ISET Journal of Earthquake Technology, Paper No. 406, Vol. 37, No. 4, December 2000, pp. 143-163

DYNAMIC COLLAPSE TESTING OF A FULL-SCALE FOUR STOREY RC FRAME

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ABSTRACT

A full-scale reinforced concrete frame was recently tested using the pseudo-dynamic approach at the ELSA laboratory at JRC (Ispra, Italy) as a joint activity within a European training network (ICONS). The frame has four storeys and three bays and is intended to represent typical design and construction practice in most South-European countries in the 1950's. After a first round of experiments, the damaged model was repaired using selective techniques and then re-tested under identical input motion. In this paper, the design and construction of the full-scale test specimen is firstly described, followed by an overview of the test-rig set-up, including the distribution of additional masses, the loading system configuration and instrumentation plan. Finally, the test procedure is described and the results summarised.

KEYWORDS: Pseudo-Dynamic, Testing, RC Frame, Selective, Retrofitting

INTRODUCTION

Unlike traditional testing methods, such as cyclic and shaking-table testing, pseudo-dynamic (PSD) testing is a hybrid procedure which combines classical experimental techniques with on-line computer simulation of structural dynamic behaviour. This method was first introduced in the mid-70's by the research group at the Institute of Industrial Science, University of Tokyo, using analogue computers and linear actuators (Takanashi et al., 1977, 1980), and it was termed 'online computer-controlled testing'. Considerable development of the method is due to Mahin and co-workers (Mahin and Shing (1985); Mahin (1987)) at UC Berkeley, followed by the work of Shing et al. (1988, 1990) on multi-axial testing at Boulder, Colorado.

Its first application in European research was carried out by Elnashai and co-workers (Elnashai et al., 1990, 1991) where the behaviour of composite beam-column connections was investigated. More recently, a series of full-scale experiments have been carried out at ELSA, which is currently the largest European installation for pseudo-dynamic testing (Donea et al., 1996).

The method became widely used in the early-to-mid nineties and presents a number of advantageous features for earthquake testing application. Most notoriously, the inertia masses are accounted for analytically, thus avoiding the need to perform the test in real time, which in turn reduces quite considerably the power requirements for the actuators, comparatively to dynamic testing. This enables the use of large or full-scale models, thus eliminating the uncertainties and inconsistencies associated with the testing of small-scale specimens using non-standard materials. Therefore, in the absence of high strain-rate effects, which may introduce important changes in the response of the structure (Shing and Mahin, 1988; Paulson and Abrams, 1990), pseudo-dynamic testing provides a valuable tool for testing of full-scale structures.

The procedure is schematically represented in Figure 1. Detailed description of the testing methodology and discussion of additional features, such as sub-structuring and control and integration algorithms, are beyond the scope of the present work. For further information, the reader is referred to the aforementioned publications describing previous research on this topic.

The present work describes the pseudo-dynamic testing of a full-scale reinforced concrete frame, carried out at the ELSA laboratory at JRC (Ispra, Italy) under the auspices of the European research network ICONS (Innovative Concepts for Seismic Design of New and Existing Structures). The frame has four storeys and three bays and was intended to represent typical design and construction practice in many southern European countries in the 1950's and 60's. Within this context, the design of the frame, carried out by Carvalho et al. (1999), did not consider modern concepts such as capacity-design or ductility detailing.

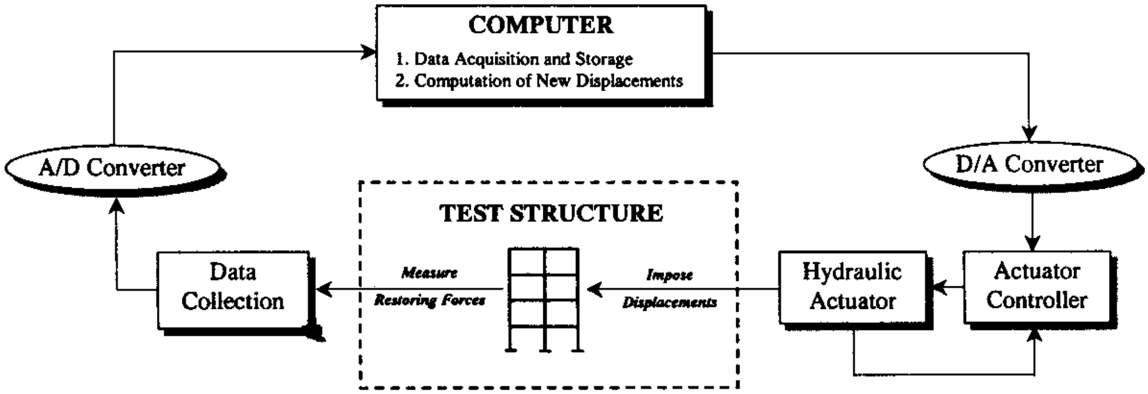


Fig. 1 Schematic representation of pseudo-dynamic testing (adapted from Elnashai et al. (1990))

The preliminary analytical study, carried out to determine the appropriate input level for testing the frame, is also discussed in subsequent sections. The input motion consisted of artificial accelerograms developed at JRC (Campos-Costa and Pinto, 1999) and calibrated to European seismological characteristics. Several time-histories, with varying energy content according to the return period of the associated event, were considered in the analyses which were carried out by using the computer code ADAPTIC (Izzuddin and Elnashai, 1989).

An overview of the construction process of the specimens, undertaken under the supervision of the staff at ELSA (Pinto et al., 1999), is also included together with the set-up of the experimental rig, instrumentation plan, data acquisition and test control.

It is important to note that the main objective of the experimental programme described in the present paper was to validate the application of selective intervention techniques, developed by Elnashai and co-workers (Elnashai, 1993; Elnashai and Pinho, 1998; Elnashai et al., 2000), for repair and strengthening of RC buildings. Such goal was indeed fully accomplished, as shown by the test results described in the work by Pinho et al. (2000). Discussion of the latter, however, is beyond the scope of the present paper where focus is on the experimental method rather than on the outcome of the testing programme.

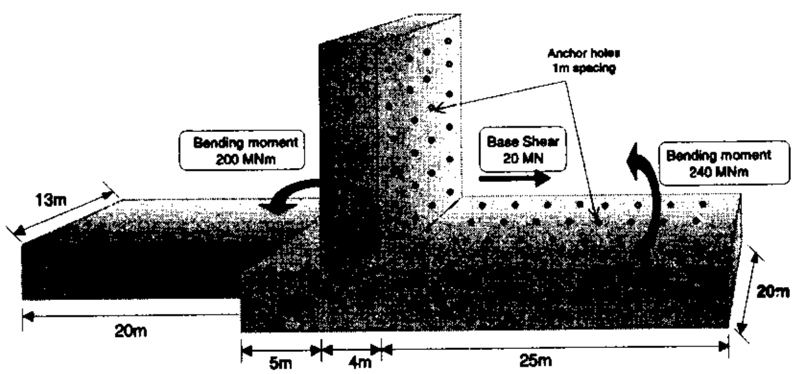


Fig. 2 Reaction-wall at the ELSA laboratory (adapted from JRC-ELSA (1990)

IMPLEMENTATION OF THE PSEUDO-DYNAMIC TESTING METHOD AT ELSA

In Figure 2, the geometric and loading characteristics of the reaction wall at the ELSA laboratory are depicted. The reaction wall is 20 m long by 16 m high and has a 200 MN-m bending capacity together with 20 MN horizontal shear resistance. The strong floor is 25 m long by 20 m wide and has a 240 MN-m bending capacity. Fixing points are placed in a square mesh of 1.0 m and feature an anchoring capacity of up to 500 kN. The actuators have capacities of 0.5 to 1.0 MN with strokes ranging from 0.25 to 1.0 m.

The loading pistons are fixed to the reinforced concrete slab via two stiff steel mounting cleaves, both of which are securely fastened to the slab by means of embedded prestressed bars. All the actuators are equipped with on-board displacement transducers that are used in the preliminary stages to guide the pistons to their respective anchorage points before the structure is loaded. The forces are measured by load cells mounted at the end of the piston rod, just before the swivel joint at the cleave-end part of the piston (Negro et al., 1994).

Once the loading assembly is set-up, the actuators become controlled by an optical digital transducer (one per piston) that measures the relative displacement between each storey and a steel reference-frame mounted on the reaction floor (Figure 3). It is also with such devices that the structural response displacements are measured and introduced in the pseudo-dynamic algorithm. The resolution of these digital transducers is of the order of 0.004 mm, independently of the stroke length of the actuator (Negro et al., 1994). Further, these interface with the control unit without the need for analogue to digital conversion or analogue conditioning, thus providing fast and accurate acquisition (Pinto, 1998).

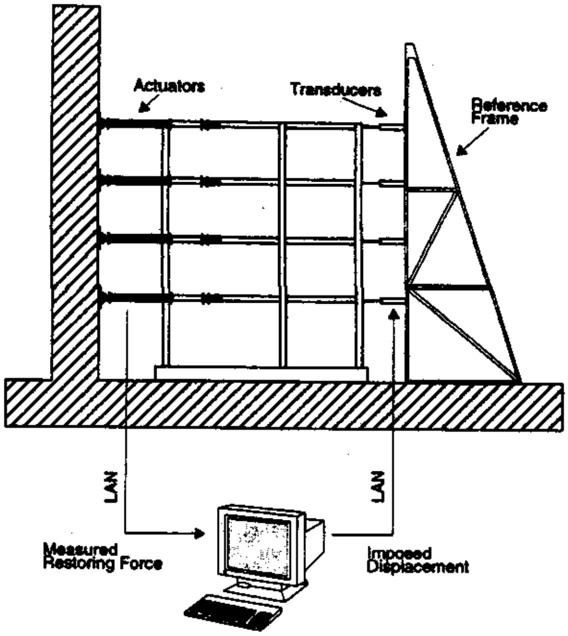


Fig. 3 Test set-up at ELSA laboratory (Negro et al., 1994)

All controllers are connected via an optical fibre local area network (LAN) to the master PSD computer, where the pseudo-dynamic algorithm is implemented and the dynamic equations of motion are solved at every time step. The system has been developed in-house; thus, any required modification and/or upgrading can be implemented with relative ease.

The model being tested is not too stiff nor does it have a great number of degrees-of-freedom (only four are being considered); thus the time-step required to guarantee numerical stability when using an explicit integration algorithm is within the range of values considered as feasible. Therefore, the central difference explicit scheme was deemed as adequate for the current application, with a time-step of $\Delta t = 0.005\,\mathrm{s}$ being adopted, and the use of more complex implicit schemes was avoided. In addition, the equivalent viscous damping was set to zero, since the response of the physically tested models already accounts for the hysteretic behaviour of the structure. The latter constitutes the main source of energy dissipation of RC structures under earthquake loading.

For further information, the reader is referred to the work by Donea et al. (1996), where a thorough review of the capabilities of the ELSA laboratory and its pseudo-dynamic implementation is given.

DESIGN AND CONSTRUCTION OF FULL-SCALE MODELS

1. General

Two separate frames were constructed at JRC for the purpose of this experimental programme. Although the two specimens were identical in terms of structural configuration and detailing, only one was constructed with infill panels. This made possible the evaluation of the effect of infills on the response of such structures through comparison with the results obtained for the "bare frame". Both infilled and bare frames weré tested at ELSA and then assessed and retrofitted using distinct approaches. In the present work, only the bare frame, where selective techniques were applied, is considered.

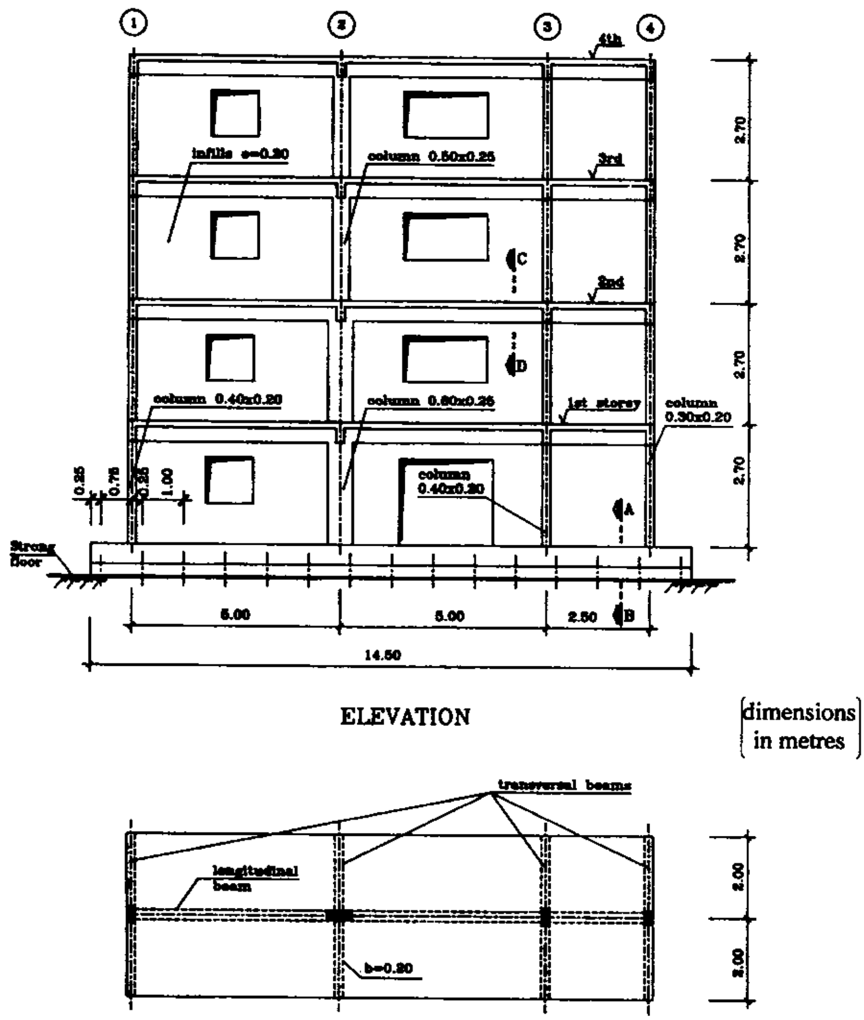


Fig. 4 Elevation and plan views of infilled frame (Carvalho et al., 1999)

2. Structural Configuration, Material and Loading Characteristics

The structure is a four-storey RC building with three bays, as depicted in Figure 4 and essentially designed for gravity loads by Carvalho et al. (1999). The vertical loads considered in the design consisted of the self-weight of the slab and transverse beams, finishings, infill walls and the quasi-permanent static load.

A typical two-way slab system was adopted, with 5 m transverse spans, 150 mm thick slabs and 500 mm deep beams throughout the building. The columns have the characteristics indicated in Figure 5 where it is noticeable that only column 2 is working in its stronger axis, as a result of the non-seismic design philosophy adopted. Consequently, this member plays a dominant role in the response of the frame and is hereafter referred to as "strong" column. The lap splice detailing, partially shown in Figure 5, also strongly influences the response of the structure since it increases further the strength differential between the second and third storeys. Further detailing of beams, slab and foundation can be found in Carvalho et al. (1999).

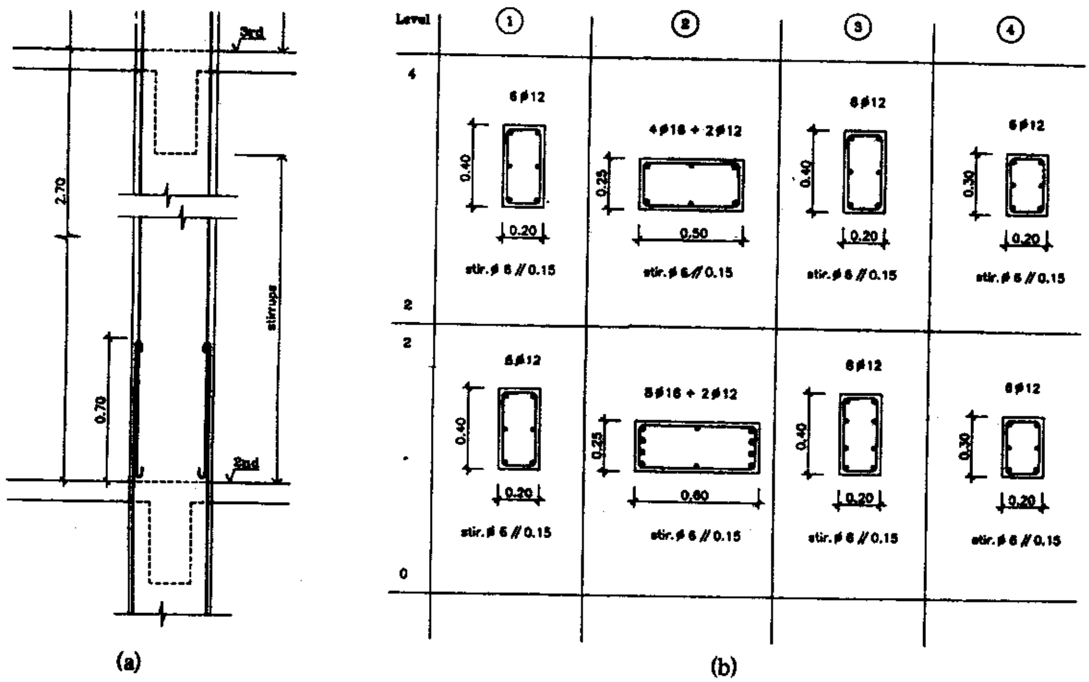


Fig. 5 Column reinforcement: (a) lap-splicing detail (b) cross-section characteristics

As for the materials, Class C16/20 concrete (CEN, 1991) was specified together with Fe B22k steel (1980 Italian Standards). The latter refers to smooth bars with a yield stress of 235 MPa and ultimate strength of 365 MPa. The bricks were of horizontally perforated type (with approximately 50% voids) with a thickness of 15 cm (40×20×15 cm^3).

3. Construction and Actual Material Characteristics

In Figure 6, a general view of the construction process is shown, together with photographs depicting interior joint detailing and lap-slice arrangements at the base of the wide column. Both details are of relevance to the outcome of the experiments, as shown later.

After completion of the construction work, the two frames were lifted 15 cm off the ground using 16 hydraulic jacks, each with a capacity of 400 kN. Roller bars, made of polythene plastic tubes, were then placed underneath the specimen, after which this was pulled to the test location inside the lab (Figure 7). It is noteworthy that the plastic tubes system were designed so that their material was stressed at a level very close to yield point. In this manner, any floor irregularities could be 'absorbed' through plastic deformation of the tubes, without causing significant deformations in the models.

In order to assess the actual characteristics of the materials employed in the construction of the model, several tests were carried out on concrete cubes and steel bar samples. For each floor level, four cube specimens were prepared during casting of columns or beams and slabs. A total of 36 concrete samples were tested under uniaxial compressive load. In Table 1, the mean values obtained for each level are given, showing considerable scatter of results. Moreover, the average concrete strength of 16.3 MPa is also significantly lower than the expected mean value of 24 specified at the design stage. This is not considered disadvantageous since such features are commonplace in southern Europe, thus rendering the test realistic.

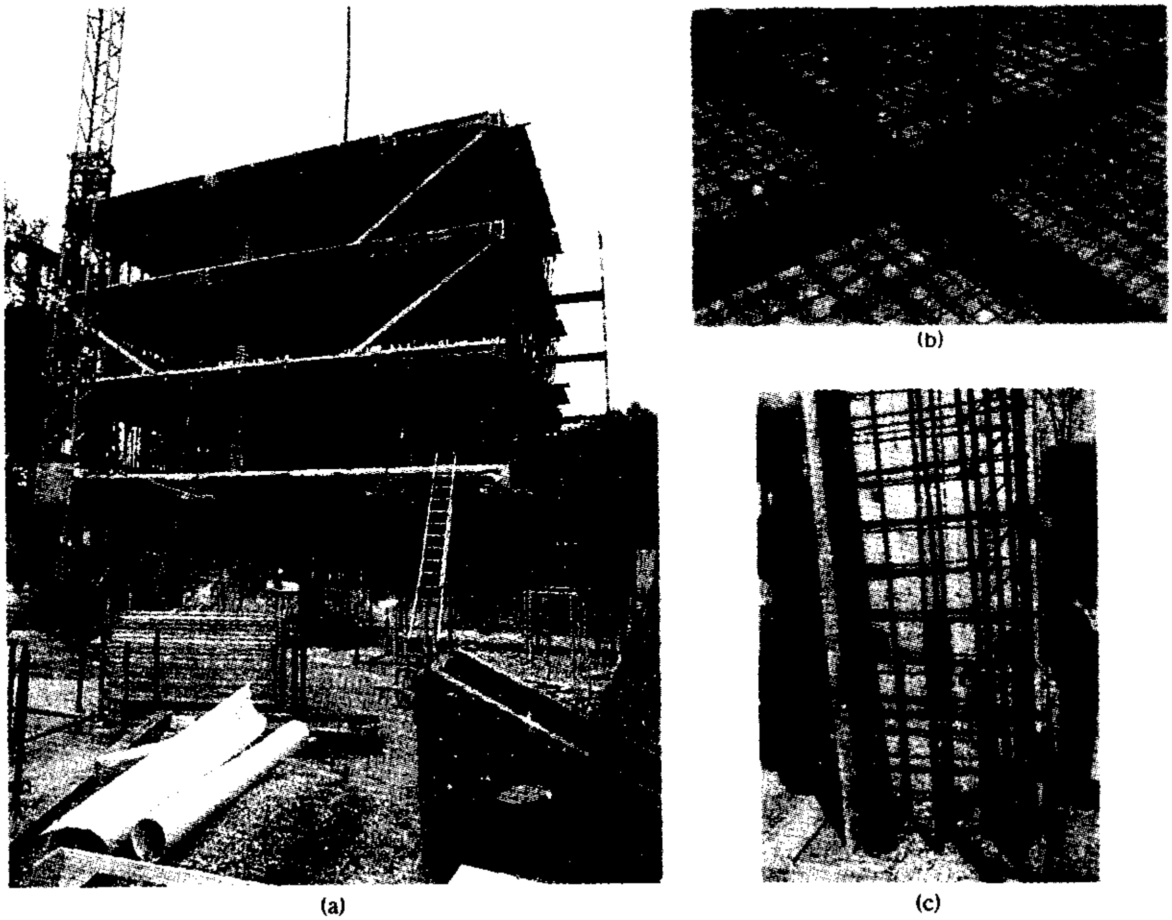
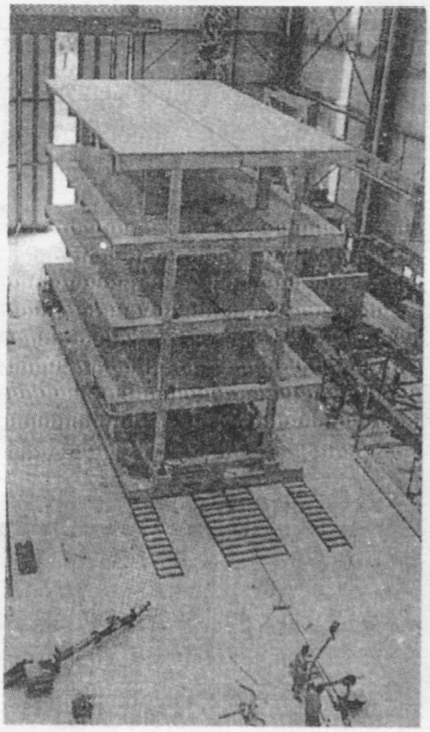


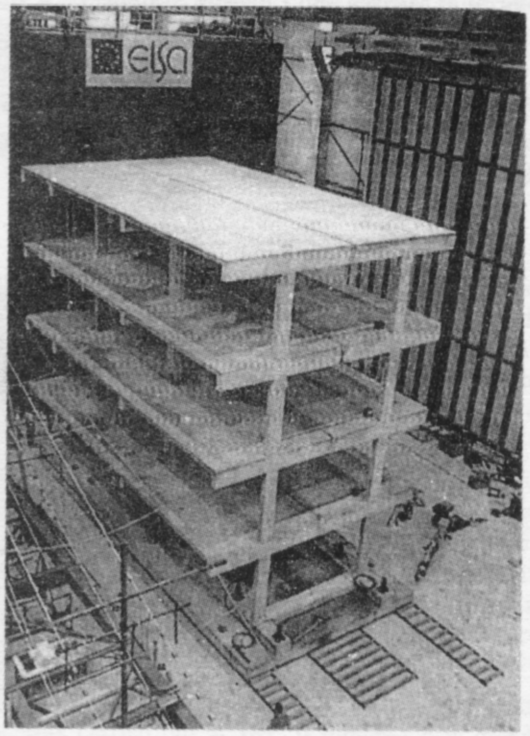
Fig. 6 Construction of models: (a) general view (b) interior joint (c) lap-splicing detail

Table 1: Mean Cube Compressive Strength of Concrete

Location		f_{cm} (MPa)
Level 4	- beams	17.0
	- columns	13.6
Level 3	- beams	21.6
	- columns	16.5
Level 2	- beams	18.1
	- columns	13.8
Level 1	- beams	13.0
	- columns	16.7
Foundation		31.8



(a)



(c)

Fig. 7 Transport of frame models: (a) frames being moved (b) frames in fixed position

With regard to the reinforcement steel, the situation was quite the opposite since the steel manufacturers were not able to provide the "weak" steel specified at the design stage. This is illustrated by the results from tensile tests undertaken for different diameters representative of those used in the specimens. The mean yield stress obtained for the 12 and 16 mm diameter bars was 343 MPa (235 MPa specified) and the mean tensile strength was 459 MPa (365 MPa specified), as shown in Figure 8.

Unlike the case of concrete, the higher values obtained for steel strength render the structures less representative of typical 1950's construction, since lower grade reinforcement was used then. Nevertheless, the shortcomings resulting from the design process such as absence of controlled failure mechanism, lack of ductile detailing and irregular strength distribution are still present. The usefulness of the experimental results therefore remains unchanged.

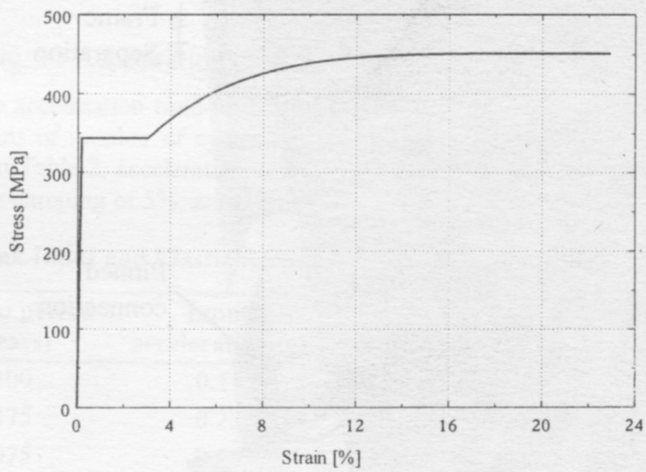


Fig. 8 Mean stress-strain diagram for reinforcement steel

4. Additional Masses and Loading System

Although pseudo-dynamic testing does not require physical representation of inertia masses, which are analytically simulated, the gravity loads associated to such mass still need to be included in the model, in order to guarantee a level of member stressing similar to that of the prototype. This is particularly important for the case of columns, where axial loading plays an important role in terms of both strength and deformation capacity.

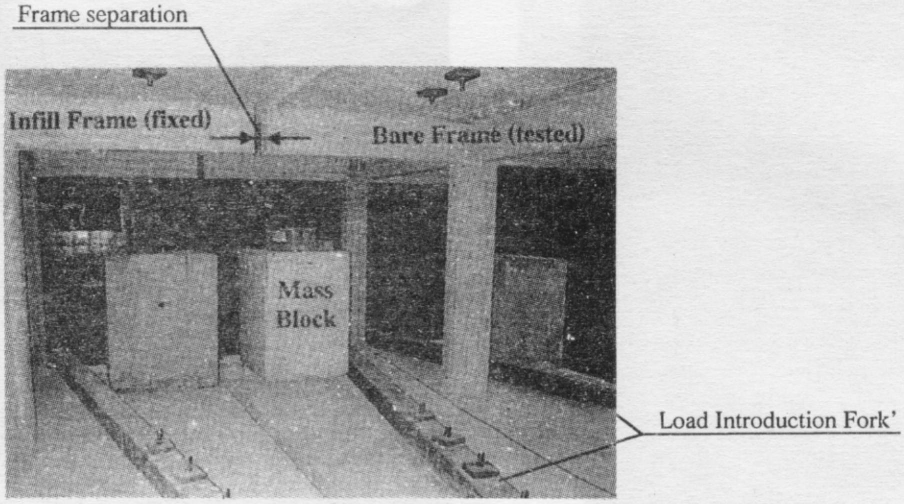


Fig. 9 Additional masses and 'loading forks'

Therefore, additional masses are used to simulate live loads, infills and finishings, and are placed at each floor by means of concrete blocks (Figure 9), large water containers and sand bags. Their distribution was intended to represent as closely as possible loading in beams and columns in a real application. The four actuators, which apply displacements at each floor level, are shown in Figure 10. The loads are transmitted to the slab by means of 6.3 m long steel "forks", which are bolted to the floor (Figure 9) at ten locations to spread the loads as evenly as possible and to avoid large localised stresses.

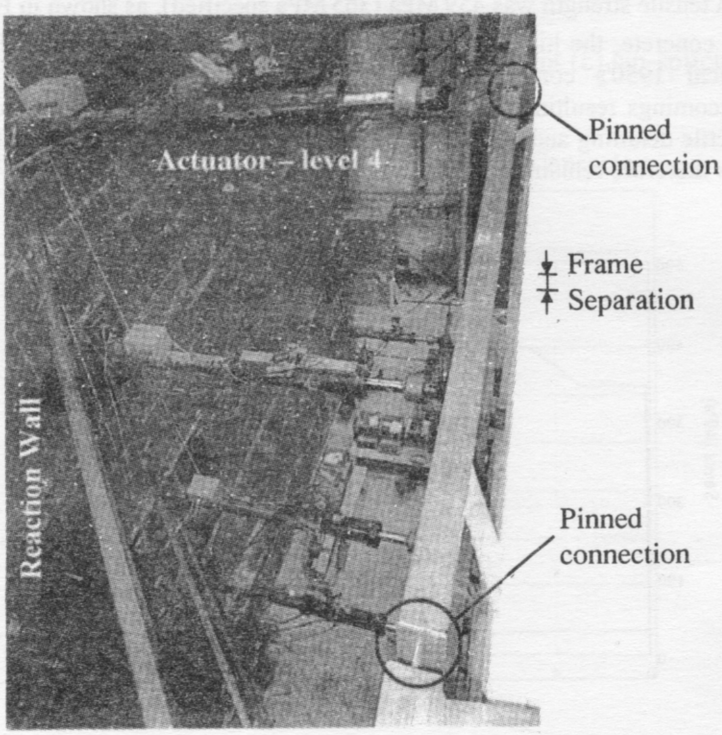


Fig. 10 Actuators and transverse connecting beams

Out-of-plane deformation is controlled through a system of pinned steel transverse beams that connect, at each floor level, the bare frame (being tested) to the infilled frame (fixed to the reaction wall and external steel truss). These connecting steel members allow the bare frame to deform in its own plane, as intended, but prevent any out-of-plane deformation to avoid the occurrence of transverse instability. The 'pins' are physically introduced by imposing a significant cross-section reduction at both ends of the beams (Figure 10), thus effectively reducing the flexure capacity to values very close to nil.

INPUT MOTION

Recent developments in earthquake engineering (SEAOC, 1995; ATC, 1997) have emphasised the need for performance-based assessment of structures. Evaluation of the response characteristics of reinforced concrete frames should thus be carried out for different limit states, under which a structure has to meet the "Operational", "Life Safety" and "Collapse Prevention" performance levels. The latter are usually checked in terms of both local (e.g., curvature, chord rotations) and global (e.g., storey drifts) deformation requirements.

Such deformation demand cannot, however, be explicitly imposed on the various members of the structure, since the dynamic response of the latter is not known in advance. Hence, the alternative becomes the use of different input motions, associated to each of the aforementioned performance limit states, as derived for a pre-determined probability of exceedance (or return period). According to VISION-2000 (SEAOC, 1995) and FEMA-273 (ATC, 1997), such return periods should be taken as 75, 475 and 975 (or 2000) years, corresponding to the "Occasional", "Rare" and "Very Rare" events, respectively.

For the purpose of the current experimental programme, a probabilistic seismic hazard analysis, calibrated to European seismological characteristics, was carried out at JRC (Campos-Costa and Pinto, 1997). The severity class "Moderate-High", typical of southern European countries, was chosen and a set of hazard-consistent accelerograms was artificially generated to fit the uniform risk spectra for return periods of 100, 475, 975 and 2000 years, hereafter referred to as Acc-100, Acc-475, Acc-975 and Acc-2000.

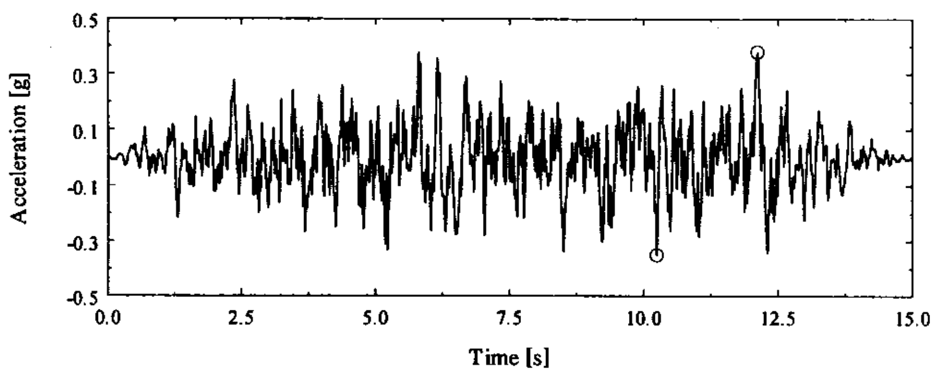


Fig. 11 Acceleration time-history for 2000 years return period

In Figure 11, the acceleration time-history of the 2000-years event is shown. The "richness" of the accelerogram, in terms of number of cycles and frequency content, is a common feature of artificially generated motions. In Table 2, acceleration and displacement elastic response peak values, computed for an equivalent viscous damping of 5%, are given.

Table 2: Peak Input and Elastic Response $(\xi = 5\%)$ Values for Each Accelerogram

Return period (years)	Input acceleration (g)	Response acceleration (g)	Response displacement (mm)
100	0.11	0.27	21
475	0.22	0.58	94
975	0.29	0.81	159
2000	0.38	1.08	258

PRELIMINARY ANALYTICAL ASSESSMENT

1. Objectives

The objectives of the preliminary analytical study were three-fold. Firstly, the seismic capacity of the frame needed to be assessed so that a decision regarding the level of input and sequence of testing could be made. Secondly, an estimate of the response characteristics and expected deformation mode of the building was also required for the definition of an optimised instrumentation plan, since measuring devices would be preferentially located in those zones where high deformation was to be expected. Finally, a preliminary decision on the most adequate retrofitting scheme was also necessary. With these objectives in mind, a finite element model was constructed for analysis by using the computer code ADAPTIC. Eigenvalue, inelastic push-over and dynamic analyses were carried out to assess the response characteristics of the intact frame and its global seismic capacity.

In what follows, analytical results relevant to the choice of input level and instrumentation are briefly reviewed. Work related to the choice of retrofitting scheme, together with a detailed description of the analytical study, is included elsewhere (Pinho et al., 2000).

2. Testing Sequence

As discussed above, assessment of the seismic response characteristics of structures should be carried out for different performance levels. This requires the employment of at least three models, so that each specimen can be independently tested under the "Occasional", "Rare" and "Very Rare" hazard levels. However, the financial costs associated to the construction, instrumentation and testing of a single full-scale multi-storey frame model, such as the one employed in the current experimental programme, are already quite significant. Thus, the use of several models for multi-level testing is rendered problematic, if not prohibitive.

To overcome such difficulties, a single model may be subjected to several input motions, each of which corresponding to the different limit states. However, in this manner, only the first test-run will correspond to a true assessment of the structure performance characteristics. All subsequent testing stages will make use of a previously damaged model, which is unrepresentative of the original intact structure.

To arrive at an optimum solution, which makes the best possible use of such expensive models, it is necessary to carefully identify which of the limit states under consideration will benefit the most from verification against the original intact model. In other words, if testing of the model under a particular input motion, associated to a particular event, will return little additional information on the response of the model (predicted through numerical modelling) at the expense of non-negligible dynamic properties changes, then such run should be avoided. This, for instance, would typically be the case of the "Operational" limit state.

On the other hand, if it is foreseen that an accelerogram is likely to cause important damage in the structure and that evaluation of the associated performance targets are highly dependent on the initial properties of the members, then this should be considered as a prime candidate for the initial run. Normally, this would be the case for the "Life Safety" limit state. Finally, if the input is so strong that major damage and member failure are likely to occur, independently of the initial state of the structure, then running such accelerogram even after the structure had already incurred some damage should not constitute a major hindrance. This is the expected behaviour when checking the "Collapse Prevention" performance scenario.

All the above assumptions need, nonetheless, to be first considered and carefully studied through numerical analysis, before final decisions on the test sequence are taken. Therefore, non-linear dynamic analyses of the intact frame model, under the four accelerograms mentioned earlier, i.e. Acc-100, Acc-475, Acc-975 and Acc-2000, were carried out. A summary of the results obtained from this analytical work is shown in Table 3. This includes the dominant inelastic period of vibration and peak values for base-shear, storey drift, curvature ductility and shear demand versus supply ratio in every column. In addition, qualitative comments describing the overall state of the structure are included. For succinctness, results from the study of the retrofitted frame are not included here.

Table 3: Summary of Results of Dynamic Analyses on the Original Frame

Return Period	Inelastic Period (s)	Max. Base Shear (kN)	Max. Drift (%)	Max. μ_ϕ (column)	Max. Column Shear (demand/supply)	Comments
100	0.9	141	0.25 3rd storey	0.66 column 1 level 4	0.37 column 2 1st storey	No Damage
475	1.1	269	0.9 3rd storey	2.7 column 2 level 3	0.75 column 2 1st storey	Light Damage
975	1.4	347	1.5 3rd storey	12.5 column 2 level 0	1.3 column 2 1st storey	Moderate / Heavy Damage
2000	1.9	344	2.8 3rd storey	23 column 2 level 3	1.5 column 2 1st storey	Storey Collapse

Taking stock of the analytical results summarised in Table 3, the following is observed regarding the effects that each input motion has on the response of the structure:

- Acc-100 does not induce any significant damage in the structure. However, the transient fundamental period of vibration (0.90 seconds) shows an increase of the order of 40% when compared to the initial elastic value (0.64 seconds), thus indicating some change in the dynamic characteristics of the frame, mainly due to cracking in members. Therefore, and in line with the comments made above, the utilisation of this time-history in the tests was deemed unnecessary.
- Under Acc-475, the frame also seems to behave in a satisfactory manner. However, localised hinging is recorded and repairable damage is likely to be introduced, though the structure as a whole does not face risk of collapse. Thus, it was envisaged that the structure should be first subjected to this input motion.
- For the Acc-975 record, high levels of damage are likely to be imposed on the "strong" column at the first and third storeys, due to its large deformation demand. Both the peak storey drift and curvature ductility are excessive in view of the non-ductile nature of the columns. In addition, the risk of brittle column failure due to shear is apparent, with the demand exceeding the supply on a number of occasions. The above, coupled with the fact that the seismic resistance of the frame is almost entirely dependent on the capacity of this member, increases the probability of storey or even global structural collapse. Hence, it was concluded that this record should constitute the second, and probably last, input motion for the test of the intact frame.
- Acc-2000 leads the frame to story collapse at the third level and to shear failure at the ground level. All the deformation indicators included in Table 3 reach values that cannot be accommodated by non-ductile reinforced concrete members. Therefore, this time-history was adopted only for the testing of the upgraded structure, to avoid a hazardous collapse scenario of the original intact model.

As mentioned earlier, the present experimental programme was carried out under the auspices of the European research network, ICONS. Therefore, other research groups, which are also members of the network, carried out parallel preliminary evaluation studies of the two frames (Carvalho et al., 1999; Panagiotakos and Fardis, 1999; Varum and Pinto, 1999; Monti and Pinto, 1999). In general, all of these independent investigations were in agreement with the results summarised above. Therefore, the testing sequence described in Table 4 was confirmed.

It is noteworthy that, for the sake of simplification, the analytical studies were carried out assuming independent analysis for each accelerogram. However, this is not the case during testing application; thus, higher level of deformation is expected due to the accumulation of damage and period elongation. For this reason, the use of Acc-2000 with the retrofitted frame was deemed dependent on the experimental response of the model under the previous input motions.

Table 4: Testing Sequence

Ref.	Frame	Input motion	Objectives
IF-475	Intact	Acc-475	Assess frame under "Rare" event (0.22g)
IF-975	Intact	Acc-975	Assess frame under "Very Rare" event (0.29g)
RF-475	Retrofitted	Acc-475	Assess retrofitted frame under "Rare" event (0.22g)
RF-975	Retrofitted	Acc-975	Assess retrofitted frame under "Very Rare" event (0.29g)
RF-2000†	Retrofitted	Acc-2000	Assess retrofitted frame under "Very Rare" event (0.38g)

† conditional on response of the structure under the previous motions

However, had it been considered necessary to achieve a higher level of rigour in the prediction of the response of the models, the numerical analyses could have been rerun introducing all the accelerograms in succession so that the effects of damage accumulation could be accurately accounted for. Alternatively, recent methodologies proposed by Coelho et al. (1998), relating the vulnerability curves obtained for a particular structure subjected to increasing excitations, with the curves obtained from independent runs, could also have been used. However, due to the preliminary nature of this analytical work, such high level of accuracy was not considered as necessary, and no further studies were thus undertaken.

3. Instrumentation Plan

As previously mentioned, assessment of seismic response characteristics of existing structures relies heavily on the accurate evaluation of deformation response parameters, which are directly related to damage. Usually, such parameters are curvature (section level), chord rotations (member level), drift (storey level) and total top displacement (global level). Further, the significance of each of these parameters varies according to the limit state being assessed. For instance, curvature ductility plays a relatively secondary part in the verification of the "Operational" limit state, whilst being a very significant parameter in the evaluation of "Collapse Prevention" assessment scenarios.

Therefore, the instrumentation layout was deployed to achieve the acquisition of the response parameters pertinent to the limit states sought. Moreover, the design of instrumentation took stock of the following frame characteristics:

- Column 2 attracts a significant percentage of the horizontal load. This column possesses considerably higher stiffness and strength than other columns since it is the only one responding in its strong-axis direction, as described earlier. Consequently, high deformation demands are observed, particularly at the base, leading to early failure of the member, which is not designed to withstand such level of inelastic deformation. Therefore, a refined mesh of instruments is required to fully capture the response of this element.
- Maximum drifts and ductility demand occur at the third storey. This is expected due to the excessive strength differential (between the second and third storeys) caused by the abrupt changes in reinforcement detailing of the "strong" column, as mentioned earlier. It comes as a result of the "old design" approach where strength regularity with height was not a primary concern, thus leading to the formation of an undesirable soft-storey failure mechanism. Hence, it is necessary that, contrary to what would be expected in a regular structure, sufficient number of instruments are located at the third storey, where failure is likely to occur.
- Shear demand exceeds supply at the "strong" column, as a result of its high contribution to the horizontal resistance of the frame and insufficient member detailing. The possibility of shear type of failure is therefore high. Thus, instrumentation at this member should be configured such that it will allow the accurate evaluation of a shear, as opposed to flexure, deformation mode.

A total of 144 acquisition channels were set up for the purpose of the current experimental programme. Four optical transducers were used to measure displacements at each level, with the corresponding restoring forces being gauged by load cells located at the actuators. Rotations were measured at columns, beams and joints. This was achieved by means of 64 digital inclinometers following the distribution indicated in Figure 12. Such configuration provides important information regarding the response of individual structural members, such as distribution of inelasticity in the "strong" column, beam rotation and joint deformation, amongst others.

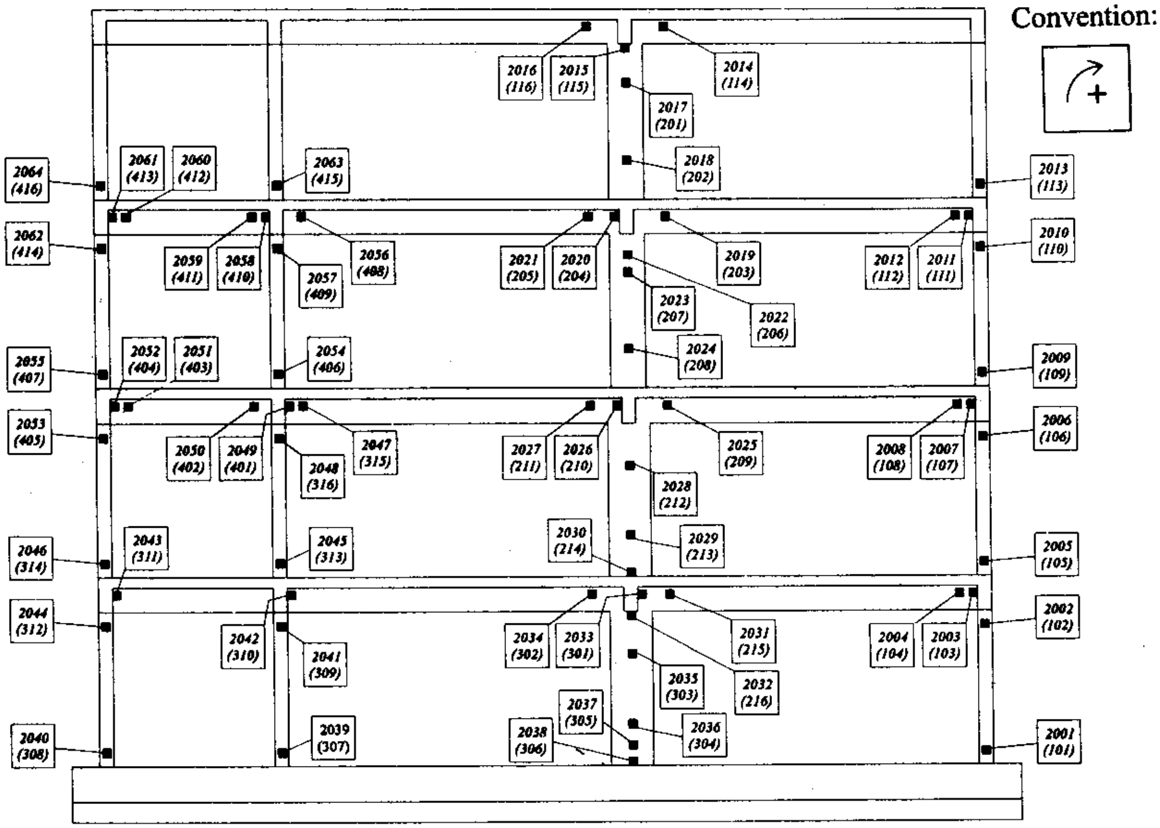


Fig. 12 Distribution of inclinometers

In addition, 27 relative displacement transducers are used at the first storey in the "strong" column where high levels of deformation are expected (particularly after retrofitting). As mentioned above, these aim at measuring both flexural and shear components of deformation in the column, as well as the behaviour of the beam-column joint and adjacent beams. Their distribution is shown in Figure 13.

Further, 26 transducers are used to measure longitudinal beam deformation at the first floor, from which section rotation, plastic hinge length and member elongation can be determined. Another 24 similar instruments are employed to measure slab deformation in critical areas. Detailed information on the distribution of all instruments is provided in the work by Pinto et al. (1999).

Finally, data input, acquisition control and monitoring of the test are accomplished by means of a cluster of 10 machines. This allows for real-time monitoring of the response of the frame during each test, thus giving the possibility of halting the test at the point of peak deformation for close inspection (or to prevent full collapse of heavily damaged models).

SUMMARY OF TEST RESULTS

Prior to the start of the pseudo-dynamic test programme, frequency evaluation was carried out. The structure was subjected to an impulse load force at the top storey, and the free-vibration displacement time-histories were measured for subsequent power spectral analysis. The latter yielded the values of 0.64, 0.21 and 0.13 seconds for the first, second and third flexural vibration modes, respectively. These results were practically identical to those obtained analytically (Pinho et al., 2000), and confirm the flexible nature of the "bare" four-storey model.

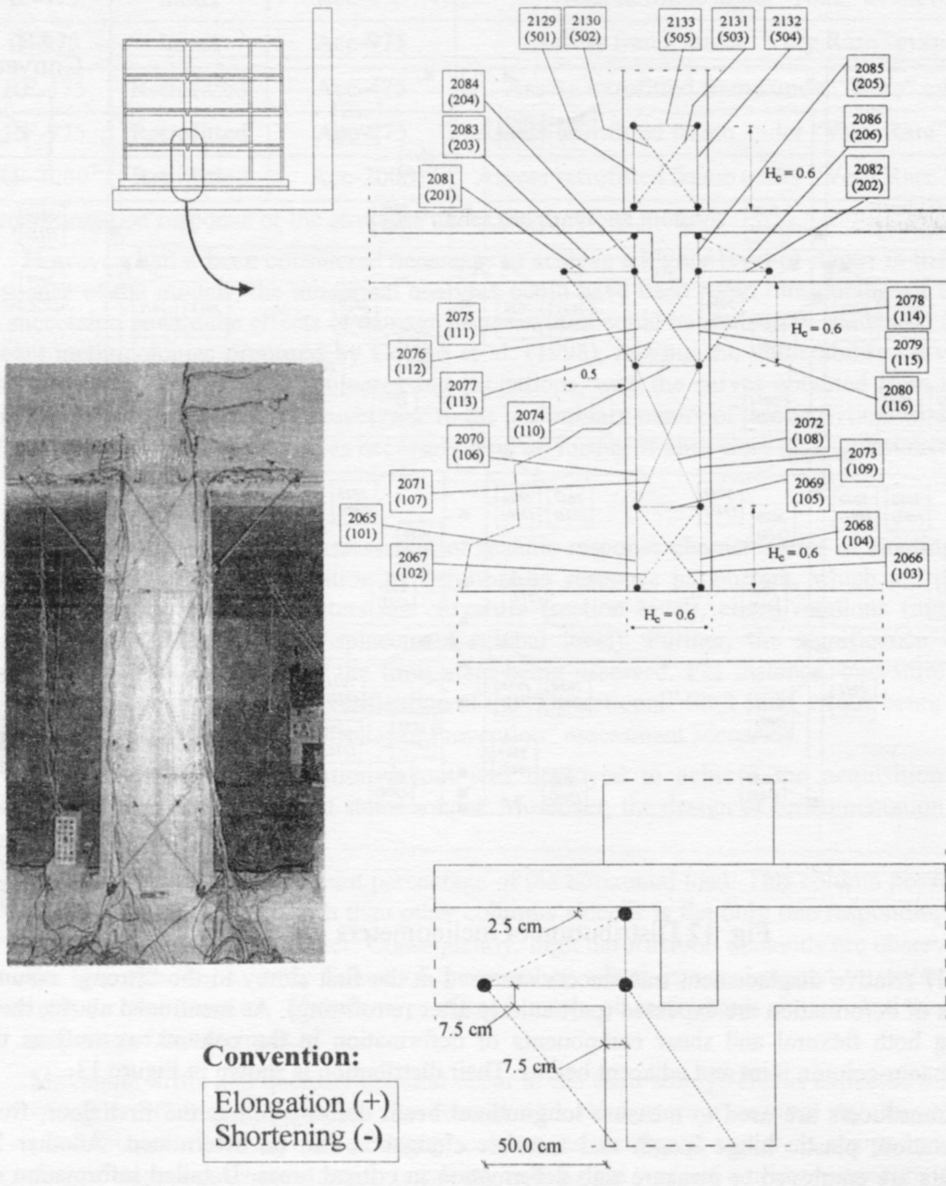


Fig. 13 Distribution of transducers in "strong" column

The structure was then tested under the Acc-475 accelerogram. No significant damage, such as full-width cracks or concrete spalling, was observed. As expected, minor cracking was indeed visible at the top and bottom of the majority of columns. The shear and drift distribution profiles, shown in Figure 14, also confirm the predicted soft-storey deformation mode, due to the significant irregularity in stiffness and strength distribution at the "strong" column. This is reiterated by the hysteretic curves, depicted in Figure 15, and energy dissipation plots, shown in Figure 16. These show that, contrary to what would be expected in a regular building, the first storey is not the main source of energy dissipation.

It is noteworthy that the structure did meet the "Life Safety" requirements associated with the level of input motion used. Such outcome is reassuring regarding the seismic behaviour of RC frames built in the 1950's in southern Europe. However, it is important to reiterate that the steel bars used in the tests were of higher grade than those typically used in the 50's, as pointed out earlier, thus contributing to an improvement in the response characteristics of the structure (this was confirmed during the preliminary numerical studies where initial analyses, carried out using the lower grade design steel, revealed lower values of seismic resistance for the model). Furthermore, other important issues, such as irregularity in plan, also typical of structures not designed according to modern codes, were equally not assessed.

The plots given in Figures 14 to 16 provide a sample of the type of output obtained in this experimental programme and refer only to the testing of the intact frame under the 475 years return period motion. A more exhaustive collection of results, covering all test stages, is included in the work by Pinho et al. (2000).

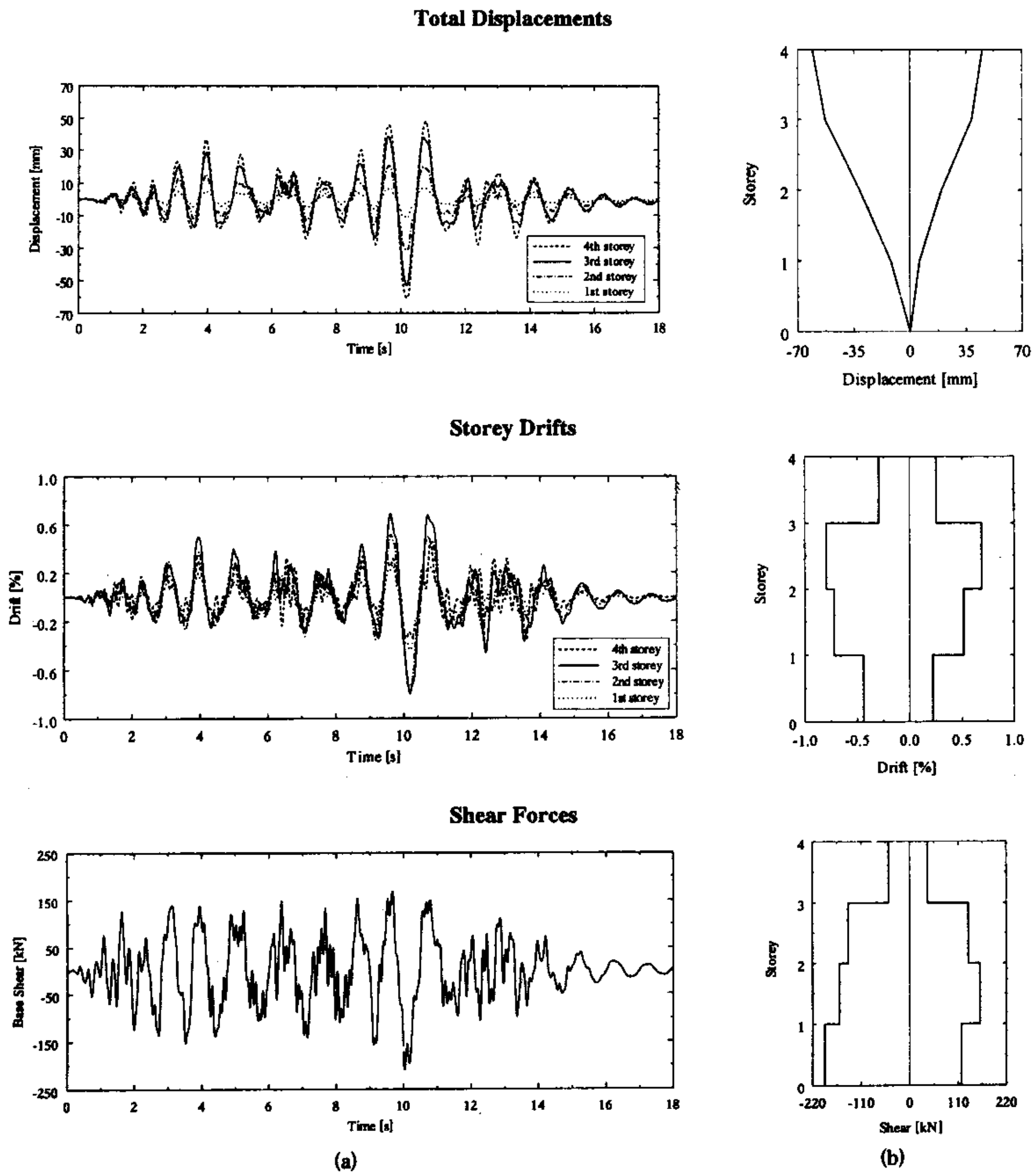


Fig. 14 Test results under the Acc-475: (a) time histories (b) distribution profiles

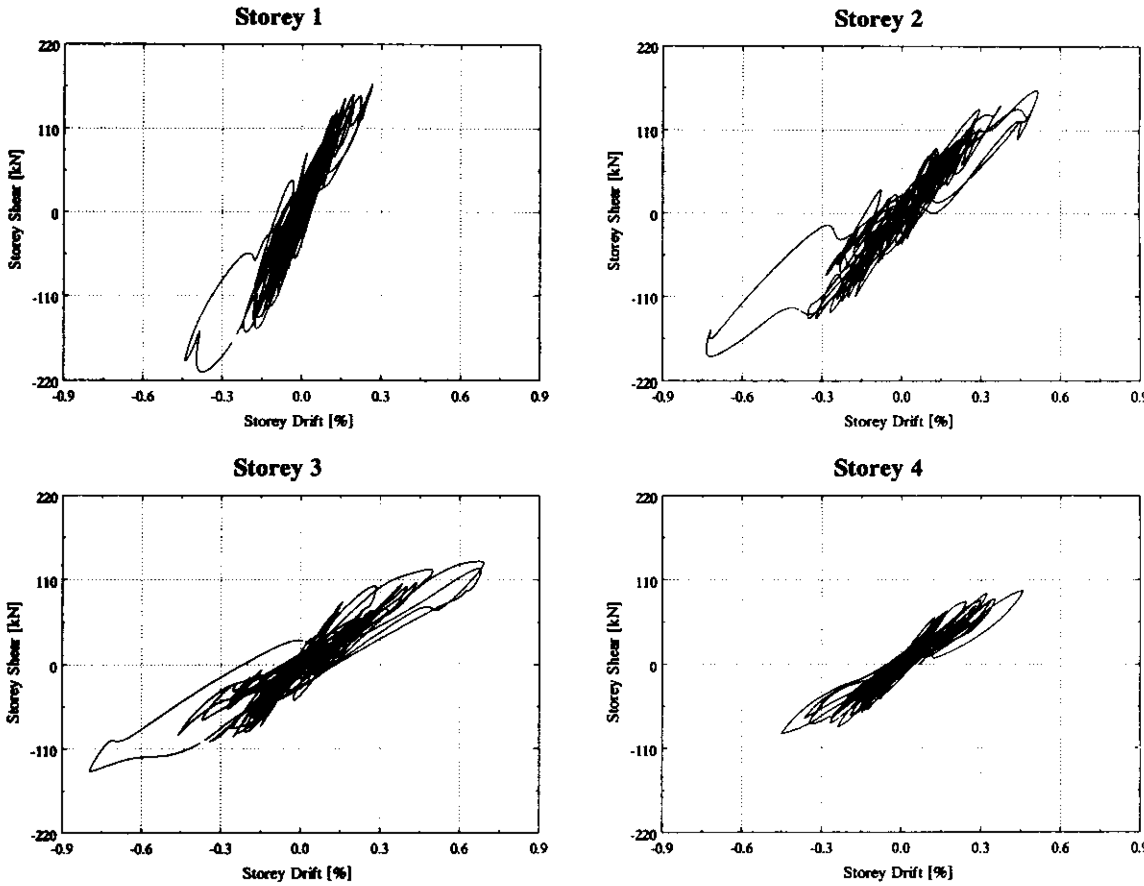


Fig. 15 Hysteretic curves and envelopes at each storey for Acc-475

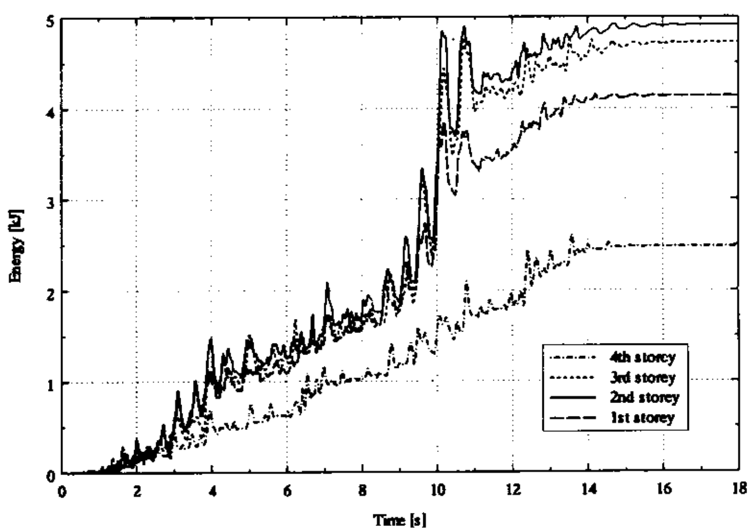


Fig. 16 Energy dissipation of each storey for Acc-475

Under the "Very Rare" event (Acc-975), heavy damage was expected, as indicated by the numerical work described earlier. This was indeed the case. The significant change in flexural capacity at level 3, coupled with the inadequate lap-splicing, induced the development of a soft-storey mechanism at that level, where the majority of energy dissipation takes place. This ultimately led to a scenario whereby storey collapse was eminent, at around 7 seconds. As observed in Figure 17, drift at the third storey registered a dramatic increase of up to 2.4%, accompanied by a drop in strength. Upon unloading, it was observed that a permanent drift ratio in excess of 1.0% was measured together with a significant stiffness degradation, thus leading to the conclusion that a state of severe damage had been reached. Such behaviour had all the characteristics of a brittle failure mode, and thus, to prevent a complete collapse situation, the test was terminated.

Further confirmation of the brittleness of this failure mode was given by the rotation time-history at the "strong" column, which shows a sudden shift in the location of hinging from the base of the column to 70 cm above, where the lap-splice finishes, at a time around 6.7 seconds (Pinho et al., 2000). Concrete spalling and crushing were recorded at the lap-splice location and at the top of the column. The mixed flexural-shear diagonal crack that originated at this point reached the full width of the column, thus causing the sudden element failure. Since this member constituted the main support for the storey, its failure inevitably led to failure of the whole storey.

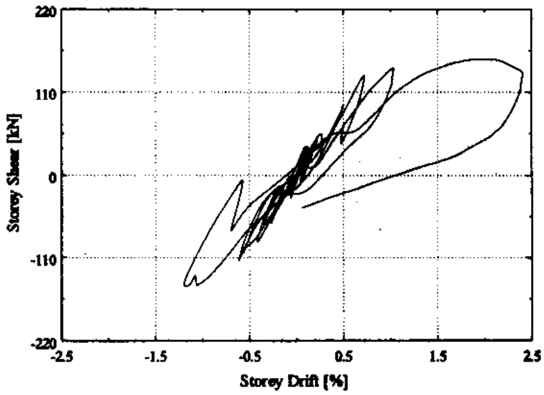


Fig. 17 Hysteretic curves and envelopes at level 3 for Acc-975

REPAIR AND UPGRADE OF DAMAGED FRAME

As mentioned earlier, the damaged frame was then repaired and upgraded using selective techniques (Pinho et al., 2000), after which it was re-tested under the Acc-475, Acc-975 and Acc-2000 records. Since the behaviour of the frame was controlled by the strong column, selective intervention targeted the upgrading of this important structural element. Retrofitting of other members would have been beneficial but not essential. Moreover, this would have contradicted the main goal of the work, which focused on application of simple and low cost retrofitting methods that would cause minimum disruption to the use of the building in a real application. Therefore, a minimalist selective intervention scheme, where only the "strong" column was upgraded whilst all other columns (also damaged) were left unchanged, was adopted. This was in clear contrast with conventional methods that are often over-conservative.

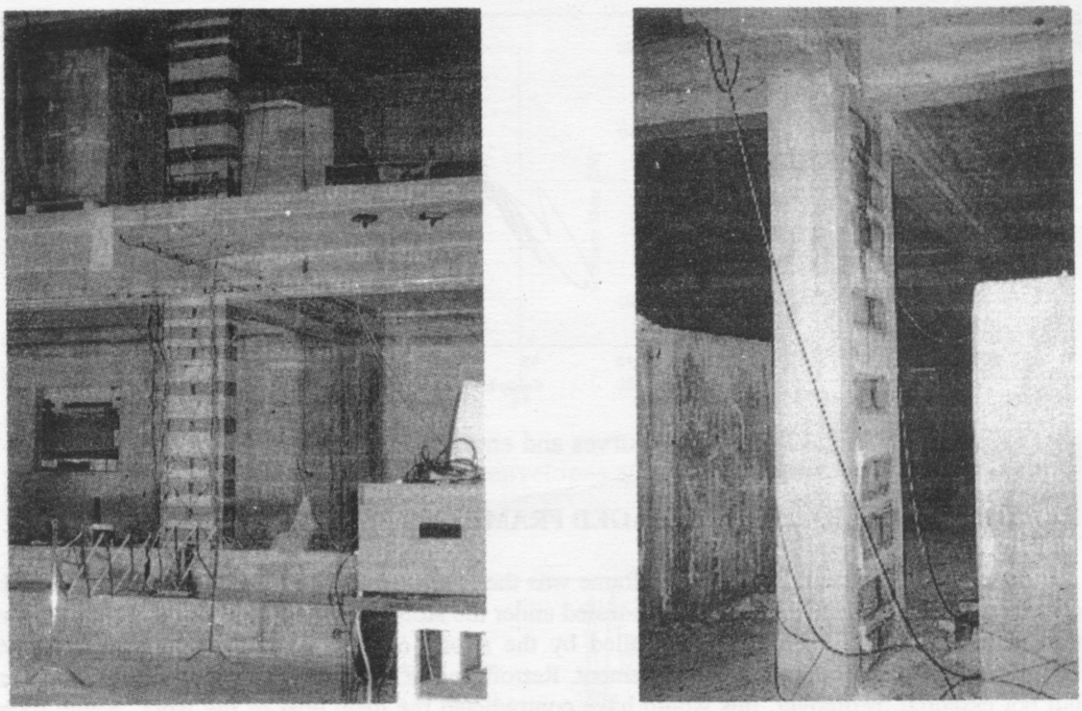
It is, however, important to note that any intervention on this column had also to minimise the increase of stiffness of the member since this would cause a rise in its already large share of the horizontal resistance of the building. Failure to achieve this would have increased deformation and shear demands to even higher levels, thus causing further deterioration of the already unsatisfactory seismic response of the structure.

Therefore, an intervention scheme comprising three distinct techniques, to tackle a similar number of major deficiencies in the structure, was adopted. The main objectives of this intervention were as follows:

- Elimination/reduction of strength irregularities: Addressing the abrupt change in the flexural capacity of the "strong" column at the third floor was of vital importance to avoid soft-storey failure mechanism at that level. This was accomplished by means of a strength-only intervention, using four 16 mm bars at the two top storeys located at a distance of 50 mm from the column edges. Steel grade S400 was considered.

- Improvement of column deformation capacity: The ductility demand in the "strong" column was very high, especially at the foundation and third-storey levels. Therefore, ductility-only intervention was envisaged, using steel collars over the expected plastic hinge length of the member. A minimum spacing between successive collars was provided to allow the spread of inelasticity. The plates were 3 mm thick by 120 mm long in grade Fe430 steel.
- Increasing shear capacity of column: To minimise the risk of brittle shear failure, likely at ground-floor level, a shear-strength only intervention was envisaged. This was achieved by means of extending the use of steel collars throughout the full height of the columns. For this situation, the spacing between the plates was larger since no ductility augment was sought.

A detailed description of the design and application of the selective retrofitting schemes described above is given elsewhere (Pinho et al., 2000). In Figure 18, the retrofitting schemes at the first, second and third storeys are shown.



(a) (b)

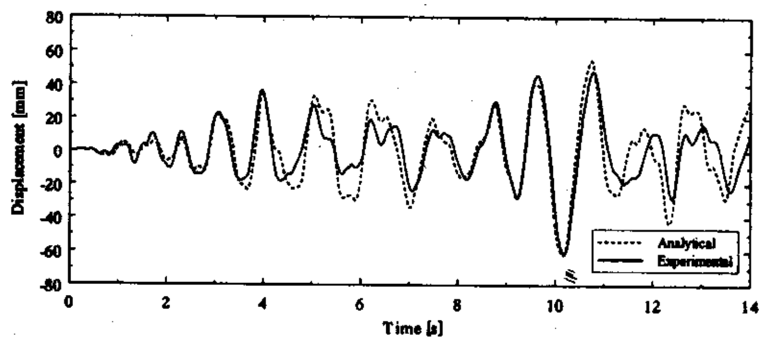
Fig. 18 Retrofitted members: (a) first/second storeys (b) third storey

All three tests carried out for the upgraded structure were successfully completed, as a result of the enhancement of the response characteristics of the structure. Since collapse was never imminent, the retrofitted structure fully complied with the "Collapse Prevention" performance target, in clear contrast to the response of the original frame where a failure condition was reached. This is attributed to the effectiveness of the selective scheme in changing the shear deformation mode of the intact structure to a more ductile flexure-dominated one, as shown in the report by Pinho et al. (2000). In addition, the deformation capacity was increased by improving the confinement supply in plastic hinge and lap-splice zones, and by increasing the shear capacity of column 2.

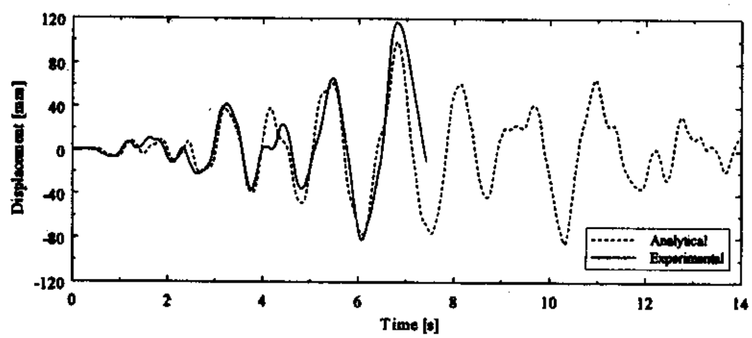
COMPARISON WITH ANALYTICAL PREDICTIONS

The costs associated to testing of such large-scale models necessitate minimisation of errors, such as using inadequate input motion or failing to instrument critical locations. Therefore, it is important that analytical tools provide reliable predictions of the behaviour of test specimens. The accuracy of the computer code ADAPTIC, which played a vital role in the planning of the current experimental programme, is briefly reviewed in this section.

Indeed, it was observed that the experimental response of the model did follow the behaviour numerically predicted prior to the test. Practically all features of the frame response, such as soft-storey mechanism, "strong" column deformation demand and damage distribution characteristics, were identified by the preliminary analytical studies. This is further emphasised by the plots shown in Figure 19, where very close agreement between analytically predicted and experimentally measured top displacement time-histories is observed.



(a)



(b)

Fig. 19 Analytical and experimental top frame displacement: (a) Acc-475 (b) Acc-975

However, some differences were observed in the prediction of peak drift values at the soft-storey and in the values of storey and total shear forces. This may be due to the fact that the analysis did not account for effects such as slippage of re-bars, shear distortion in plastic hinge regions and incomplete crack healing. It is believed that further refinement of the model to account for the aforementioned factors would lead to an even better duplication of the experiments. This, however, was beyond the scope of the current work, where analytical tools were used to steer the test programme.

CLOSING REMARKS

This paper has described the pseudo-dynamic testing of a full scale reinforced concrete frame. The four-storey structure was designed according to southern European practice typical of the 1950's and 60's. Since a priori knowledge of the behaviour of the model is absolutely essential, a preliminary analytical assessment of the frame was carried out to assess its response characteristics and assist in the test planning procedure.

A performance-based assessment methodology, whereby three performance limit states are considered, was adopted to evaluate the seismic response characteristics of the model. Since only one test specimen was available for the whole experimental programme, choice of testing sequence had to ensure an optimum balance between the level of damage inflicted on the structure by a particular accelerogram and its influence on the structural response in subsequent runs. Results from the preliminary numerical study, coupled with careful consideration of each limit state requirements, were considered in the selection of such test sequence, thus resulting in the use of artificial accelerograms representing earthquakes with return periods of 475 and 975 years, associated to "Rare" and "Very Rare" events, respectively (SEAOC, 1995).

The instrumentation layout was deployed to achieve the acquisition of the response parameters pertinent to the limit states sought. The latter require evaluation of total displacements, storey drifts, member chord rotations and section curvatures, all of which are good indicators of damage. Moreover, the design of instrumentation took also into account the observations from the analytical study, which accurately predicted the zones where higher deformations were to be expected, thus indicating the areas where higher density of instruments was desirable.

Finally, under the 475-years motion, minimal damage was observed in the structure, thus the structure complied with the "Life Safety" performance requirements as specified in the VISION 2000 reference. As for the 975-years accelerogram, the test was terminated before the end of the record was reached, due to failure at third storey; hence, the "Collapse Prevention" objective was not met.

ACKNOWLEDGEMENT

This research was funded by the European Commission through the training and mobility of researchers (TMR) programme and by the Calouste Gulbenkian Foundation. The former provided support to the experimental work carried out at the ELSA laboratory through the TMR Large-Scale Facilities Network - ECOEST II (European Consortium of Earthquake Shaking Tables). The analytical work, undertaken at Imperial College, was funded by another TMR network - ICONS (Innovative Concepts for Seismic Design of New and Existing Structures) and by the Calouste Gulbenkian Foundation. Such financial support and the co-operation of the co-ordinators and members of the two networks is gratefully acknowledged. The authors would also like to thank Dr. A. Pinto, Mr. H. Varum, Dr. J. Molina and Mr. G. Verzeletti who where in charge of the experimental programme at ELSA.

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