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EXPERIMENTAL DYNAMIC BEHAVIOUR OF LARGE STEEL STRUCTURES OF POWER PLANTS

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SUMMARY

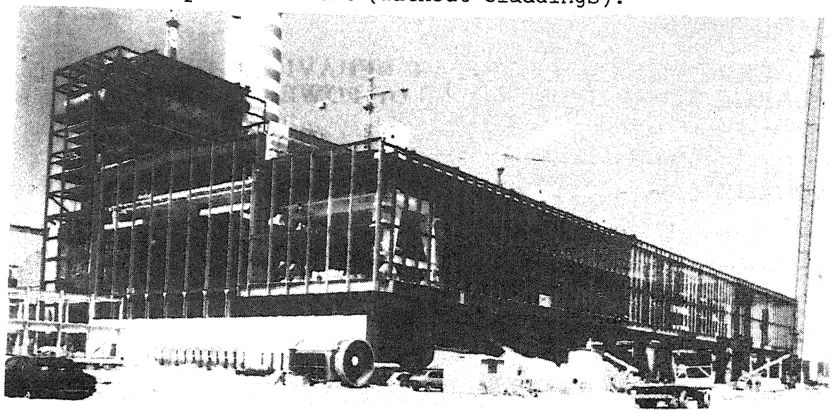
Experimental results of a series of tests on a large steel-frame structure are presented. Dynamic excitation has been applied by excentric mass shakers. Amplitude and phase of the response at various points allow to determine the transfer functions and then modal frequencies, mode shapes and critical damping of the structure. A numerical model has been formulated, using standard design methods. Comparison between theoretical and experimental results suggest to considerthe influence of additional stiffening elements and reduction of partecipating masses. Methods of sensitivity analysis and mean square extimation allow to identify the best structural and design parameters to be used to fit experimental frequencies and modal shapes.

THE BUILDING AND THE OBJECT OF THE STUDY

A typical building for thermo-electric power plants (four 660 MV generators) now adopted in Italy is shown in fig.1. The main structure of the Turbine Hall of the plant consists in a series of 17 parallel steel frames, made by box-type columns and girders. Each frame has two bays of different height. Above the lower bay a plane steel truss is hinged in two points to the main frame. Bracing and longitudinal girders rigidly connect the main frames in two groups of 8+9, with an expansion joint of sliding type in the middle. These frames are completely disconnected from the sorrounding buildings (in the back of the picture). Inside, turbines and electrical generators are resting each on its own massive reinforced concrete foundation, disconnected by a an expansion joint from the surrounding beams and columns (that support pipes and equipments). The laters form pin ended space frames rigidly connected to the main frames below the operative floor (+12m). The foundation of the main frames are large reinforced concrete girdes all around the perimeter, resting on piles.

The experiments have been planned to test the dynamical behaviour of the large steel structure of the Turbine Hall, whose sketch is described in fig.2 and 3. The object of the study is also a calibration of current theoretical models for the design of such structures, against earthquakes: basically to increase our capability of predicting frequencies and modal shapes at least in the elastic range.

Fig. 1. View of power station (without claddings).



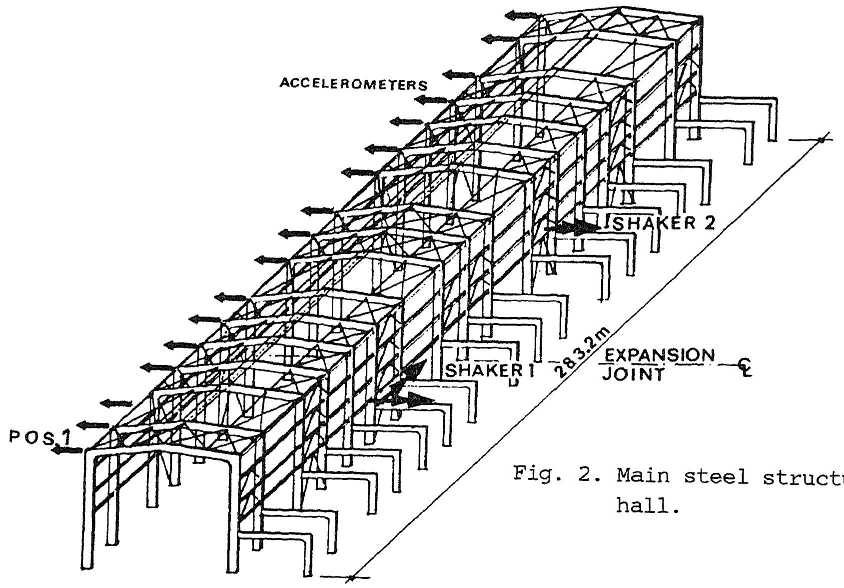


Fig. 2. Main steel structure of turbine hall.

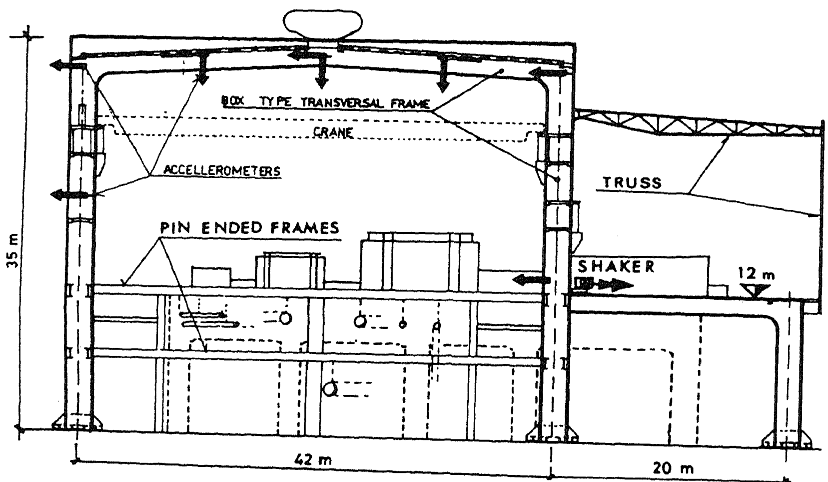


Fig. 3. Transversal section.

DESCRIPTION OF TESTS AND RESULTS

A shaker with excentric masses, able to produce 5.5 ton forces at 2.5 Hz, has been placed in two positions (both in transversal and longitudinal direction) at the operative floor (+12m from ground level), see fig. 2 and 3. Explored frequency range: 0.3 to 5.8 Hz. Accelerometers were placed in 28 position, at the top of each frame, (see fig.2) and at several points along one single frame (see fig.3). Frequency of the excitation was changed very slowly, such as to have quasi stationary response at each frequency.

Modulus and phase (with respect the excitation) of the harmonic response at each accelerometer was plotted versus the frequency, obtaining the experimental transfer functions, see fig.4, 5. It may be seen that at low frequencies the resonances are very clean. Detection of some higher modes is helped by inspection of the contemporary phase inversion. Using the maximum recorded moduli, at least four modal shapes, can be clearly detected, as shown in fig.6. The same figure gives experimental frequency and damping ratio (obtained by the $\sqrt{2}$ method), that seem to be rather high for a steel structure (may be because of rivet connection of claddings, friction in structural joints and in supports of attached mechanical components, energy dissipation of foundation and some aerodinamic affect due to the large facade/mass ratio). Damping ratios are increasing with modal frequencies. Others resonance frequencies around 3 Hz are detected in the vertical direction at mid span of the higher girder of the frame, corresponding to vertical flexural modes of each frame independently. Again, a damping ratio rather high is observed. It is interesting to note that in the longitudinal mode the two groups of 8+9 frames moves in phase: in other words the sliding supports in the expansion joint act as a rigid connection, at low amplitude. No out of phase experimental displacement was detected even for higher frequencies.

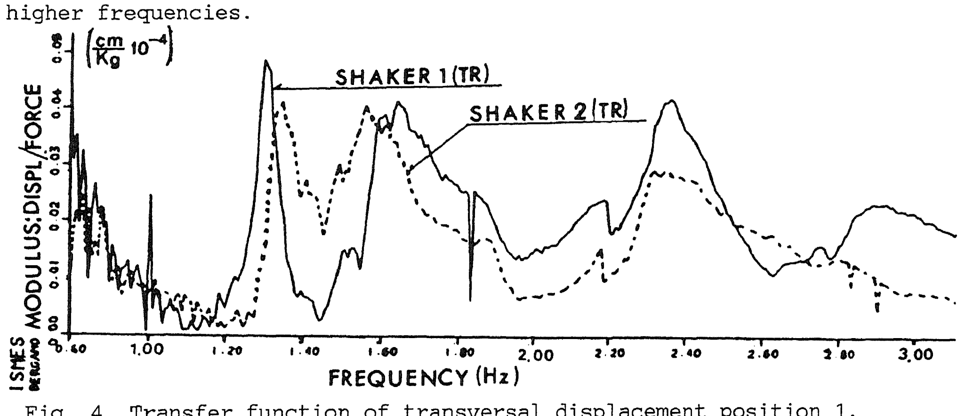


Fig. 4. Transfer function of transversal displacement position 1.

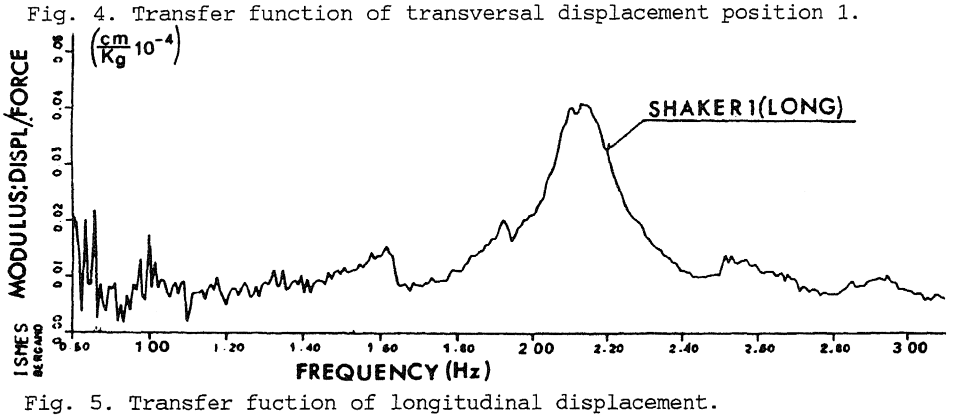


Fig. 5. Transfer fuction of longitudinal displacement.

THE THEORETICAL MODEL AND DYNAMIC IDENTIFICATION

A multi frame model has been formulated, each frame having 16 nodes and 17 beam elements. Bracings and girders connecting the frames are simulated by truss elements. Masses are extimated using the real bills of quantity recorded during construction. Secondary structures (internal pin-ended steel frames below the operative level and 3-hinge trusses above the lower bays) are considered as bare masses (zero stiffness) attached to the main frames. Foundation on piles are simulated by rotational and horizontal springs using current design methods. The obtained results in terms of modal frequencies and shapes are summarized in fig.6. Note that a theoretical mode (flexural transversal at 1.14 Hz) has not been experimentally excided, since the shakers were close to a "node" of the mode.

Comparison with experimental results allows few interesting observations:

a) Theoretical frequencies are lower, i.e. the real structure is stiffer and/or has lower (participating) masses. Let $z_1 = E/M$, where E = Young Modulus of steel (to take into account the effect of a distributed stiffness) and M = total partecipating masses.

b) The transversal experimental mode (1^{st}) has lower transversal displacements in the first and last frames than in the central frames. This points out the real contribution of the claddings as stiffners (double folded steel sheet of 1mm each) that are disregarded in the theoretical model. Let z_2 = shear stiffness of the two frontal bays.

c) The consideration above applies also for the stiffness of the cladding of the longitudinal walls (z_3) and of the roof (z_4), the former influencing the longitudinal frequency (4th mode) the latter the flexural frequency (3rd mode).

d) An alternative theoretical model with infinite stiffness of the foundations was calculated. Since frequencies and ratio of modal displacements of this case differ by less then 1%, from the previonsions ones, foundation are not considered as important for the dynamic identification of this building.

Let us now consider the vector h of few structural effects: namely the modal frequencies and the ratios of maximum and minimum trasversal displacements of the roof. Be Y the vector of the corresponding recorded values in all the experiments. The errors between theoretical and experimental values are given by

$$u_i = h_i - y_i.$$

The values of h depend on the structural parameters: we have consider only the four above mentioned parameters z_i, groupped in the vector Z. We want to identify the vector Z' that solves

$$\min \; (u_i^2 \, v_i) \tag{1}$$

where v_i in this case have only the meaning of scaling factors. Let z_j° be the value of the structural parameters of the theoretical "basic" model, then

$$h_i(z_j) = h_i(z_j^\circ) + \sum_j \frac{\partial h_i}{\partial z_j} (z_j - z_j^\circ) + \ldots \tag{2}$$

If we consider only the linear part, the (2) becomes in matrix form

$$h = h^\circ + H(z - z^\circ)$$

where:

$$H_{ij} = \frac{\partial h_i}{\partial z_j}$$

In practice H is obtained by finite numerical perturbation of the structural parameters of the theoretical "basic" model.

The solution of (1) becomes (see ref. 2)

$$Z' - Z^\circ = [H^T V H]^{-1} H^T V (Y - h^\circ)$$

where $V = diag[v_i]$

One iteration only of the method is enough to obtain an "updated" theoretical modal in good agreement with the experimental behaviour. The corresponding structural parameter are given in fig.6 and are discussed in the following section.

CONCLUSIONS AND ACKNOWLEDGEMENTS

1) Experimental frequencies are higher than the theoretical ones.
2) They can be reproduced in a theoretical model that takes into account the stiffness of claddings. In our case this was found abont 35% of the total stiffness of the steel bracings of the walls and of the roof.
3) An increase of 40% of the ratio E/M for the main steel structure is also required to fit experimental results. This can be explained by theoretically considering also the stiffners of the box girders, and some rigidity of the internal pin ended frame type structures. In addition a reduction in the masses of pipe, equipments etc, because of their loose connection with the main structure can explain a reduction of the total theoretical mass M.
4) Damping ratios of 1.8 to 3.5% are detected. They are increasing with modal frequencies thus suggesting some viscous behaviour.
5) Friction of sliding expansion joint produces in-phase displacements of the two parts of building.

Care must be taken in extrapolating all the above conclusions to strong motion excitation. Experimental and theoretical modal shapes show that non simultaneus supports excitation must be considered in a seismic design of this type of structures.

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2) J.G. Beliveau, "The role of system identification in design", Basic questions of design theory, W.R. Spillers Editor, North Holland Publishing Co., 1974.

TRANSVERSAL MODE

f = 1.3 Hz (exper.)
a = 4.35 "
ν = 1.8 % "
f = 1.013 Hz (theor. basic)
a = 1.05 "
f = 1.24 Hz (theor. updated)
a = 3.04 "

TORSIONAL MODE

f = 1.6 Hz (exper.)
ν = 2.5 % "
f = 1.04 Hz (theor. basic)
f = 1.40 Hz (theor. updated)

FLEXURAL MODE

f = 2.35 Hz (exper.)
ν = 2.7 % "
f = 1.45 Hz (theor. basic)
f = 2.07 Hz (theor. updated)

LONGITUDINAL MODE

f = 2.11 Hz (exper.)
ν = 3.2 % "
f = 1.65 Hz (theor. basic)
f = 2.04 Hz (theor. updated)

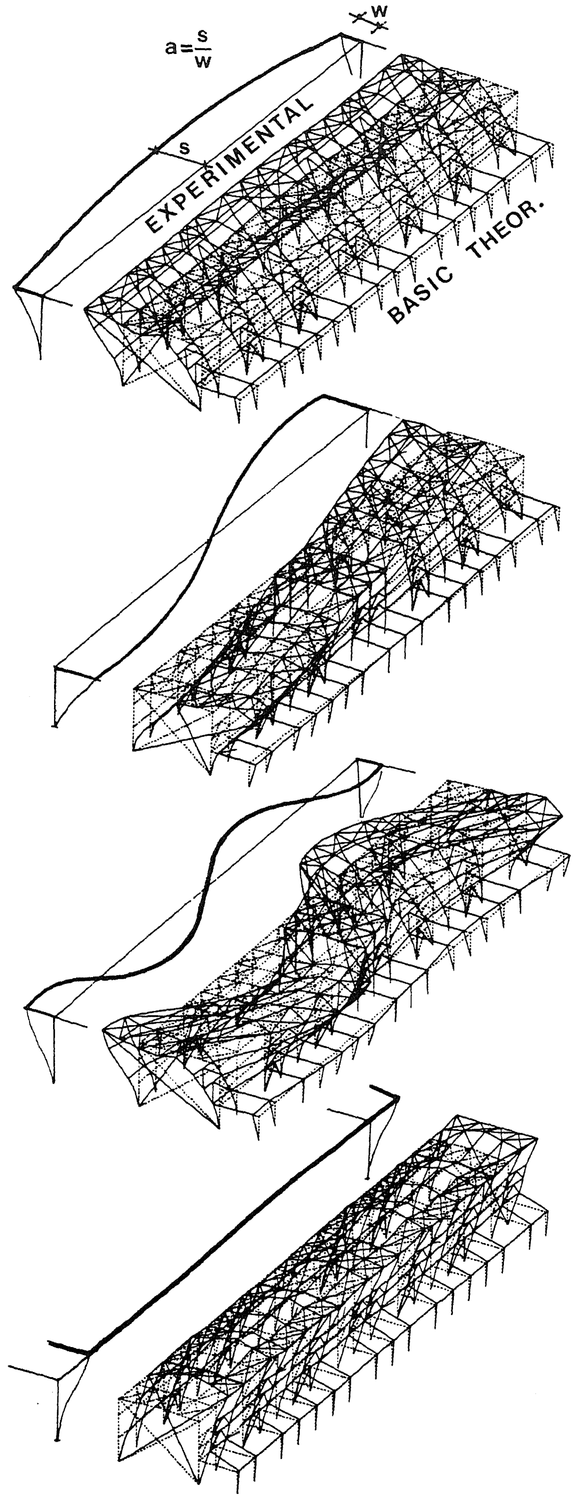

Fig. 6. Experimental and theoretical modal shapes, frequencies and damping ratios.