.25m-thick upper and lower solid blocks and, moreover, right and left borders of the wall were provided with stiffeners that guaranteed out-of-plane stability (Fig. 1). A steel case fixed to the upper block served to the attachment of the actuators: five on right side and five on the left side with a total load capacity of 7 MN. The lower block was directly fixed to the strong floor of the laboratory. The actuators on the left reacted against the reaction wall of the laboratory, while those on the right reacted against a provisory reaction block set.

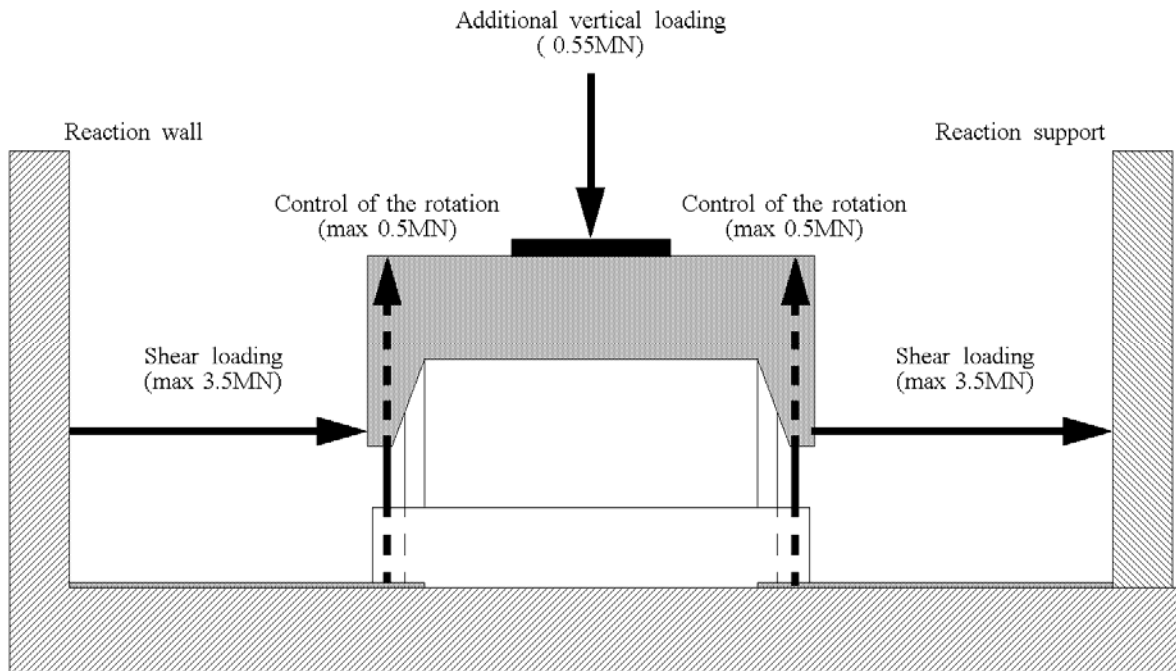


Fig. 1. Test set-up for SAFE project.

Right and left horizontal hydraulic actuators applied a shear load with resultant at mid height of the wall, while vertical actuators at the lateral borders prevented the upper block pitching. Those vertical actuators were controlled so that the total vertical load was null and the vertical displacements at right and left were equal. For some of the specimens there was, additionally, a vertical load of 0.55MN that was applied at the centre of the upper block by independent constant-force actuators. The imposed displacement to the structure was the horizontal one relative between the upper and the lower block as corresponding to a horizontal seismic input. That displacement was measured by two digital optical transducers with a resolution of $2\ \mu\text{m}$ and respectively positioned at the front and the back of the wall. Two of the ten horizontal actuators were fed back by those two displacement measures, while the other eight actuators were controlled to apply forces proportional to the ones of the first two.

During the test, the growth of several cracks as well as some relative displacements were recorded by a total number of 48 potentiometer displacement transducers. The position of some of those was decided only after the generation of the first cracks. The crack pattern, typically at $\pm 45^\circ$ was recorded on photographs.

This loading set-up allowed to impose shear deformations with a high accuracy on the wall specimen. Through the use of the PsD method, it served to obtain the seismic response. Within SAFE project, the direction of the earthquake was always horizontal longitudinal with respect to the wall (Left-right in Fig. 1) and the only DoF was the relative displacement of the upper block with respect to the lower one. The theoretical mass associated to that DoF depended on the adopted theoretical initial frequency and the nominal characteristics of the wall. The intensity of the design artificial earthquake depended also on those characteristics (Pegon et al., 1998).

Example: T12 wall

As an example, we will show here the experimental response and some relative analysis obtained for wall T12 within project SAFE. The reinforcement ratio of this specimen was of

0.11 either in vertical and horizontal directions. The additional vertical loading of 0.55 MN was present during the tests of this wall (Pegon et al., 1998). Fig. 2 shows the response to the five earthquakes applied to this wall, each with duration of 20 seconds. The first one had a maximum acceleration of 0.030g, which corresponded to the design earthquake (DE). For the adopted theoretical nominal frequency of 4 Hz, the design earthquake should produce a shear stress of 1.44 MPa. The inertial mass for the PsD integration was 11272 tons. The other four earthquakes were equal to the first one but multiplied by the increasing factors of 3, 5, 10 and 15. The upper graph of these figures shows the obtained displacement with respective maximum values of 0.25, 0.85, 2.6, 7.8 and 18.9 mm for the five accelerograms.

The lower graphs of Fig. 2 show the eigen frequency and damping ratio as identified by using the described spatial model. Since that model assumes an invariant system, at every time instant an identification was applied based on a data time window of two seconds centred around that instant. The adopted time window has to be narrow enough so that the system does not change too much inside it, but, at the same time, it has to contain enough data to allow the compensation of different existing data noises. The selection of the most appropriate window length is done by try and error. The tendency of the frequency is decreasing with the time as a consequence of the larger amplitudes and the accumulated damage. On the other hand, the tendency of the damping is also growing but much more erratic.

Looking at these results, it is clear that with an only value of the eigen frequency and damping ratio it would not be possible to explain the response of this structure to the applied accelerograms. Nevertheless, it is still possible to find a correlation between eigen frequency and oscillation amplitude. To do so, we have computed a displacement amplitude by using the Hilbert Transform as obtained through the Fast Fourier Transform (Herlufsen, 1984). This technique has the advantage of being easy to automate. The obtained amplitude has then been averaged over the same 2-second time window as the one used for the identification of frequency and damping. The result is shown at the upper graph of Fig. 2 in dashed lines.

Now in Fig. 3, for every earthquake, we have traced the evolution of the wall T12 (solid lines) in an eigen frequency-displacement amplitude axis system, using logarithmic scale for the displacement. At the same, in the same axis, we have also represented the linear response spectrum for every earthquake for damping ratios of 4 and 8% (dashed lines). For consistency, those spectra were also computed based on a two-second averaged displacement amplitude.

The first observation from this figure is that the correlation between displacement amplitude and frequency is strong in the response to a wide range of earthquakes. Such correlation is negative and follows more or less a general straight line of decreasing frequency for increasing amplitude whenever the current amplitude is attained for the first time in the history of the specimen. However, in those other moments in which the oscillation amplitude is lower than the attained maximum (branches internal to the envelope in the graph), the changes of frequency are not so strong and can even have the same sign as the changes in amplitude. An increase of frequency with an increase of amplitude is due to the pinching of the open cracks.

The second interesting observation from the same figure is that, in the case of knowing a priori the envelope of that general correlation, it would be possible to predict the maximum amplitude of the response by finding the intersection with the linear spectrum for roughly 8% of damping. For wall T12 this is true at every earthquake except the first one, for which a 4% damping spectrum is more appropriate.

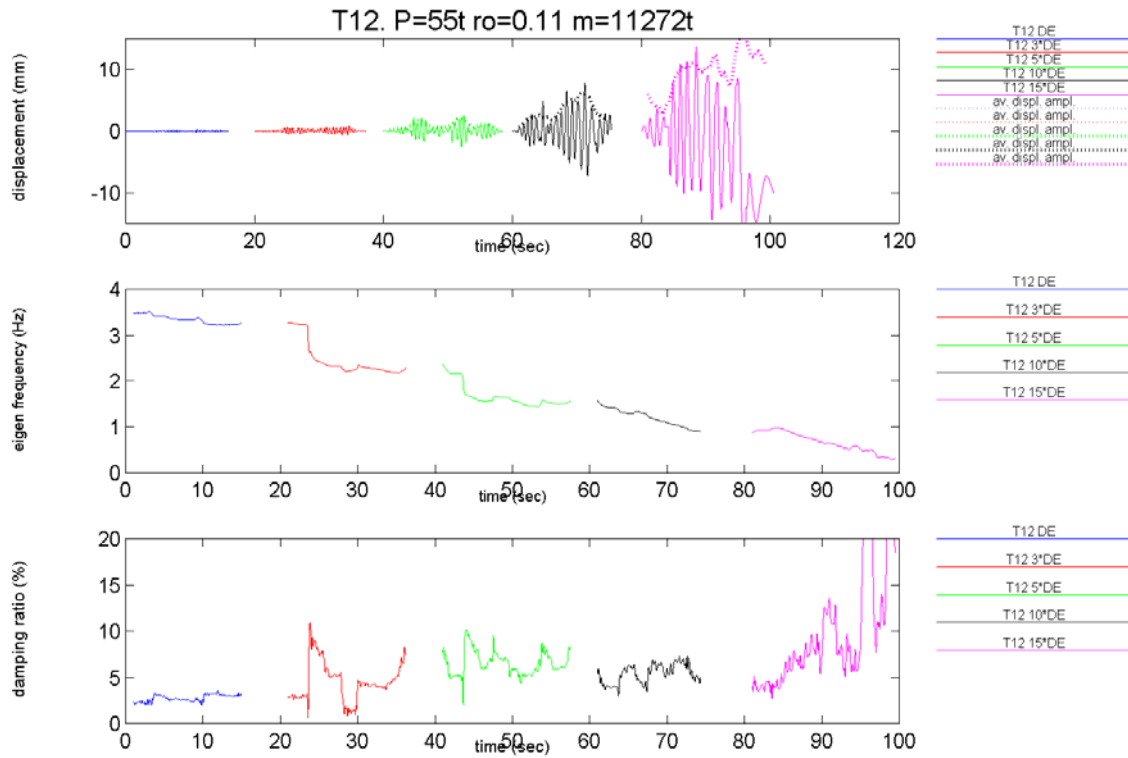


Fig. 2. Seismic response of wall T12.

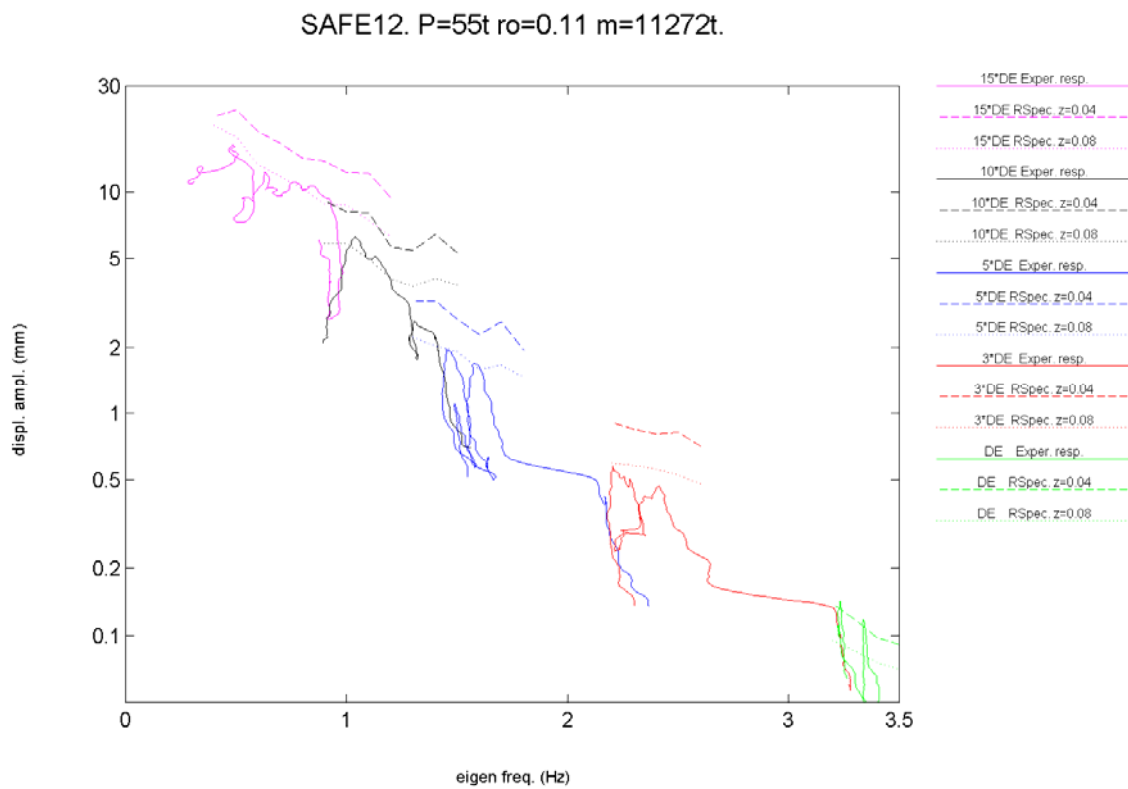


Fig. 3. Correlation between displacement amplitude and frequency of wall T12.

Comparison among the specimens

The main difference among the specimen walls of the SAFE project was the vertical and horizontal reinforcement ratio which ranged from 0.11% to 0.8%. However, the effect of the reinforcement on the cyclic behaviour of these walls is not clear and is still under analysis. Apparently, the differences among specimens with the same reinforcement was comparable to the ones among specimens with different reinforcement (Gonzalez et al., 2000). There is no space in this paper to show the graphs for each case, but we can see in Fig. 4 that the envelope for all the tests of most of the specimens could be very similar. Nevertheless, some differences may be detected in the behaviour of the internal branches corresponding to oscillation at displacement amplitude lower than the attained maximum. This could be due probably to the presence of normal load that could delay the effect of pinching. Note that, in this figure, the horizontal axis refers to stiffness and not to frequency as before. We introduced this variable here in order to be able to compare walls with different theoretical inertial masses used in the PsD integration.

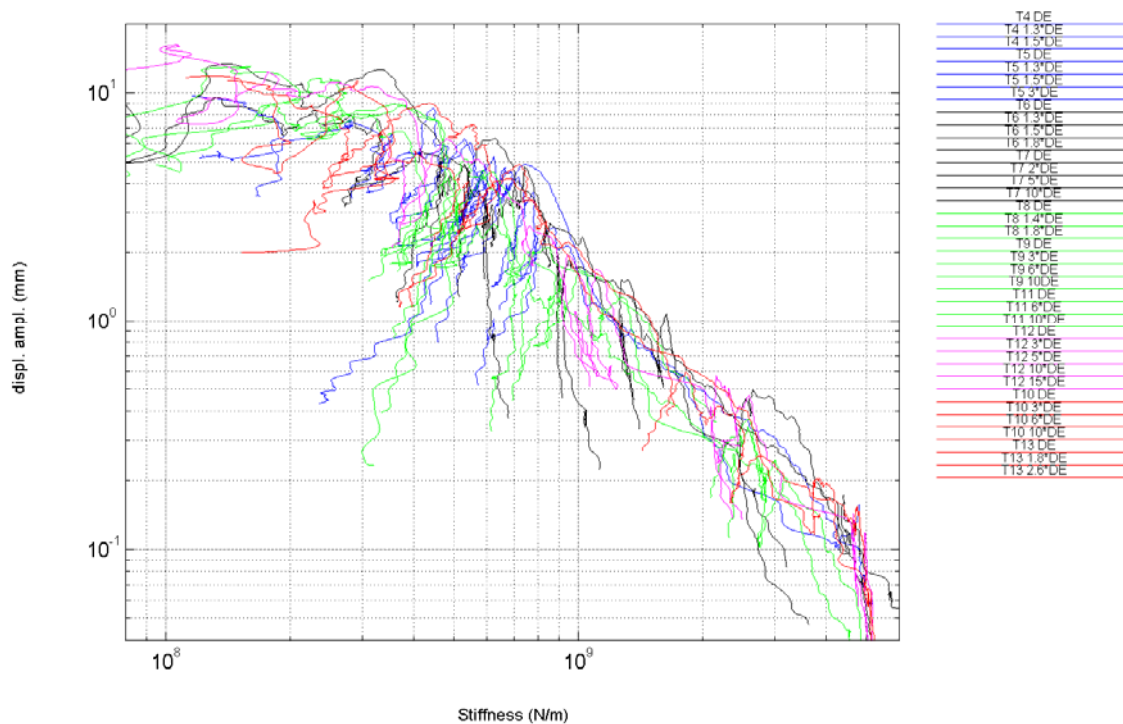


Fig. 4. Correlation between displacement amplitude and stiffness of walls T4 to T13.

ICONS ASR BARE FRAME

Testing Set-up

The general layout of the structure is given in Fig. 5. The concrete frame was designed essentially only for gravity loads and a nominal lateral load of 8% of its weight was assumed. The reinforcement details were specified to be representative of buildings constructed over 40 years ago in European countries such as Italy, Portugal, Spain and Greece.

The building is a 4-storey frame with three bays, two with 5m span and one with 2.5m span. The Inter-storey height is 2.7m and a slab 4m wide and 0.15m thick was cast together with the beams. Equal beams (geometry and reinforcement) were considered at all floors and the columns have equal geometric characteristics, except the stocky column, which develops in the 1st and 2nd storey with dimensions (0.6x0.25) and in the 3rd and 4th storeys with (0.5x0.25).

It should be underlined that the reinforcing steel consisted of smooth round bars and not the deformed steel bars used for reinforcement today. All beams in the direction of loading are 250mm wide and 500mm deep. The transverse beams are 200mm wide and 500mm deep.

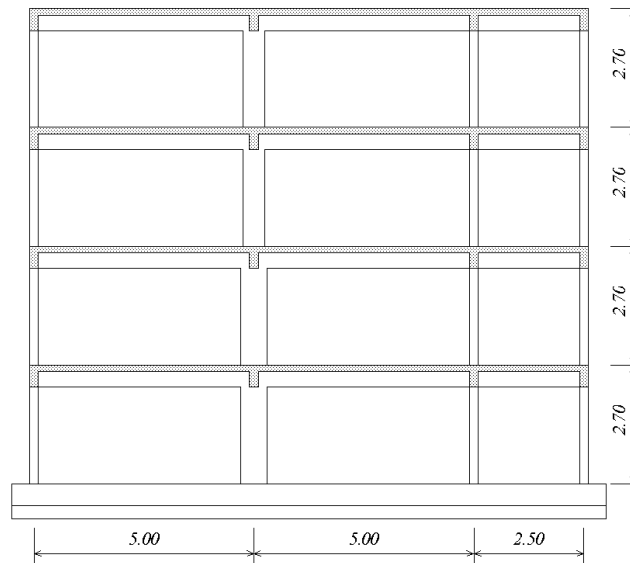


Fig. 5. Elevation view of ICONS-ASR reinforced concrete bare frame

Vertical distributed loads on beams and concentrated loads on the column nodes were considered, in order to simulate the dead load other than the self-weight of the frame (live-load, weight of partitions, finishings).

The PsD displacements (4 DoFs corresponding to the horizontal longitudinal floor displacements) were imposed at each floor by a 500 kN hydraulic actuator that was attached over the central bay of the floor deck.

Executed tests

The PsD tests performed on this frame (Pinto et al., 2000) corresponded to the artificial 15-second duration accelerograms associated to earthquakes with 475 years (2.180 m/s^2) and 975 years (2.884 m/s^2) of return period for a representative European area with moderately high risk. As predicted by the numerical pre-calculations, the frame did not perform satisfactorily and the wide column at the third level suffered severe damage. In fact, the 975-year return period earthquake was applied only for 7.5 seconds in order to limit that damage. At that moment, in order to improve the seismic performance of such structure, selective repair and retrofit interventions were undertaken. Afterwards, the selectively repaired structure was submitted again to the 475, 975 and additionally to the 2000-year return period accelerogram (3.728 m/s^2). After this second series of earthquakes, the frame had developed larger global displacements and some limited damages, but its stability was not compromised thanks the applied retrofitting.

By taking pieces of data of one-second duration, the described identification method was applied to the response of this frame to those earthquakes. This rendered the four eigen frequencies, damping ratios and mode shapes and their time evolution. In order to extend the techniques applied to the previous example to this case with more than one DoF, we have worked with the frequency of the first mode and the top displacement of the frame. Such displacement is mainly due to the response of the first mode. Then, using also a time window of one second for averaging the amplitude of that displacement, the correlation with the frequency is shown in Fig. 6. This figure also shows the respective response spectra of the earthquakes for damping ratios of 4 and 8%. These spectra were calculated also in averaged displacement amplitude and the result was multiplied by a factor of 1.35, which corresponded to the extrapolation to the last floor when assuming a linear first mode shape for this frame. It is interesting to see how the response to all the earthquakes follows again a common envelope, which more or less intersects the response spectrum at the maximum response for every earthquake. Due to the initial design deficiencies, that envelope had been incidentally lost in the middle of the second earthquake. Then, for the last three earthquakes the regular behaviour was recovered thanks to the applied selective retrofitting.

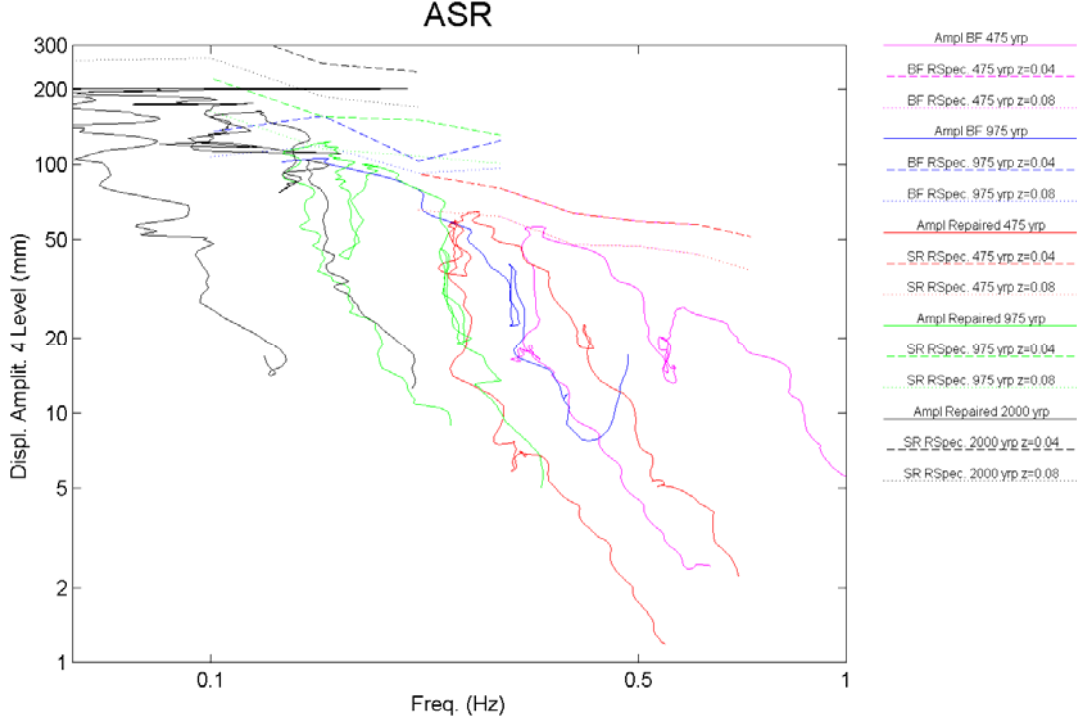


Fig. 6. Correlation between displacement amplitude and frequency for ICONS-ASR frame.

CONCLUSIONS

The high quality of the results obtained from PsD tests allows applying powerful identification techniques. The identified frequencies and dampings of the tested structures render an effective analysis of their behaviour that possibly may improve the design methods. For example, the obtained correlation between the measured displacement amplitude and eigen frequency showed a good match with the response spectrum of the respective accelerogram for the single DoF walls. This approach was also successfully extended to the

case of the multi-DoF frame by working with the top displacement and the first eigen frequency.

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ACKNOWLEDGEMENTS

The tests described in this paper were performed within the frame of the following projects:

- SAFE, contract with COGEMA (Compagnie Générale des Matières Nucléaires) and EDF (Electricité de France).
- TMR "Networks" (ICONS, Topic 2 - Assessment, Strengthening and Repair) and TMR "Large-Scale Facilities" of the European Commission.