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SEISMIC TEST OF THE NEW LAWCOURT IN NAPLES

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ABSTRACT

The dynamic test of the New Lawcourt in Naples was carried out, in order to analyze its dynamic behaviour and to check its seismic reliability. The investigation regarded the skylight of the central portion, formed by steel frames supported by the concrete structure. The dynamic characterization tests allowed to identify the dynamic properties of the structure. The structural behaviour was quite complex. Frequencies associated both to the global modal shapes of the whole structure, often coupled, and to the local resonances, relative to single members, were pointed out. The seismic test showed a not negligible reduction of the first structural frequency when increasing the level of stress. This behaviour is to be related to the not negligible collaboration given by the non-structural components, when stresses are low.
The structural identification allowed to define the best finite element model, whose dynamic characteristics were very close to the experimental ones.

KEYWORDS

Seismic test; experimental modal analysis; spectral analysis; transfer function; dynamic behaviour; seismic design; system identification.

INTRODUCTION

The New Lawcourt in Naples is a group of buildings of about 1200000 m^3, where all the law activities of the city are carried out. It is composed of three blocks, which are separated by seismic joints. In Fig. 1 the structure of one of the three blocks is shown. The maximum size of the structure are 54*52 m in plan. Its height is 48 m. The carrying structure is formed by six concrete frames parallel to plane *xz*. Each frame is composed of a very rigid wall and four pillars. The walls are founded on piles and have variable thickness. The frames are spaced of about 11 m, and linked one to another by means of horizontal diaphragms made of precasted concrete. The steel structure of the skylight (thick lines in Fig. 1) stands up from the concrete structure. It is composed of six steel frames, which are made up of a pillar and two horizontal beams. The pillar is constrained with the concrete structure by means of a steel plate fixed at the pillar base. The upper horizontal beam is constrained to the concrete structure by means of a bolted joint. The lower beam, which is missing in the fourth frame, is fixed to the concrete structure. These frames support the properly called skylight, which is composed of HEA300 beams. The secondary structure is made up of longitudinal beams composed of 4L 120*12 steel girders and 4L 80*8 diagonal braces. The roofing are supported by means of HEA200 purlins, which are spaced of 2 m.

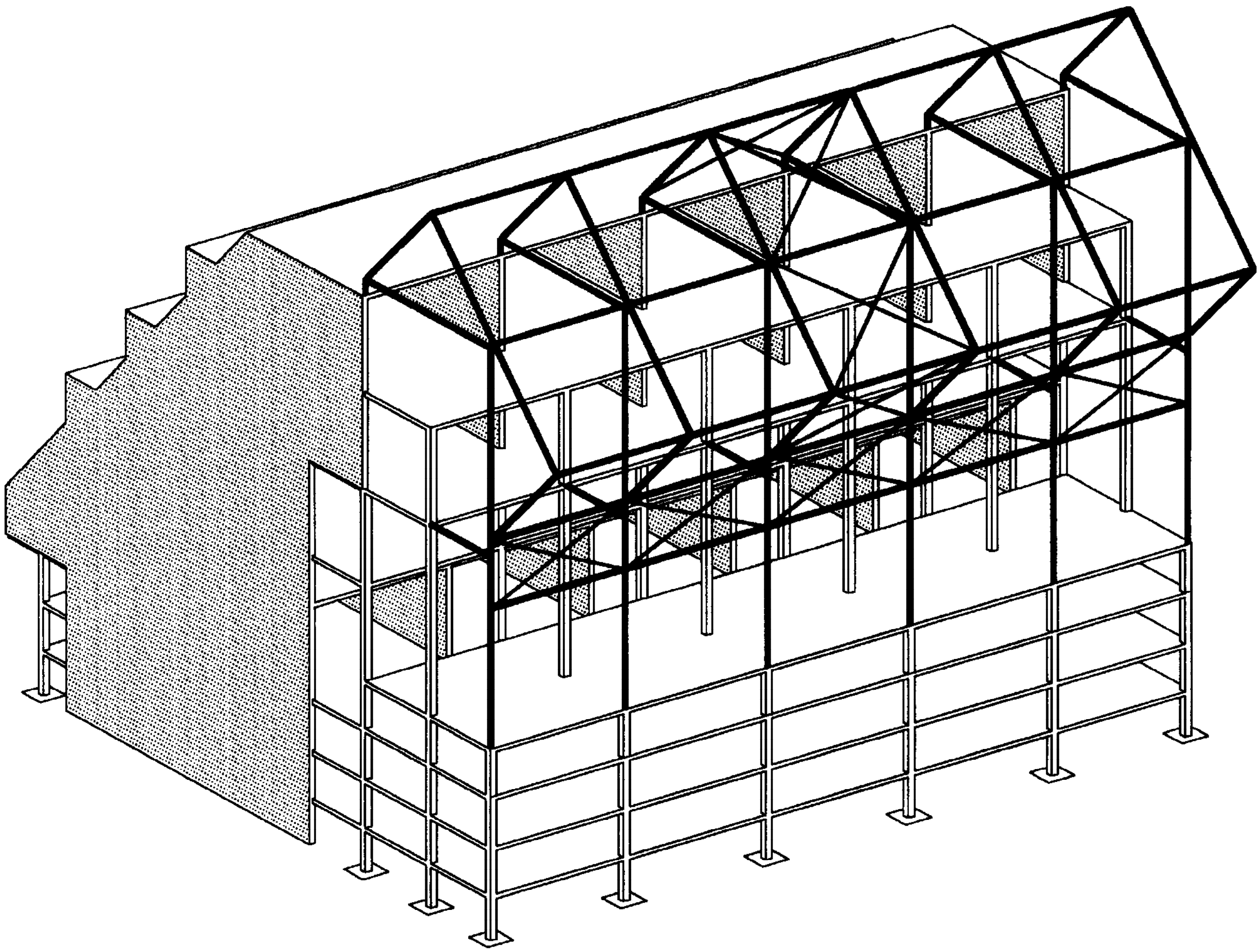

Fig. 1 Scheme of the tested building structure

The dynamic test of the structure was carried out, in order to analyse its dynamic behaviour and to check its seismic reliability. The investigation regarded the skylight of the central block. A finite element analysis was also performed.

INSTRUMENTATION AND TESTS

The instrumentation was done by using 27 accelerometers ENDEVCO 2265-25, deployed in two different configurations (Fig. 2). Three transducers of relative displacements (W50K) were located between two adjacent blocks. Two vibrodynes were used to test the building. The first one, whose weight was 3.8 kN, was able to produce a maximum force of 20 kN with a maximum frequency of 50 Hz. The second one, which was able to generate a 200 kN force with a maximum frequency of 20 Hz, weighted 38 kN. The vibrodyne was located at the pillar of frame D. Fig. 2 shows sensor locations in the two configurations. Frames A to F belong to the tested structure, frames A' and F' to the adjacent blocks. The vibrodyne location is also shown. The measurements were done by ISMES in April 1993 (ISMES, 1993).

The vibrodyne generated an horizontal sinusoidal forces

$$P(t) = p \cdot sin\left(2\pi ft + \varphi_p\right)$$

were f is the frequency of the testing force, and $p = C\, cos(\alpha/2)\, f^2$, C being a characteristic constant of the machine, and α the angle between the two masses in each disk. During each test the frequency varied slowly, so that the input could be supposed to be steady for each f. With good approximation the response in the i-th sensor was (Ewins 1984)

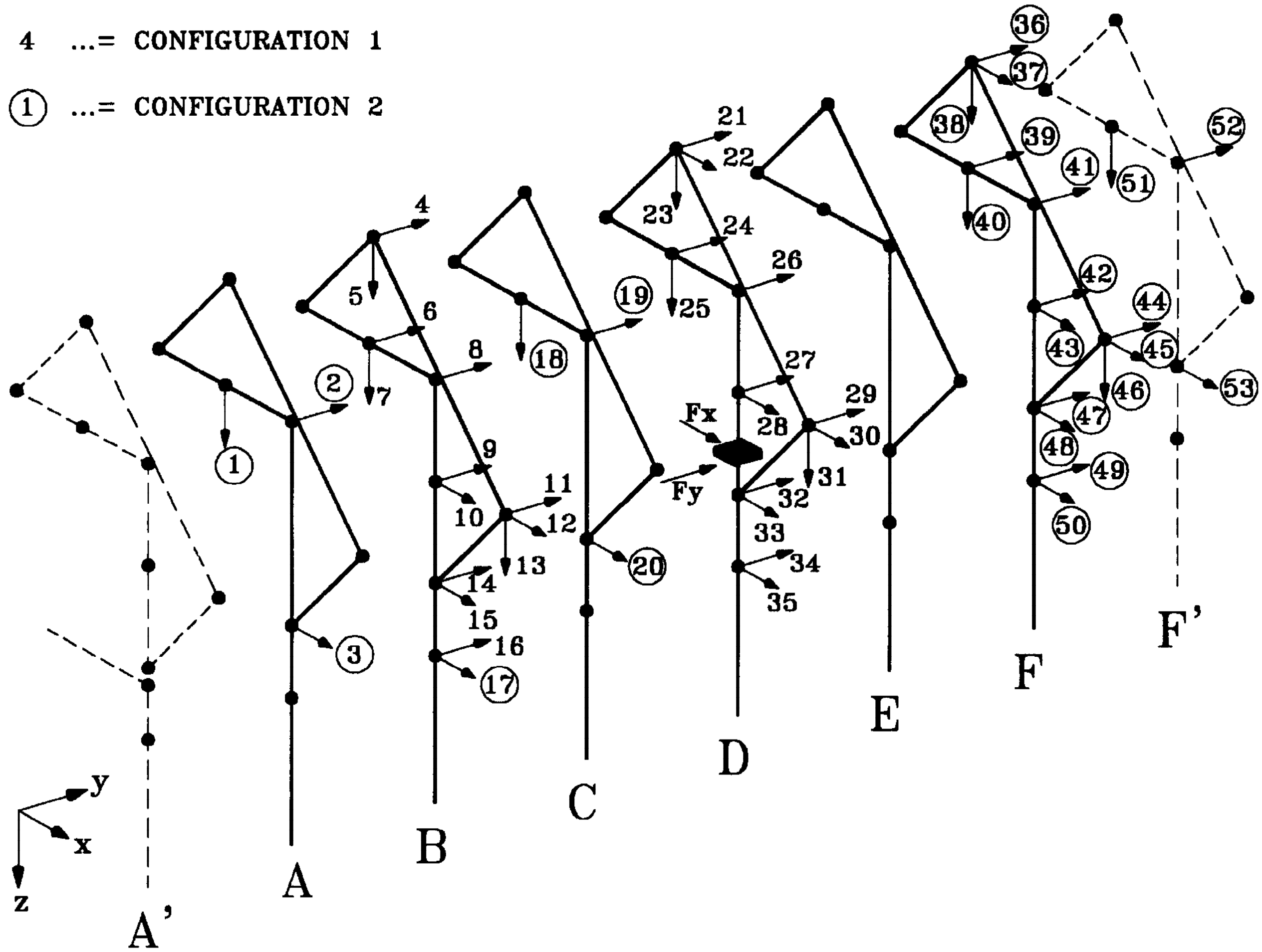

Fig. 2 Sensor locations

$$Q_j(t) = q_j \cdot sin(2\pi ft + \varphi_j)$$

The components relative to other frequencies were supposed to be negligible. The analysis was carried out by using the code ISA by ISMES, on a computer Digital Microvax. The input and output signals were amplified, filtered and digitised in real time. A computer system drove the frequency, collected the sensors records and computed the transfer functions (FRF) of the system in real time:

$$h_j(f) = q_j / p \cdot e^{i(\varphi_j - \varphi_p)}$$

Twenty tests were performed. In nineteen of them the structure was tested as described before. In each test a force acting in x or y direction was considered. The [1,15] Hz frequency interval was considered. Three tests with different values of α were necessary to cover the interval of interest, in order to limit the value of p. In the last test seismic actions were simulated, by applying a sinusoidal force, characterised by a frequency very close to the fundamental frequency of the structure and by an amplitude capable of generating the expected seismic stresses state in the structure.

DATA ANALYSIS

The dynamic characterisation tests allowed to identify the dynamic properties of the structure (Bendat and Piersol, 1984). The experimental behaviour was quite complex. Frequencies associated to both the modal shapes of the whole building, and the local resonances were pointed out. The following procedure was used.

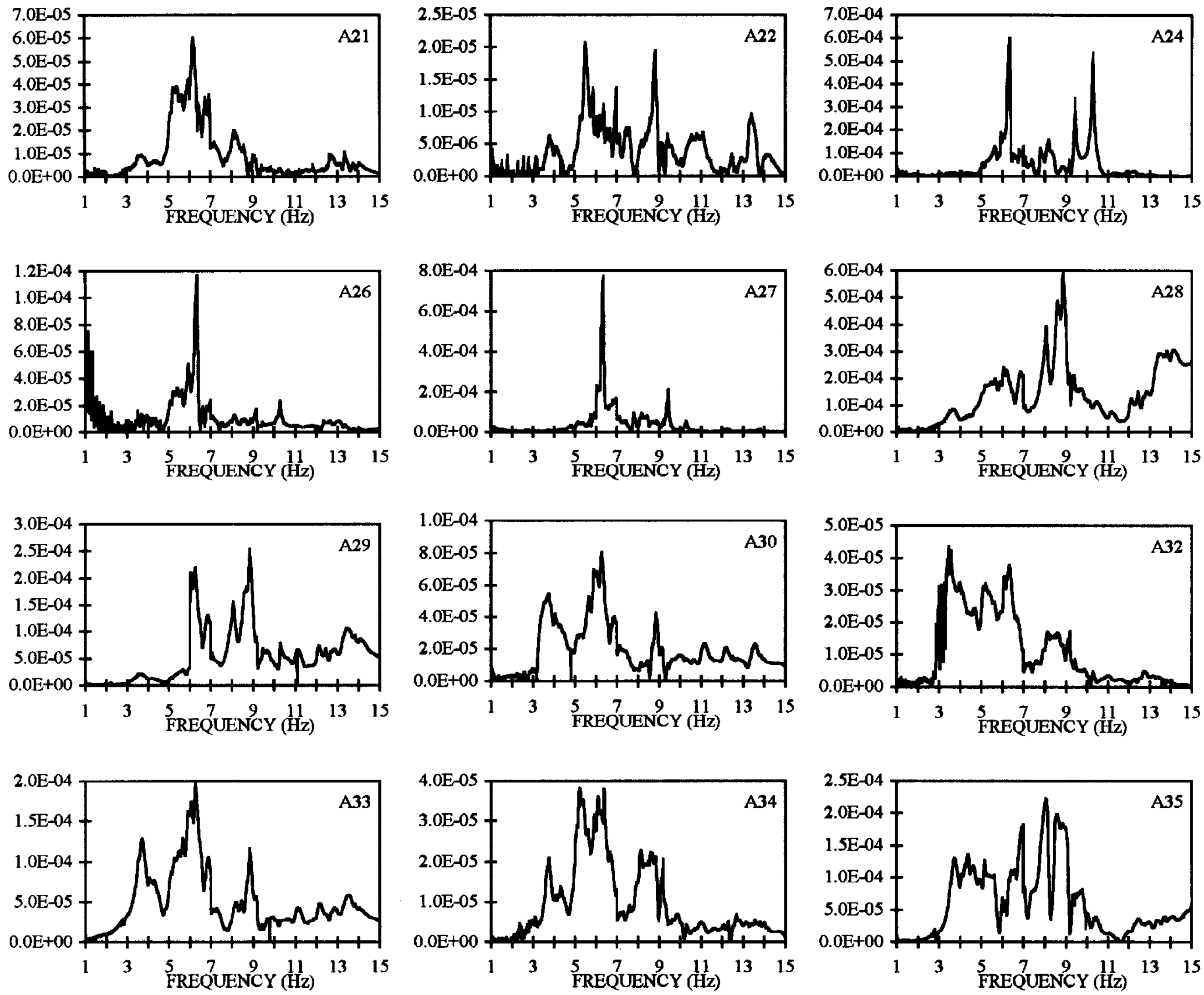

Fig. 3 - FRF of records at locations A21 to A35 - Force in x direction

The steel frames were first considered separately. For each of them the structural resonances were pointed out. Only the frequencies common to all the frames were considered as frequencies associated to modal shapes of the building. The other structural resonances were considered as local resonances, relative to a single frame or to a single member.

Figs. 3 show the FRF moduli relative to sensors located on frame D (Configuration 1), when the force is acting in x direction. In Figs. 4 the FRF moduli of the same sensors, relative to a sinusoidal force acting in y direction, are plotted. The phase factors, not shown here, were also plotted. Damping was calculated using the half-power bandwidth method (Ewins 1984). The frequencies summarized in Tab. 1 with the corresponding damping were assumed as structural resonances.

Tab. 1 Experimental resonances and damping

Mode No.	Frequency (Hz)	Damping (%)
1	2.3	4.0
2	3.8	3.9
3	6.3	1.3
4	7.7	1.0
5	9.8	0.5

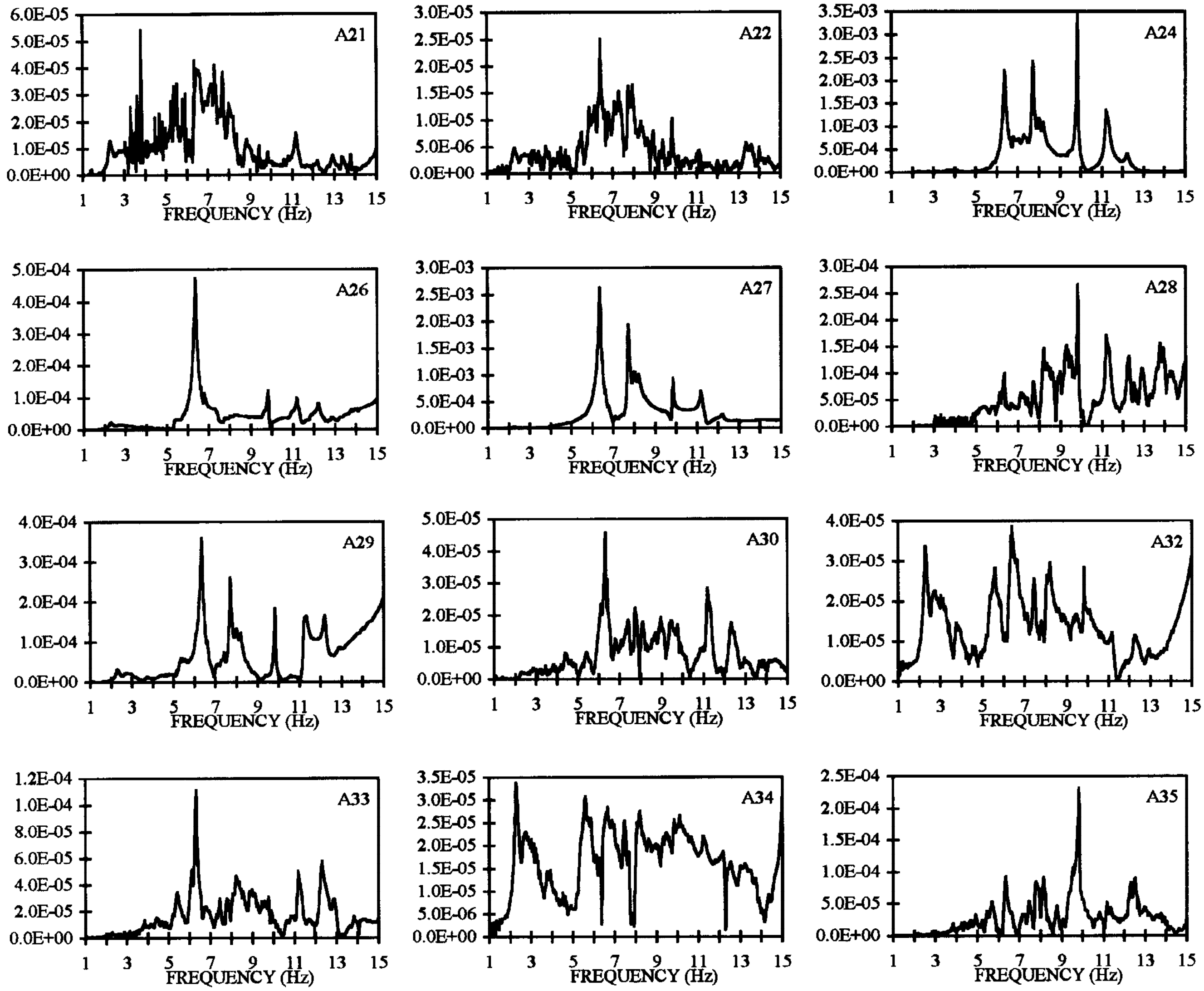

Fig. 4 FRF of records at locations A21 to A35 - Force in y direction

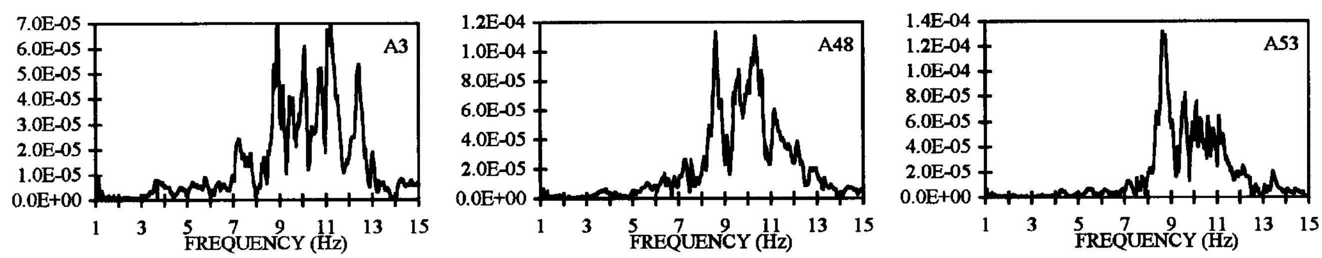

Fig. 5 FRF of records at locations A3, A48, A53 - Force in x direction

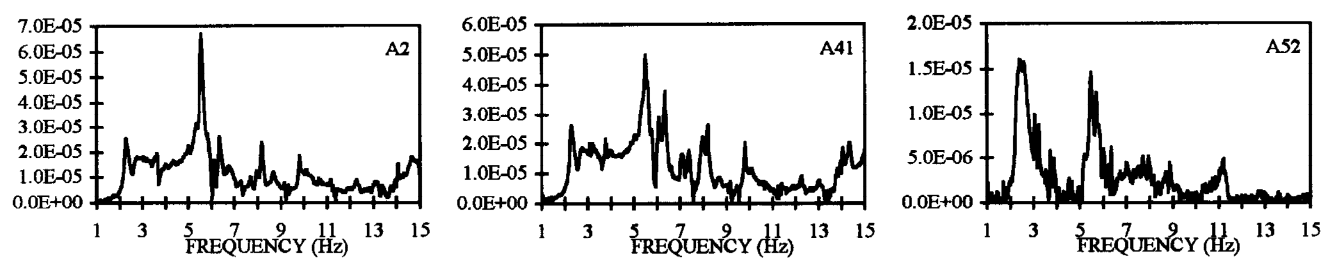

Fig. 6 FRF of records at locations A2, A41, A52 - Force in y direction

The comparison between records of frame A (A2 and A3) and frame F (A41 and A48) showed a non-symmetric behaviour of the skylight (Figs. 5 and 6). In fact, the structure is not symmetric, because of the absence of the lower horizontal beam of frame D. In the same figures the FRF relative to sensors located on frame F' (A52 and A53) are shown. The presence of some amplifications is apparent. This is probably due to the seismic joint between the concrete structures of the two blocks, which performed not very well.

From the FRF the first modal shapes can also be deduced. Only the second modal shape shows prevalent displacements in x direction. In the others the structure is prevalently displaced in y direction. In particular, with respect to the first modal shape, the main structure of each frame behaves as a vertical beam fixed at ends.

FINITE ELEMENT MODELS

Structural identification allowed to define the best finite element model, whose dynamic characteristics were very similar to the experimental ones. Models of the steel structure were first considered. Afterwards models of the whole building were analyzed.

In the simplest model, only the steel structure was considered, by using beam elements, fully constrained one to another. Different hypotheses were assumed for external constraints. A numerical investigation allowed to choose that of Fig. 7 as the best model. Pillars were fixed at the concrete structure and so were the lower horizontal beams. Only rotation around y axis were allowed at the end of upper horizontal beam. Masses of structural elements were distributed along the members. Masses of non-structural elements were lumped at nodes. Modal analysis produced the structural frequencies listed in Tab. 2.

The first modal shapes were quite similar to the experimental ones, but the values of frequencies were not very close to the experimental values. This disagreement between experimental and numerical results was due to the presence of non-structural elements. Their interaction with the structure is very difficult to model. A non-linear analysis would be advisable.

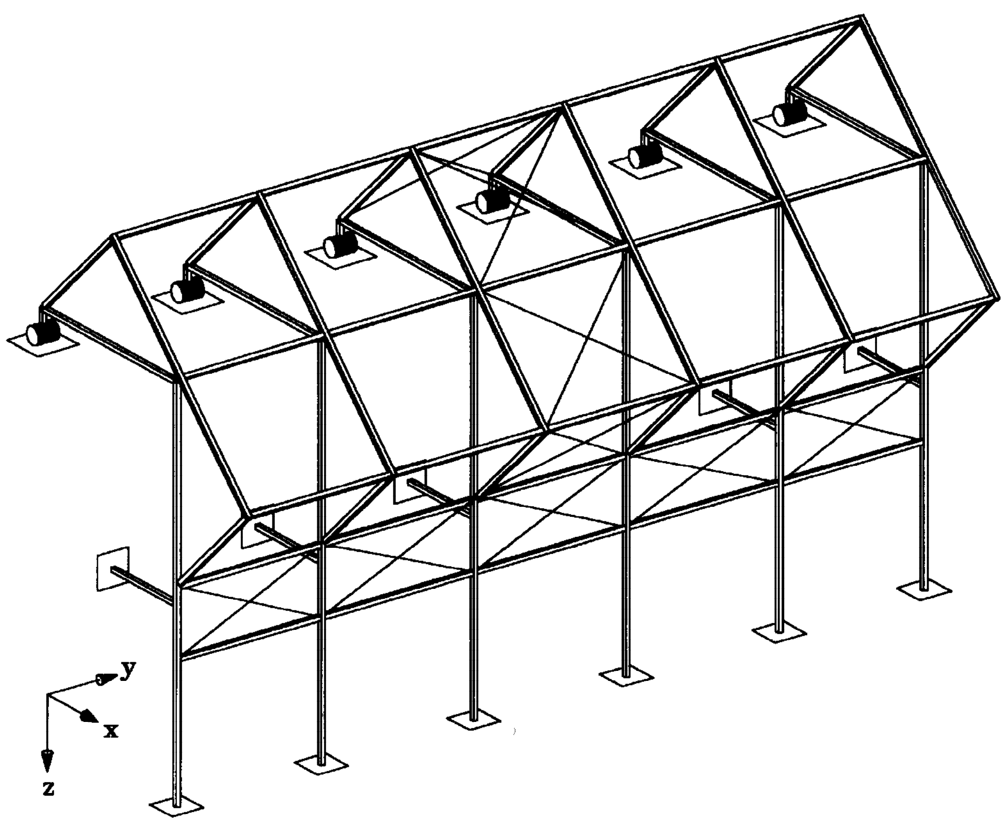

Fig. 7 Steel structure model

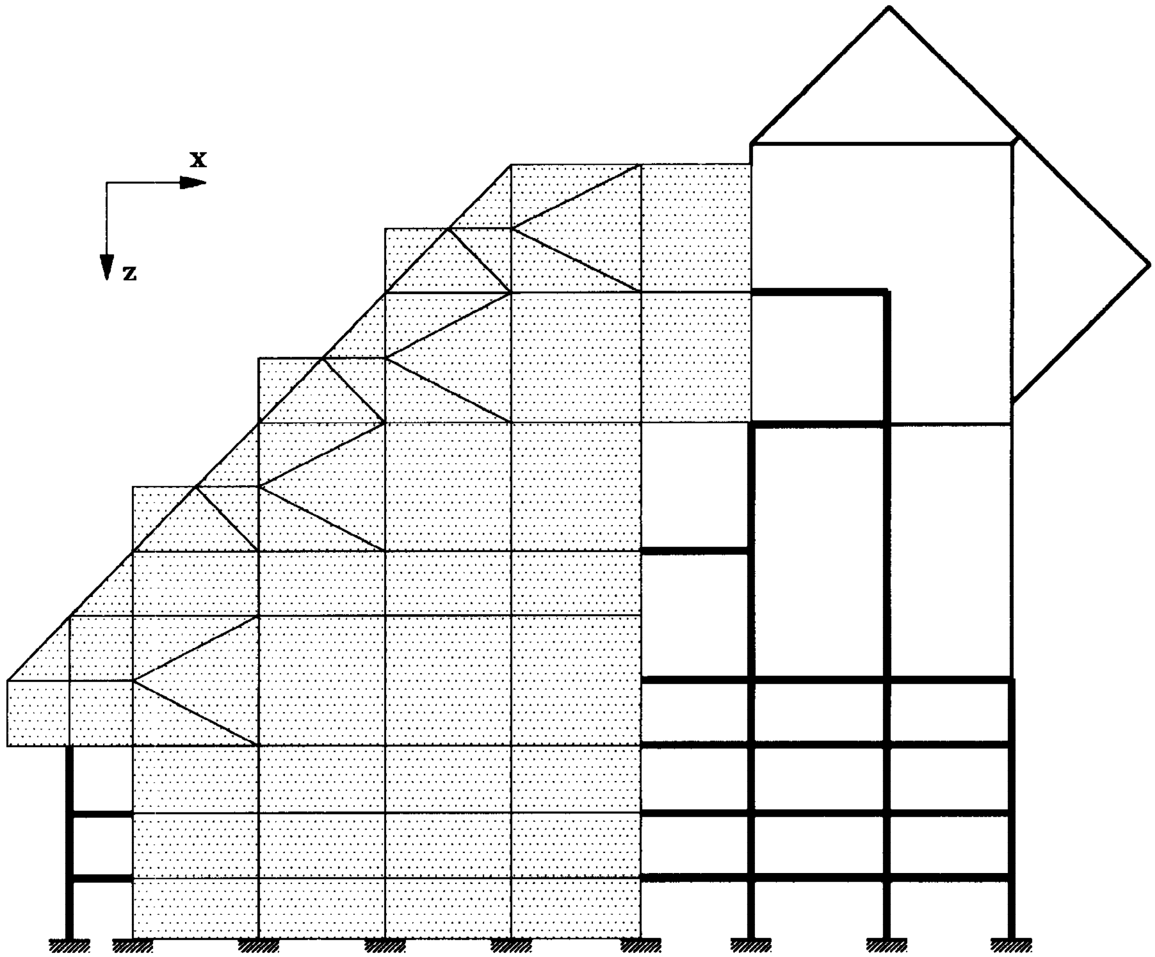

Fig. 8 FEM of the whole building: lateral view

Tab. 2 Structural resonances of the steel structure model

	Mode No.	1	2	3	4	5	6	7
Steel Structure	Freq. (Hz)	1.34	1.81	4.55	6.12	6.8	6.97	8.63

To remain in the linear hypothesis, bracing elements were introduced for accounting the contribution of the glass panels of the skylight. Size of diagonal beams were evaluated by equalizing their strain energy in a deformed configuration with that of the panels. In such system structural frequencies were very high and modal shapes were very different from the experimental ones. For this reason that model was dropped. Finally also the purlins were considered, and modelled as simply supported beams. In such system structural frequencies were very similar to those of the first model. Therefore we supposed the contribution of the purlins to be negligible.

Tab. 3 Structural frequencies of the whole building and of the concrete structure only

	Mode No.	1	2	3	4	5	6	7	8	9
Concrete + Steel S.	Freq. (Hz)	0.61	1.37	1.58	1.88	2.63	3.31	4.09	4.33	4.57
Concrete S.	Freq. (Hz)	0.62	-	1.64	-	2.19	3.15	3.85	4.26	-

In the previous models the concrete structure was assumed as infinitely rigid with respect to the skylight. This is almost true when considering displacements in x direction, because of the very high stiffness of the concrete walls, but not in y direction. A finite element model of the whole building was performed (Fig. 8). The steel skylight being modelled like in the previous case, the concrete structure was modelled as following. Walls and floors were discretized by using shell elements, having both axial and bending stiffness. Beams and pillars were modelled with beam elements. All the structure elements were considered fixed at the foundation.

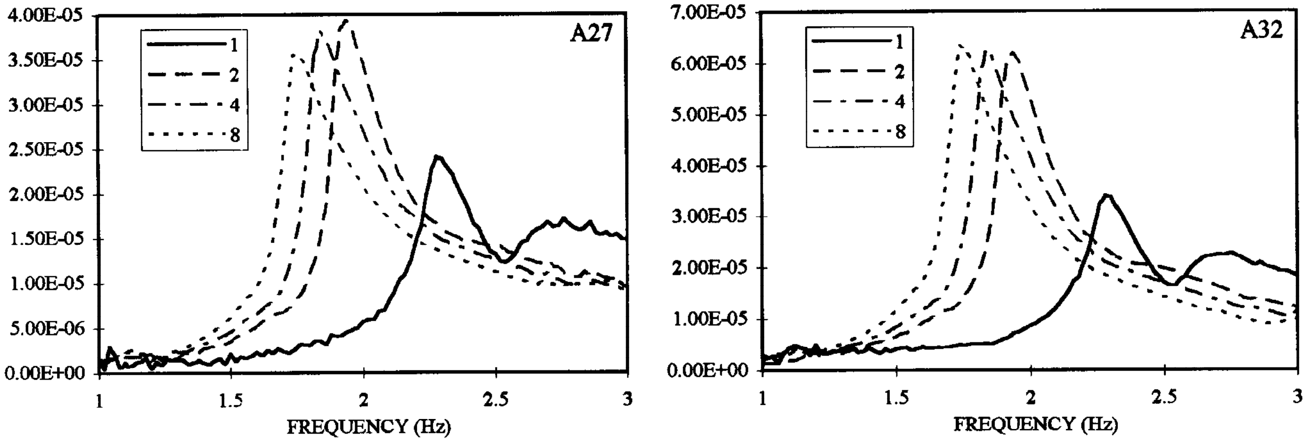

Fig. 9 FRF of records at locations A27 and A32 for different values of the force

In Tab. 3 the structural frequencies relative to the whole building (Concrete + Steel S.) and those of the concrete structure without the skylight (Concrete S.) are listed. As you can see the dynamic interaction between the concrete structure and the steel structure is very low. In fact, the frequencies of the two structures can easily be distinguished. In particular, the first six modes are relative to displacements in y direction. The seventh mode is torsional, while the eight mode shows displacements in both x and y directions. It is worth to point out that the first frequency of the whole structure, 0.61 Hz, is out of the investigated range [1, 15] Hz. So it doesn't appear in the FRF relative to the experimental analysis.

CONCLUSIONS

The presented analysis is based on the hypothesis of linear behaviour of the structure. The comparison between experimental and numerical results pointed out some disagreements between them. The reason was imputed to the contribution of the non-structural elements.

A series of tests was carried out with higher values of the force, in order to examine the linear behaviour of the structure. The analysis was limited to the frequency interval [1, 3] Hz. Force amplitudes respectively equal to 2, 4 and 8 times those relative to the shown FRF were considered. The FRF relative to records at locations A27 and A32 are plotted in Fig. 7. A reduction of the first frequency was pointed out. This behaviour was due to the contribution of non-structural components, which becomes negligible when the force get higher.

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