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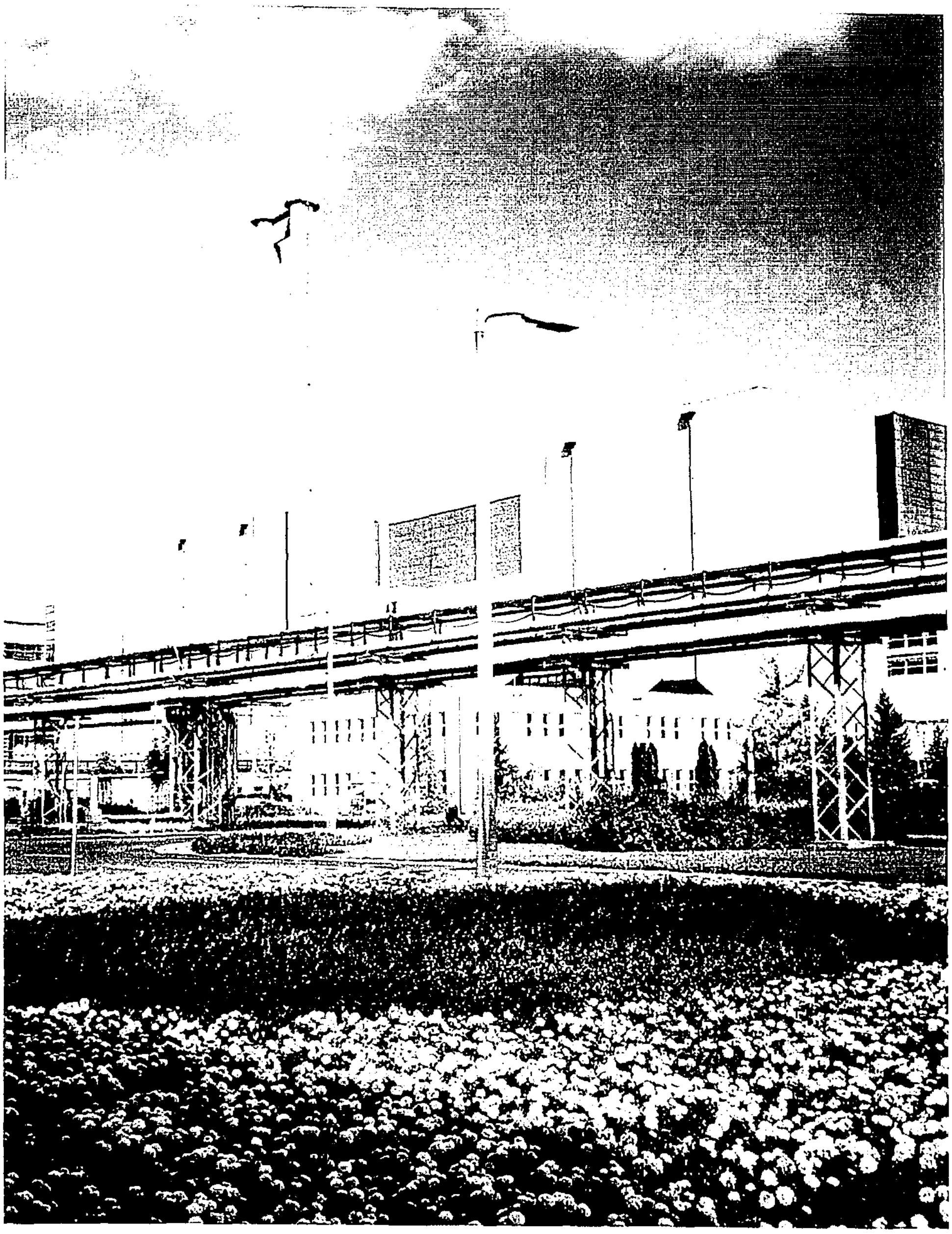

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REVIEW REPORT ON THE DYNAMICAL STUDY OF THE MAIN BUILDING OF THE PAKS NPP

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SUMMARY

The present report deals with the review of the report "Dynamical Study of the main building of the Paks NPP", issued by Paks NPP (Hungary) on April, 1993, within the frame of the IAEA benchmarck study for the seismic analysis and testing of an existing Nuclear Power Plant (NPP), and on behalf of ENEL DSR/VDN Rome, in the aims of the nuclear activities of ENEL DSR/VDN (Rome).

After a foreword to define the aims of the job (Chapter 1) and the identification of the scope of the work (Chapter 2), a short list of references is given (Chapter 3). In Chapter 4, the criteria followed in the review activity are listed; in Chapter 5, the contents of the Paks NPP report are summarized.

In Chapter 6 the results of the review are given, while the main conclusions of the review activities are summarized in the Chapter 7.

This document has 29 pages and 11 figures.

I N D E X

1. FOREWORD

In order to study the dynamic characteristics of the main building of the Paks NPP, explosion tests were carried out on 1988, 1990 and 1991. The explosion tests performed during 1988 were preliminary, in order to tune the analytical and experimental procedures for this type of tests.

The tests performed during 1990 had the aim to study the dynamic response of the primary part of the main building, the auxiliary building and the Diesel housing.

The tests performed during 1991 had the aim to study the whole coupled building complex, including the soil structure interaction.

All the tests are analyzed in reference document [1].

2. SCOPE

Within the frame of the IAEA benchmarck study for the seismic analysis and testing of an existing Nuclear Power Plant (NPP), ISMES has performed the review of the report concerning the explosion tests on Paks NPP (see doc. [1]). This review has been funded by ENEL DSR/VDN Rome.

The scope of the work is mainly to understand the experimental and analytical procedures used by Paks NPP and to integrate them, if necessary, with some observations coming from the western countries current practice, taking into account in particular ISMES past experience on similar jobs (for example explosion tests on Garigliano NPP, see ref. [5]).

According to this scope, the report [1] has been analyzed in detail and some limited processing job was performed on the data supplied by Paks NPP (see doc. [2]), in order to support the conclusions drawn from the review activity.

3. REFERENCES

[1] L. Turi: "Dynamical study of the main building of the Paks NPP", Paks Nuclear Power Plant, Paks (Hungary), April, 1993

[2] Letter from Mr. L. Turi to ISMES, February, 1994

[3] US Nuclear Regulatory Commission Sect. 3.7.1: Seismic Design Parameters

[4] US Nuclear Regulatory Commission NUREG 75/087

[5] ISMES Doc. REL-DPD-0405: "Records on the Garigliano NPP relevant to "seismic" vibrations generated by explosions" (in Italian), on behalf of ENEA-PAS, June 1987

[6] A. Bianchi, F. De Pasquali, F. Gatti, F. Muzzi, M. Zola: "Commissioning of qualification of structures, systems and components for seismic and environmental loads of CIRENE NPP", presented in "Upgrading of existing NPPs with 440 and 1000 MW VVER type pressurized water reactors for severe external loading conditions", Vienna, 23-25 August 1993

4. REVIEW CRITERIA

The criteria followed during the review activities were the following ones:

1) verification of the completness of the report;

2) verification of the pre-processing of the rough data acquired during the experimental tests;

3) verification of the analytical procedures used for the response spectra computation;

4) verification of the analytical procedures used for the amplification factors and damping ratios computation.

5. PAKS NPP REPORT CONTENTS

For sake of completness, the table of contents of the report [1] is reported hereinafter:

2.3.3 Experiences: results

3. **Data processing**

3.1 Time signals

3.2 Response spectra

3.3 Dynamical amplification

3.4 Free-field measurements

3.5 Damping

A short summary of the content of the report is listed hereinafter.

Referring to the table of Fig. 1, drawn from the doc. [1], the data analyzed in the report [1] are relevant to the following tests:

- tests I/4500: the tests were carried out in November 1990; the signals processed are relevant to two explosion tests, carried out with 500 kg of explosive at a reference distance from the plant of 4500 m, along a direction SE;

- tests II/2100: the tests were carried out in November 1991; the signals processed are relevant to two explosion tests, carried out with 300 kg of explosive at a reference distance from the plant of 2100 m, along a direction E;

- tests II/3200: the tests were carried out in November 1991; the signals processed are relevant to two explosion tests, carried out with 450 kg of explosive at a reference distance from the plant of 3200 m, along a direction S.

For each test, the time histories of the free field accelerations and of the structural response accelerations in the measurement positions are shown (see Fig. 2 for the measurement positions for which the time histories are shown).

For each measurement position, the response spectra at three different damping ratio values (5%, 7% and 10%) were computed.

The ratios between the response spectra at the various measurement positions and the basemat response spectrum (or the spectrum at the lowest level of the building) were also computed; from these functions, the dynamic amplification was detected.

It was also indicated a fitting procedure to compute, from the time histories of the signals, the damping ratio at different measurement positions.

6. REVIEW RESULTS

6.1 Verification of the completness of the report

In the report, all the main data regarding the tests and the processing procedures are described. Nevertheless the report would be more complete if the following data were listed:

- characteristics of the measuring instruments used during the tests: type of accelerometers used, their sensitivity, frequency bandwitdh, type of amplifiers, etc.;
- layout of the measuring instruments (Figs. 1-3 of the report [1]), completed with the coordinates (with respect to a reference system) of the measurement positions;
- detailed procedures used in the computations of the response spectra (see para. 6.3 of this report).

6.2 Verification of the correctness of the pre-processing of rough data

6.2.1 Averaging of the time data

As far as the time histories of the explosions with the same parameters (distance and load) show a "good reproducibility" (para. 2.2.3 of [1]), the average of different time signals could be a critical item, bringing to the risk to obtain an averaged signal with a frequency content different from that of the original signals, due to the different starting time of the acquisition, which requires a very precise fitting of the data, not so simple to obtain and to an "apparent" similarity among the signals relevant to different tests. In this case, the number of tests with the same explosion parameters is not so high (two or three tests) so the analyses could be performed on each set of recorded time histories: at the end, the results of the analyses could be organized in order to have a range of values (natural frequencies, damping factors, amplifications, response spectra, etc.) on which statistical considerations (mean values, scattering, upper and lower bounds) could be made.

6.2.2 Computation of P-wave and S-wave velocities

Probably in the report there is a typographic mistake on the values of P- and S-waves velocities: the right values should be 2000 and 350 m/s instead of 2000 and 3500 m/s. In fact, the P-wave velocity is always greater than the S-wave velocity. The report would be more complete if the procedure used to compute the values of these velocities was given.

6.2.3 Selection of the time window to process

The procedure according to which the time windows of the recorded signals to process were choosen should be clarified; it seems that the portion of the signals submitted to the processing is that in which the effect of the initial high-frequency package is ended. Nevertheless, it is not so clear if this choice is based only on a "visual" examination of the accelerograms or if it is based on different criteria. Perhaps, a filtering of the signals (for example through a low-pass filter with a cut-off frequency of 3-5 Hz would have been more rigorous, assuring that all the higher frequencies components are not present in the signals.

6.3 Verification of the response spectra computations

The review of the para. 3.2 "Response spectra" of the doc. [1] has originated the following comments:

1) it is not clear which procedure has been followed for the computations of the response spectra: western standards (such as [3]), suggest frequency intervals for calculation of response spectra (see Fig. 3) or, in alternative ([4]), give a fixed frequency interval ranging from 0,5 to 32 Hz in which the frequencies greater than 0,5 Hz are obtained with an increase of 8,86% of the previuos one; the frequency resolution in the computation of response spectra is a big problem in order to understand the frequency content of the signal, mainly in the cases (such that presented in this report) in which were not used Fourier transforms of the signals.

2) In order to substantiate the previous comment, the time histories supplied by Paks NPP relevant to measurement position M1, M2, M3 and M5 (direction X) at elevation -6,5 m in the Main Building were processed, following ref. [4], in order to obtain the corresponding response spectra at a damping ratio of 5%. The results are shown in Fig. 11. As it can be noted, the shape and the spectral accelerations of the spectra are different from those shown in Fig. 4, drawn from doc. [1]. The signals processed have the same time duration and starting instant of those shown in Fig. 5.1 of the doc. [1].

3) It is not clear the reason why the response spectra corresponding to signals measured in different positions at the same elevation of the buildings were averaged; they do not seem to be "approximately identical"

both in frequency distribution and in amplitude, to justify such a procedure (see, for example, spectra at level -6,5 m for test I/4500, Fig. 8.2 of the doc. [1], shown also as Fig. 4 in this report).

6.4 Verification of the dynamic amplification computations

The computation of transfer functions from signals relevant to explosion tests is a hard problem, as correctly highlighted in para. 3.3 "Dynamical amplification" of the doc. [1]. From the transfer functions obtained in such a way, it is not possible to detect natural frequencies and the relevant amplifications and dampings. On the other hand, due to the non-stationary nature of the explosion, it is not possible to compute the transfer functions in the classical way, dividing the cross spectral density of the response signal by the autospectral power density of the excitation signal, averaged on a number of samples of the original signals.

The way chosen in the doc. [1] to compute amplifications of the structure could be seen only as a rough estimate of the dynamic behaviour of the structure. In this case, the amplification factors were detected on diagrams resulting from the ratio between the response spectra relevant to the building measurement positions and the response spectrum of the base excitation.

From a theoretical point of view, this ratio has no meaning, because response spectra are histograms, not functions. In response spectra, only maxima of the response of a single-degree-of-freedom (SDOF) system of given natural frequency and damping are plotted: the spectrum is indicative of the frequency content of the signal only roughly, because the values of the spectrum at a particular natural frequency of the SDOF depend not only upon the presence in the signal of that frequency but also upon the contributions of all the other frequencies contained in the signal itself. In this way, it is doubtful that the maximum values of the diagrams shown on the report [1] could be seen as amplification factors. Moreover, it has to be underlined that in the response spectrum is completely missing the information about phase relationship among different components of the analyzed signal.

In order to clarify this very important item, some numerical computations have been performed.

A synthesized acceleration time history (with a time resolution of 8 ms, sampling frequency 125 Hz), assumed to be a base acceleration, has been input into a SDOF, with a damping ratio of 5% and a natural frequency of 6 Hz. The response of the SDOF was computed by the 4^{th} order Runge-Kutta method; the base acceleration and the response acceleration are shown in Fig. 5.

The response spectra of these two numerical signals were computed, at a damping ratio of 5% (see Fig. 6), with the method proposed in [4]. Then the procedure followed in the report [1] has been applied, computing the ratio between these two response spectra; in the Fig. 7, the resulting diagram is shown, together with the analytical transfer function of the SDOF system.

The comparison between the ratio of response spectra and the analytical transfer function for the SDOF system is shown in Fig. 8; as it could be seen, through the ratio of response spectra is possible to detect, with the approximation related to the frequency intervals used in the calculation, the natural frequency of the system; for what concerns the amplification factor, the ratio of the response spectra seems underestimate (about -30%) the true amplification of the system. Although this conclusion is limited to the numerical example, it can demonstrate the difficulty to detect the true amplification factors through the procedure followed in the reviewed report.

For sake of completness, in Fig. 9 the transfer function of the SDOF system computed through the ratio of Fourier transforms of the base and response accelerations and the analytical transfer function are shown; in Fig. 10, these two functions are plotted together, showing, in the case of numerical example, their good agreement.

Another item that should be clarified consists in the consideration that dynamic amplification computed through the procedure outlined above at a damping ratio of 5% are "slightly higher than those calculated with 10% ones, the reason for that are the different frequencies and the different signal widths" (see pag. 13

of [1]). There is not any closed-form theoretical relationship among spectra computed with different damping ratios because, as it has been said before, spectra are histograms not functions. So, in principle, it is not possible, from a rigorous point of view, to foresee the amount of the variation in the spectral quantities (i.e. accelerations) from a damping ratio to another one.

For what concerns the damping factors computation, they correctly were not evaluated on the power spectrum of the signals, because the concept of power spectrum is not applicable to non-stationary signals as those generated during explosion tests. The procedure outlined in the para. 3.5 of [1] is very interesting, taking into account that the coefficient β theoretically is not the damping factor, but it is depending upon it through the following relationship:

$$\beta = \zeta / (1-\zeta^2)^{1/2}$$

in which the true damping factor is ζ.

This is due to the particular function used to fit the decaying portion of the experimental data, which is simply the free response of a single degree of freedom system:

$$a(t) = c_0 \exp(-c_1 t) \sin(c_2 t + c_3) + c_4$$

in which:

$c_1 = \zeta\, \omega_n$ with ω_n natural angular frequency

$c_2 = \omega_n\, (1-\zeta^2)^{1/2}$

So, the ratio c_1/c_2 ($= \beta$) is, with a good approximation, equal to the damping factor ζ, only if ζ has a value much smaller than the unity.

6.5 Experimental evaluation of the soil-structure interaction

6.5.1 General procedure for the soil-structure interaction evaluation

Although not present in the report [1], an interesting improvement in the processing of the data of the explosion tests could be the study of the soil-structure interaction (SSI) problem.

In the following a detailed description of the theoretical basis of the SSI problem and of the analytical procedure to manage the experimental data in order to compute the parameters of the SSI is given. This procedure is briefly described in ref. doc. [6].

In order to evaluate the soil-structure interaction the following procedure can be adopted.

A) A lumped mass model of the structure should be adopted and in correspondence of each general mass, translational mass or rotational moment of inertia depending upon the type of degree of freedom, a measuring point should be placed on the real structure in order to measure the motion associated with the considered degree of freedom.

B) Through the excitation of the structure, the absolute accelerations in every selected point of the structure should be measured.

C) The soil-structure interaction should be reproduced with a black box system reproducing the structure together with its foundations interacting with the soil through a system of six general forces: three forces acting along the orthogonal axes of a reference system and three moments about the same three reference axes.

D) The motion of the foundations should be measured in order to determine its six components: three translations and three rotations referenced to the same above defined orthogonal axes.

E) The transfer matrix relating the above defined general displacements and general forces is called the dynamic modulus of the system; should the general velocities be measured, the transfer matrix becomes the wellknown impedance matrix of the system.

F) Through a suitable processing from each element of the above determined matrix is possible to evaluate the soil characteristics: stiffness and energy dissipation capabilities.

6.5.2 Lumped mass model

The equilibrium of a lumped mass model is described by the following equation:

$$[M] * \{ d^2q_A/dt^2 \} + [C] * \{ dq_A/dt \} + [K] * \{ q_A \} = \{ g_E \}$$

where:

[M] is the mass matrix of the model comprehensive of structure, foundation and near field soil (the near field soil is that part of the soil surrounding the foundation and whose motion is determined not only by the far field excitation motion, but also by the structure reaction)

[C] is the damping matrix of the model representing structure, foundation and near field soil

[K] is the stiffness matrix of the model representing structure, foundation and near field soil

$\{ d^2q_A/dt^2 \}$ is the vector of the absolute accelerations of the masses, functions of time

$\{ g_E \}$ is the vector of the external forces, functions of time, applied to the structure, and on the boundary of the near field soil.

It is worthnoting that not only the mass matrix describes structure, foundation and near fiel soil, but the same holds for damping, stiffness and motion components.

The equilibrium equation can be rearranged highlighting the motion components of the degrees of freedom of the boundary elements of the near field soil in the following way:

$$[M_p] * \{d^2q_A/dt^2\} + [C_p] * \{dq_A/dt\} + [K_p] * \{q_A\} = \{g_E\}$$

where:

$$[M_p] = \begin{matrix} M_{ll} & M_{lv} \\ M_{vl} & M_{vv} \end{matrix}$$

is the partitioned mass matrix where the subscripts ll are referred to the free degrees of freedom and vv to the boundary elements

$$[C_p] = \begin{matrix} C_{ll} & C_{lv} \\ C_{vl} & C_{vv} \end{matrix}$$

is the partitioned damping matrix where the subscripts ll are referred to the free degrees of freedom and vv to the boundary elements

$$[K_p] = \begin{matrix} K_{ll} & K_{lv} \\ K_{vl} & K_{vv} \end{matrix}$$

is the partitioned stiffness matrix where the subscripts ll are referred to the free degrees of freedom and vv to the boundary elements

$$\{q_A\} = \begin{matrix} q_{Al} \\ q_{Av} \end{matrix}$$

is the vector of the absolute displacements and q_{Al} are the displacements of the free degrees of freedom and q_{Av} are the displacements of the soil at the boundary of the near field soil and they are equal to the displacements of the far field (seismic excitation or explosion excitation)

$$\{g_E\} = \begin{matrix} g_s \\ g_v \end{matrix}$$

g_s is the vector of the external forces applied to the structure, if any, and g_v is the vector of the unknown boundary reaction forces applied to the boundary elements of the near field soil.

The set of equations can be splitted in two sets of equations depending upon there is excitation

just by external forces applied to the structure

$$[M_p] * \{ d^2q_A/dt^2 \} + [C_p] * \{ dq_A/dt \} + [K_p] * \{ q_A \} = \{ g_E \}$$

or just by the far field soil

$$[M_p] * \{d^2q_A/dt^2\} + [C_p] * \{dq_A/dt\} + [K_p] * \{q_A\} = \{ g_V \}$$

In case of soil excitation the vector of the external forces contains just the reaction of the far field soil on the near field boundary elements.

In the hypothesis of validity of the modal analysis both equations can be transformed introducing the modal shapes concept and the absolute acceleration of each mass, i.e. in each measuring point, can be expressed as:

with sinusoidal force excitation

$$x_n = ({}_h\sum^N P_h * H_h * \phi_{nh}) * \sin(\omega * t)$$

where

$P_h = ({}_j\sum^N F_{0j} * \phi_{jh})/({}_j\sum^N \phi_{jh} * \phi_{jh} * m_j)$ is the modal participation coefficient for force excitation

N is the total number of degrees of freedom

F_{0j} is the peak value of the force applied in point j

m_j is the j-th lumped mass

ϕ_{jh} is the modal displacement in point j for the h-th mode

$$H_h = [\omega_h^2 - \omega^2 + i * 2 * \xi_h * \omega_h * \omega]^{-1}$$

i is the imaginary unit

ω_h is the natural angular frequency of the h-th mode

ω is the angular frequency of the excitation

ξ_h is the damping ratio associated to the h-th mode

and with sinusoidal soil excitation

$$x_n = ({}_h\sum^N P_h * H_h * \phi_{nh}) * A_0 * \sin(\omega * t) + A_0 * \sin(\omega * t)$$

where

$P_h = ({}_j\sum^N m_j * \phi_{jh})/({}_j\sum^N \phi_{jh} * \phi_{jh} * m_j)$ is the modal participation coefficient for base excitation

A_0 is the displacement peak amplitude of the soil

$$H_h = \omega^2/[\ \omega_h^2 - \omega^2 + i * 2 * \xi_h * \omega_h * \omega\ ]$$

6.5.3 Modal parameters evaluation

The above reported modal analysis formulas can be simplified looking at the response in correspondence of each resonance frequency of the structure: in case of well separated resonance frequencies the structure behaves in correspondence of each one as a single degree of freedom system. In this case the absolute acceleration of each mass can be expressed in correspondence of the h-th resonance frequency in an approximate way by

in case of a single force excitation in point r

$$x_n = (\phi_{rh} * F_{0r} * \sin(\omega * t)) * (H_h * \phi_{nh})/({}_j\sum^N \phi_{jh} * \phi_{jh} * m_j)$$

and with sinusoidal soil excitation

$$x_n = (P_h * H_h * \phi_{nh}) * A_0 * \sin(\omega * t) + A_0 * \sin(\omega * t)$$

In case of forced excitation the experimental response of the structure in each measuring point can be expressed as:

$$x_n = X_{0n} * \sin(\omega * t) + XR_n(t)$$

where

X_{0n} is the peak value of the harmonic component of the response having the same frequency of the excitation

$XR_n(t)$ is the residual part of the response at other frequencies due to noise, non-linear behaviour or other excitations such as wind or operating machinery.

It is easy to see that:

$$X_{0n} = (\phi_{rh} * F_{0r}) * (H_h * \phi_{nh})/({}_j\sum^N \phi_{jh} * \phi_{jh} * m_j)$$

and thus the ratio between X_{0n} and F_{0r} gives a frequency response function through which it is possible to evaluate the natural frequencies of the structure and the associated damping ratios and modal shapes.

In case of sinusoidal soil excitation the experimental response of the structure in each measuring point can be expressed in a similar way, but in this case the relative motion should be evaluated. Namely it holds:

$$X_{0n} * \sin(\omega * t) - A_0 * \sin(\omega * t) = (P_h * H_h * \phi_{nh}) * A_0 * \sin(\omega * t)$$

where

X_{0n} is the peak value of the harmonic component of the absolute response having the same frequency of the excitation.

Thus with soil excitation it holds:

$$X_{0n} - A_0 = (P_h * H_h * \phi_{nh}) * A_0$$

and thus the ratio between (X_{0n} - A_0) and A_0 gives a frequency response function through which it is possible to evaluate the natural frequencies of the structure and the associated damping ratios and modal shapes.

In the case of soil excitation of a structure through explosions in the soil the above derived formula still holds, but X_{0n} and A_0 are respectively the Fourier transforms of the response in n-th point and of the soil acceleration.

6.5.4 Dynamic modulus matrix evaluation

In order to evaluate the characteristics of the soil a simplified model can be used where the structure is supported by the soil through a foundation acting with six degrees of freedom: three translations, two horizontal and one vertical, and three rotations, about the three orthogonal axes adopted to define the translations.

The relationship between the motion of the foundations and the reacting forces of the structure can then be written in the following way in the frequency domain:

$$\{ F_F(\omega) \} = [Z(\omega)] * \{ X_F(\omega) \}$$

where

$\{ F_F(\omega) \}$ is the general forces vector (three forces and three moments) acting on the foundation

$[Z(\omega)]$ is a 6 by 6 transfer matrix between displacements of the foundation and forces acting on the foundation (dynamic modulus)

$\{ X_F(\omega) \}$ is the general relative displacements vector (three displacements and three rotations) of the foundation.

In this case there are 36 unknowns, the elements of the dynamic modulus matrix, which can be reduced to 21 taking into account the symmetry of the matrix for elastic and linear systems.

To determine the unknowns is necessary to solve 21 equations which can be obtained measuring the foundation displacements and the forces of the structure in different tests.

The forces of the structure can be obtained by:

$$\{ g_F \} = [D] * \{ g_s \} - [B] * [M_{SS}] * \{ d^2q_A/dt^2 \}$$

where

$\{ g_F \}$ is the vector of the forces of the structure on the foundation in the time domain (6 elements vector)

$\{ g_s \}$ is the vector of the forces applied to the structure in the time domain (N elements vector)

$[M_{SS}] * \{ d^2q_A/dt^2 \}$ is the vector of the inertia forces of the structure

(just the part above the foundation) reacting to the excitation $\{ g_s \}$

$[D]$ is a topologic matrix which refers the forces applied to the structure to the reference point of the foundation motion (6 rows and N columns matrix)

$[B]$ is a topologic matrix which refers the inertia forces of the structure to the reference point of the foundation motion (6 rows and N columns matrix).

Through a Fourier transformation of the above cited equation, in stationary excitation condition, the following equation can be written:

$$\{ F_F(\omega) \} = [D] * \{ F_s(\omega) \} - [B] * [M_{SS}] * \{ X_S(\omega) \}$$

where

$\{ F_F(\omega) \}$ is the vector of the Fourier transforms of the forces acting on the foundation

$\{ F_s(\omega) \}$ is the vector of the Fourier transforms of the forces exciting the structure

$\{ X_S(\omega) \}$ is the vector of the Fourier transforms of the absolute accelerations of the structure.

In case of explosions excitation of the structure the forces of the structure on the foundation are given by:

$$\{ F_F(\omega) \} = - [B] * [M_{SS}] * \{ X_S(\omega) \}$$

and then the procedure for the evaluation of the dynamic modulus matrix is similar. In any case the relative displacements of the foundation should be considered.

6.5.5 Soil behaviour evaluation

6.5.5.1 Kinematic and inertial interaction evaluation

The equation

$$[M_p] * \{d^2q_A/dt^2\} + [C_p] * \{dq_A/dt\} + [K_p] * \{q_A\} = \{g_E\}$$

can be written as a set of n equations

$$[M_{11}] * \{d^2q_{A1}/dt^2\} + [C_{11}] * \{dq_{A1}/dt\} + [K_{11}] * \{q_{A1}\} =$$

$$= - [M_{1v}] * \{d^2q_{Av}/dt^2\} - [C_{1v}] * \{dq_{Av}/dt\} - [K_{1v}] * \{q_{Av}\} + \{g_s\}$$

and a set of m equations

$$[M_{v1}] * \{d^2q_{A1}/dt^2\} + [C_{v1}] * \{dq_{A1}/dt\} + [K_{v1}] * \{q_{A1}\} +$$

$$+ [M_{vv}] * \{d^2q_{Av}/dt^2\} + [C_{vv}] * \{dq_{Av}/dt\} + [K_{vv}] * \{q_{Av}\} =$$

$$= \{g_v\}$$

The total absolute displacements can be separated in two components: kinematic (k label) and inertial component (i label)

$\{q_{A1}\} = \{q_{A1}^k + q_{A1}^i\}$

Then the mass matrix $[M_{11}]$ is splitted into the component pertaining just to the structure and its foundation $[M_{11}^s]$ and to the near field soil $[M_{11}^g]$:

$$[M_{11}] = [M_{11}^s] + [M_{11}^g]$$

Then it is possible to write:

$$[M_{11}^g] * \{d^2q_{A1}^k/dt^2\} + [C_{11}] * \{dq_{A1}^k/dt\} + [K_{11}] * \{q_{A1}^k\} =$$

$$= - [M_{1v}] * \{d^2q_{Av}/dt^2\} - [C_{1v}] * \{dq_{Av}/dt\} - [K_{1v}] * \{q_{Av}\} + \{g_s\}$$

which gives the kinematic interaction motion and

$$[M_{11}] * \{d^2q_{A1}^i/dt^2\} + [C_{11}] * \{dq_{A1}^i/dt\} + [K_{11}] * \{q_{A1}^i\} =$$

$$= - [M_{11}^s] * \{d^2q_{A1}^k/dt^2\}$$

which gives the inertial interaction motion.

6.5.5.2 Soil parameters evaluation

For a single degree of freedom system the dynamic modulus is given by:

$$F(\omega)/X(\omega) = k - m * \omega^2 + i * \omega * c$$

with trivial meaning of the symbols. Then reporting graphically the real and imaginary parts of the ratio between applied force and displacement two functions are found: the real part following a parabolic law versus frequency which allow the determination of the stiffness and the imaginary part following a linear law versus the frequency which allow the determination of the damping coefficient.

For a multi degrees of freedom system, i. e. a six degrees of freedom foundation, the equilibrium equation becomes:

$[M_F] * \{d^2q_A/dt^2\} + [C_F] * \{dq_A/dt\} + [K_F] * \{q_A\} = \{g_F\}$

and with a Fourier transformation:

$(- \omega^2 * [M_F] + i * \omega * [C_F] + [K_F]) * \{X_F\} = \{F_F\}$

Also in this case use should be made of the decoupling modal analysis procedure and considering the response of the foundation in correspondence of each natural frequency, with no coupling between adjacent frequencies, it holds:

$F_{Fhr} / X_{Fhn} = ({}_j\sum^6 \phi_{hj} * \phi_{hj} * m_j)/(\phi_{hn} * H_h * \phi_{hr}) =$

$= G_{hrn} * 1 / H_h$

where

G_{hrn} is depending upon the natural frequency of the h-th mode, the r-th "force" acting on the foundation and the response of the n-th degree of freedom of the foundation

$1 / H_h = [\omega_h^2 - \omega^2 + i * 2 * \xi_h * \omega_h * \omega]$.

Thus it is possible to obtain the value of the soil parameters, stiffness and damping coefficient, through the following transformation:

$F_{Fhr} / X_{Fhn} = k_{rhn} - m_{rhn} * \omega^2 + i * \omega * c_{rhn}$

where

$k_{rhn} = G_{hrn} * \omega_h^2$

$m_{rhn} = G_{hrn}$

$c_{rhn} = G_{hrn} * 2 * \omega_h * \xi_h$

6.5.6 Final remarks

As it can be easily seen the experimental evaluation of the soil-structure interaction is a complex problem; the problem can be simplified in practice if through the analysis of the experimental data some assumptions can be made:

a) the behaviour of the foundation should be separatly studied in each direction verifying that the different rigid body modes are decoupled: this means that each mode can be regarded as a single degree of freedom system mode;

b) the behaviour of the structure and of the soil should be linear: this allows to apply the modal analysis procedures;

c) the foundation should be considered as a rigid body (no elastic mode in the low frequency range): the foundation can be modelled with a six degrees of freedom system;

d) the structural damping of the building should be taken as proportional to stiffness and mass: this enables the use of the decoupling formulas of the modal analysis theory.

In practice quite none of the above cited hypotheses are verified, but a solution of the problem can yet be searched following the exposed theory, noting that the more the effective behaviour of the structure is far from the simplifying hypotheses, the more the results of the data processing are approximate.

For what concerns the choice of the excitation type the following remarks should be considered:

a) the excitation through a sinusoidal force is desirable, because it is more easy to control the energy distribution in the frequency domain and tha data processing for the estimation of the dynamic modulus of the soil is more easy and reliable; but it is very difficult to generate big forces in the low frequency range which is the most interesting for the soil-structure interaction evaluation;

b) the excitation through the foundation with explosion is very similar to the seismic excitation, but it is very difficult to control the frequency distribution of the released energy: very carefull data processing should be performed.

7. CONCLUSIONS

Within the frame of the IAEA benchmarck study for the seismic analysis and testing of an existing NPP, the report issued by Paks NPP on the explosion tests performed on buildings and structures of Paks NPP has been reviewed.

In particular, attention has been paid to the procedure to compute response spectra and dynamic amplification in the different measurement positions, giving some comments based on previous ISMES experience on similar jobs.

With minor implementation on the aspects suggested in the review report, it can be considered that the report on explosion tests can represent in a sufficient way, a preliminary study on the dynamic behaviour of some structures of the plant. An improvement of the item could be represented by other experimental tests, with a larger number of measurement transducers, located in other positions of the plant, in order to detect, through explosion tests, the dynamic behaviour of selected mechanical components, electrical equipment and some civil structures safety related.

8. FIGURE LISTS

series	load [kg]	date	distance[m]	direction
T/700	10	02.11.88	700	W
T/700	20	02.11.88	700	W
T/700	20	02.11.88	700	W
T/780	10	02.11.88	780	W
T/780	20	02.11.88	780	W
T/780	40	02.11.88	780	W
T/700	20	03.11.88	700	W
T/700	2*20+	03.11.88	700	W
T/700	20	03.11.88	700	W
T/700	3*20+	04.11.88	750	W
T/740	3*20+	04.11.88	740	W
I/2300	50	19.11.90	2300	SSE
I/2300	100	20.11.90	2300	SSE
I/2300	200	20.11.90	2300	SSE
I/2300	200	21.11.90	2300	SSE
I/4500	200	22.11.90	4500	SE
I/4500	500	22.11.90	4500	SE
I/4500	500	23.11.90	4500	SE
II/2100	100	19.11.91	2100	E
II/2100	300	19.11.91	2100	E
II/2100	300	20.11.91	2100	E
II/3200	450	20.11.91	3200	S
II/3200	450	21.11.91	3200	S

+ Consecutive explosions, with one second intervals

FIG. 1

LISTS OF THE MEASUREMENTS POSITIONS FOR EACH TEST ANALYZED IN THE REPORT [1]

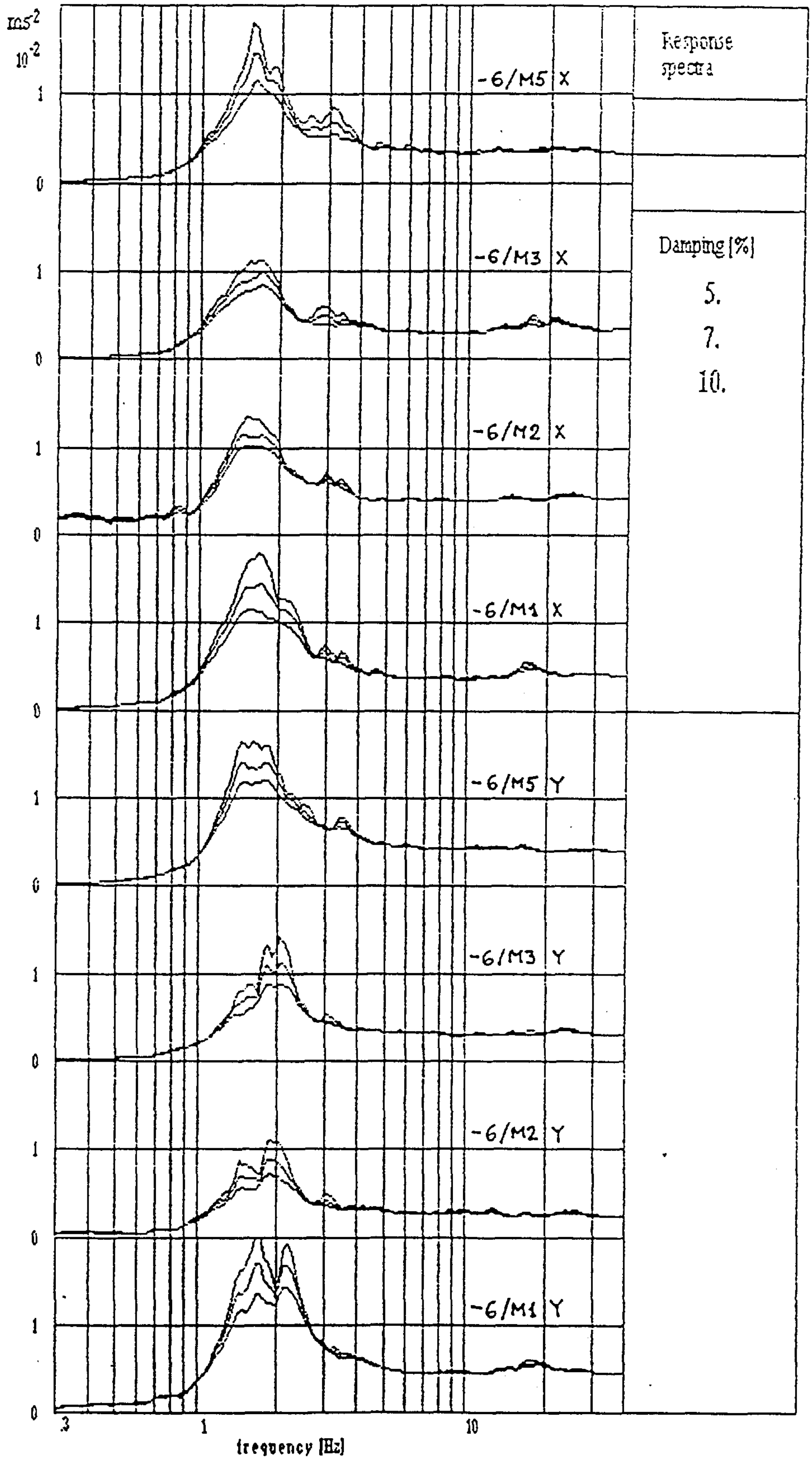
TEST	BUILDING	ELEVATION (mm)	MEASUREMENT POSITION	SENSITIVITY DIRECTION OF TRANSDUCERS
I/4500	MAIN	-6,5	-6/M1,M2,M3,M5	X ,Y
	MAIN	0	0/M1,M2,M4,M5	X ,Y
	MAIN	6	6/M1÷ M5	X ,Y
	MAIN	18,9	18/M1÷ M5	X ,Y
	MAIN	22	22/M1,M2	X ,Y
	MAIN	27	27/M	X ,Y
	MAIN	32	32/M	X ,Y
	MAIN	38	38/M	X ,Y
	FREE FIELD	0	K4	X , Y , Z
	FREE FIELD	0	K7	X , Z
II/2100	MAIN GALLERY	-3	- 3/MG	X ,Y,Z
	MAIN GALLERY	18	18/MG	X ,Y,Z
	MAIN GALLERY	38	38/MG	X ,Y,Z
	TURBINE	0	K4	X,Y,Z
	TURBINE	0	K7	X,Y,Z
	GALLERY	-3	- 3/G	X ,Y,Z
	GALLERY	9	9/G	X ,Y,Z
	GALLERY	19	19/G	X ,Y,Z
	GALLERY	48	48/G	X ,Y,Z
	FREE FIELD	0	K3	X ,Y,Z
	FREE FIELD	0	0/K4	X ,,Z
	FREE FIELD	0	0/K5	X
	DOWN HOLE	-5	-5/K4,K5	X,Y,Z
	DOWN HOLE	-10	-10/K4	X,Z
	DOWN HOLE	-10	-10/K3	Y,Z
	DOWN HOLE	-20	-20/K3	X
	DOWN HOLE	-20	-20/K4	X,Y
	DOWN HOLE	-15	-15/K3,K4,K5	Y
II/3200	MAIN GALLERY	-3	-3/MG	X ,Y,Z
	MAIN GALLERY	18	18/MG	X ,Y,Z
	MAIN GALLERY	38	38/MG	X ,Y,Z
	TURBINE	0	0/T1,T2	X,Y,Z
	TURBINE	18	18/T1,T2	X,Y,Z
	GALLERY	-3	- 3/G	X ,Y,Z
	GALLERY	9	9/G	X ,Y,Z
	GALLERY	19	19/G	X ,Y,Z
	GALLERY	48	48/G	X ,Y,Z
	FREE FIELD	0	K3	X,Z
	FREE FIELD	0	0/K4,K5	X,Y,Z
	DOWN HOLE	-5	-5/K3,K4,K5	X
	DOWN HOLE	-10	-10/K4	X
	DOWN HOLE	-20	-20/K3	X

FIG. 2

Frequency range (Hz)	Increment (Hz)
0.2 - 3.0	0.10
3.0 - 3.6	0.15
3.6 - 5.0	0.20
5.0 - 8.0	0.25
8.0 - 15.0	0.50
15.0 - 18.0	1.0
18.0 - 22.0	2.0
22.0 - 34.0	3.0

Suggested frequency intervals for calculation of response spectra.

FIG. 3



I/4500. Response spectra at level -6.50 m.

FIG. 4

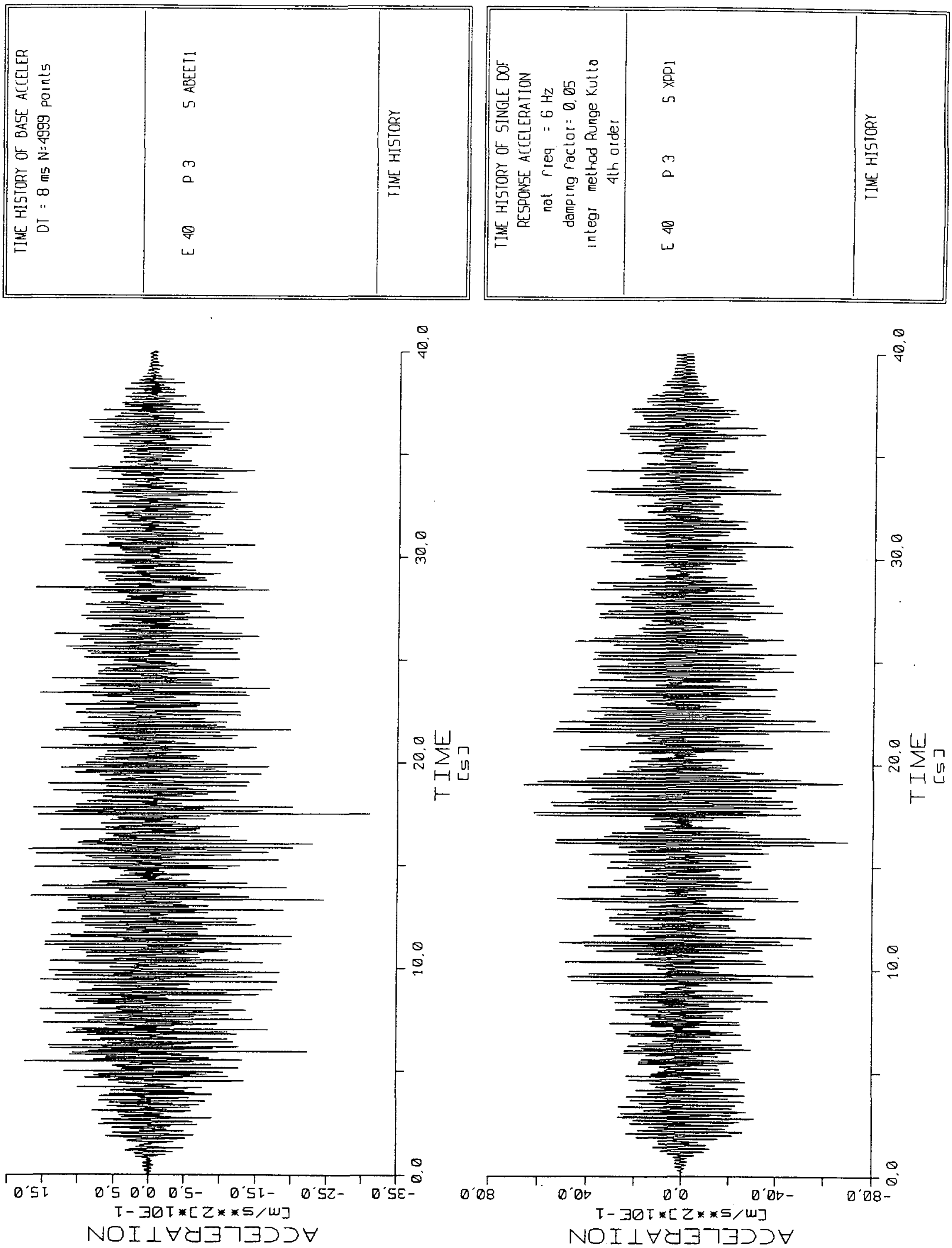

TIME HISTORY OF BASE ACCELER
DT : 8 ms N:4999 points
E 40 P 3 5 ABEET1
TIME HISTORY
TIME HISTORY OF SINGLE DOF
RESPONSE ACCELERATION
nat freq : 6 Hz
damping factor: 0.05
integr method Runge Kutta
4th order
E 40 P 3 5 XPP1
TIME HISTORY
TIME
[s]
ACCELERATION
[m/s**2]*10E-1
0.0
10.0
20.0
30.0
40.0
15.0
5.0
0.0
-5.0
-15.0
-25.0
-35.0
80.0
40.0
-40.0
-80.0

FIG. 5

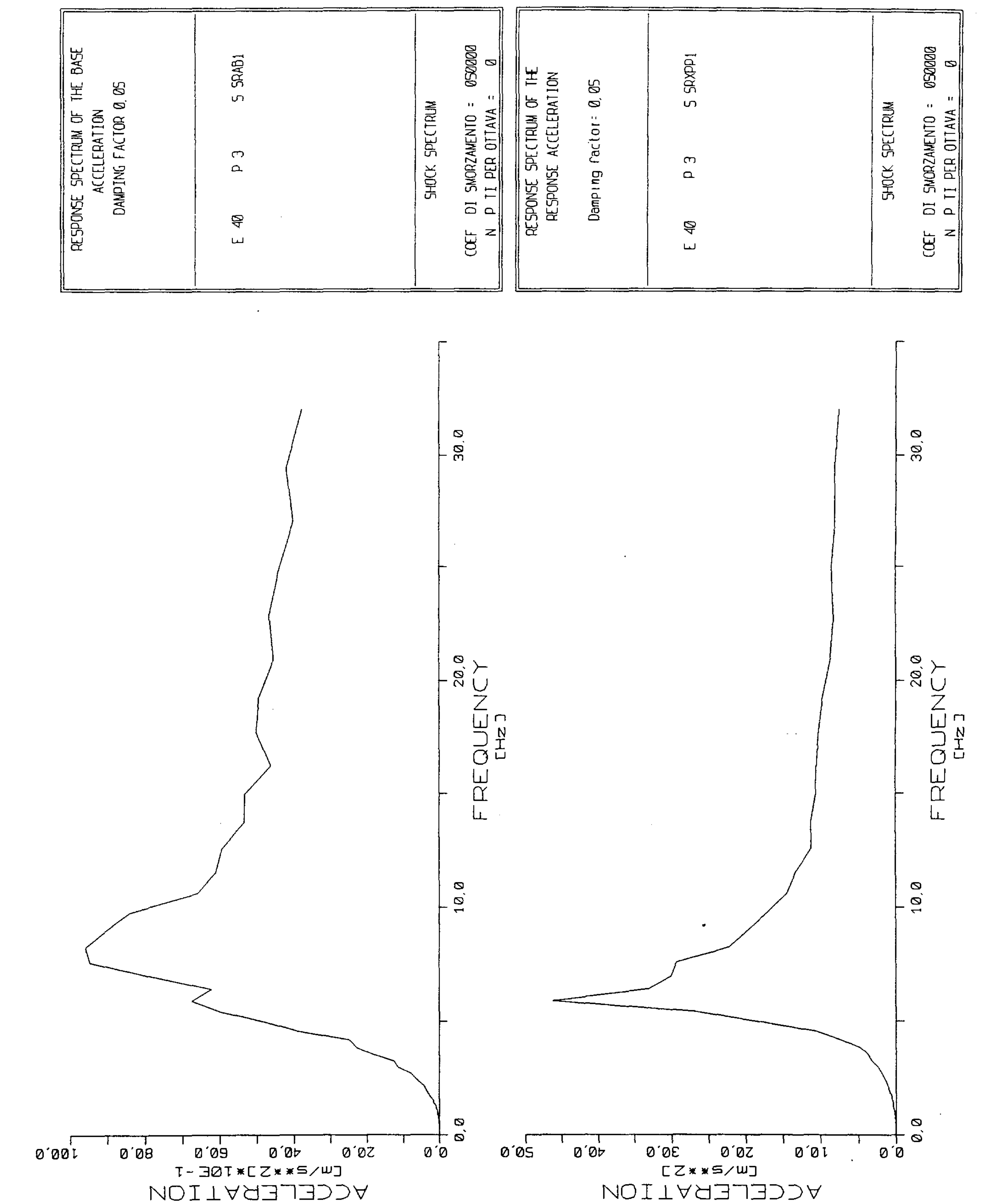
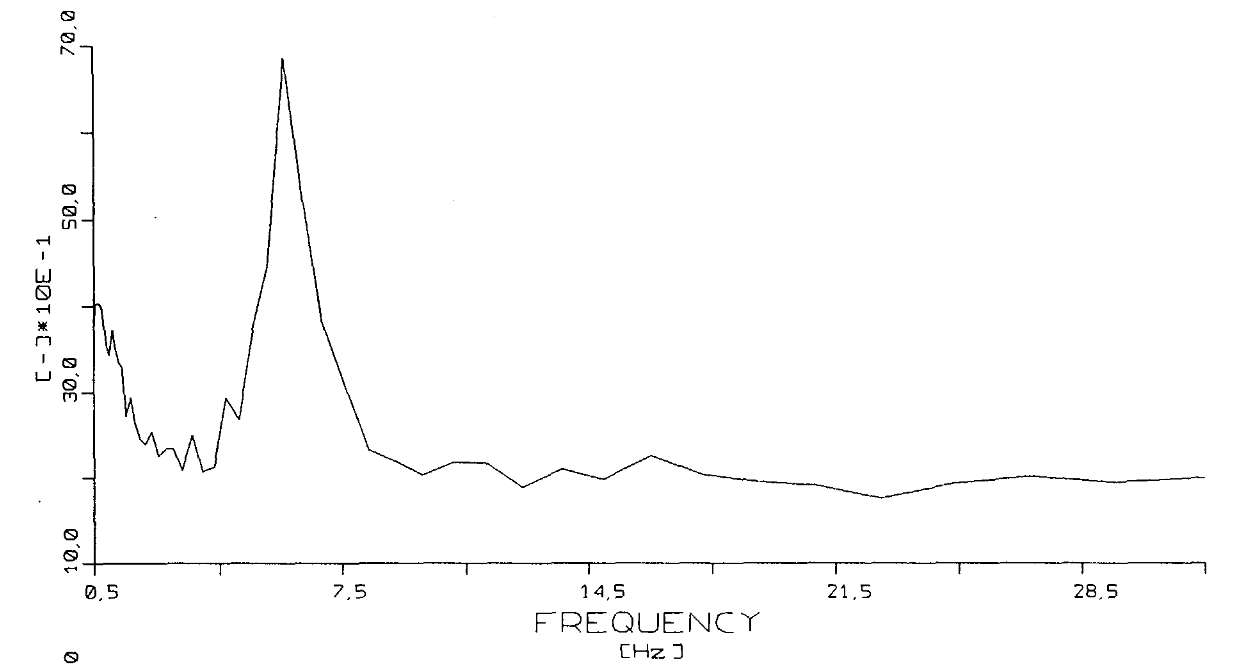

RESPONSE SPECTRUM OF THE BASE
ACCELERATION
DAMPING FACTOR 0,05
E 40 P 3 S SRAB1
SHOCK SPECTRUM
COEF DI SMORZAMENTO = 050000
N P TI PER OTTAVA = 0
ACCELERATION
[m/s**2]*10E-1
FREQUENCY
[Hz]
RESPONSE SPECTRUM OF THE
RESPONSE ACCELERATION
Damping factor: 0,05
E 40 P 3 S SRXPP1
SHOCK SPECTRUM
COEF DI SMORZAMENTO = 050000
N P TI PER OTTAVA = 0
ACCELERATION
[m/s**2]
FREQUENCY
[Hz]

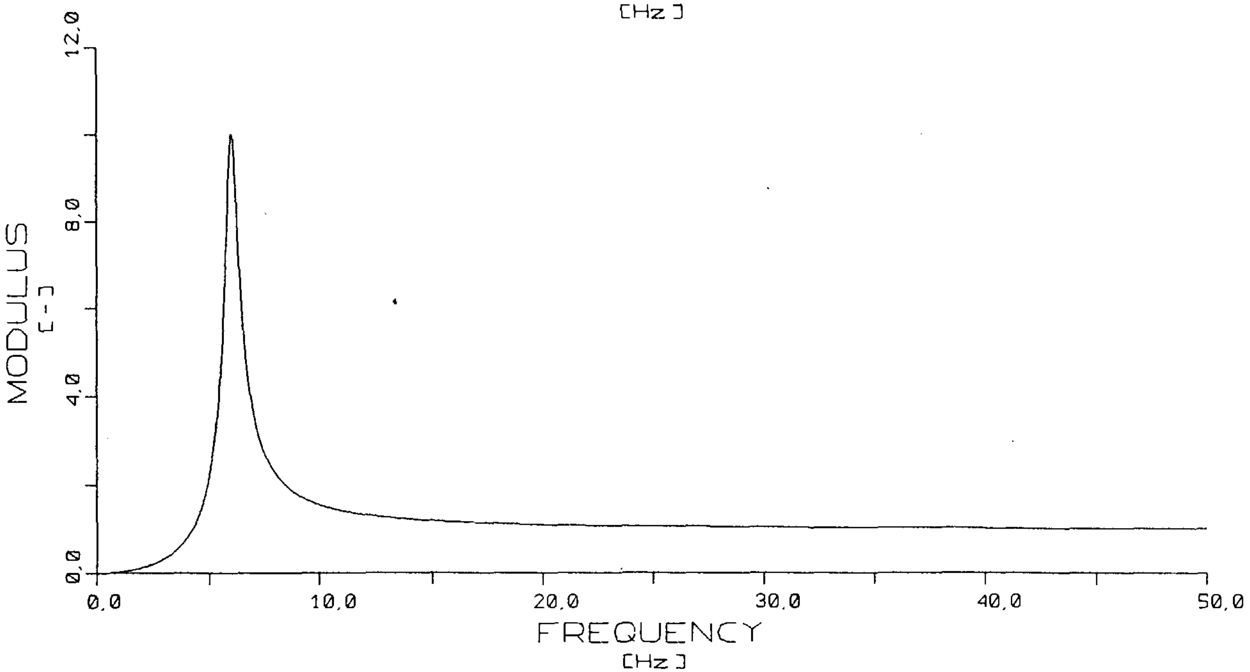
FIG. 6



RATIO BETWEEN THE RESPONSE SPECTRUM OF RESP ACCELERATION AND THE RESPONSE SPECTRUM OF BASE ACCELERATION DAMPING FACTOR 0,05

E 40 P 3 S AMPSR1

SHOCK SPECTRUM



ANALYTICAL TRANSFER FUNCTION OF SINGLE DOF SYSTEM NAT FREQ 6 Hz DAMPING FACTOR 0,05

E PAKS1 P 1 S 9

FIG. 7

ISMES

REWIEW REPORT ON THE DYNAMICAL STUDY OF THE MAIN BUILDING OF THE PAKS NPP

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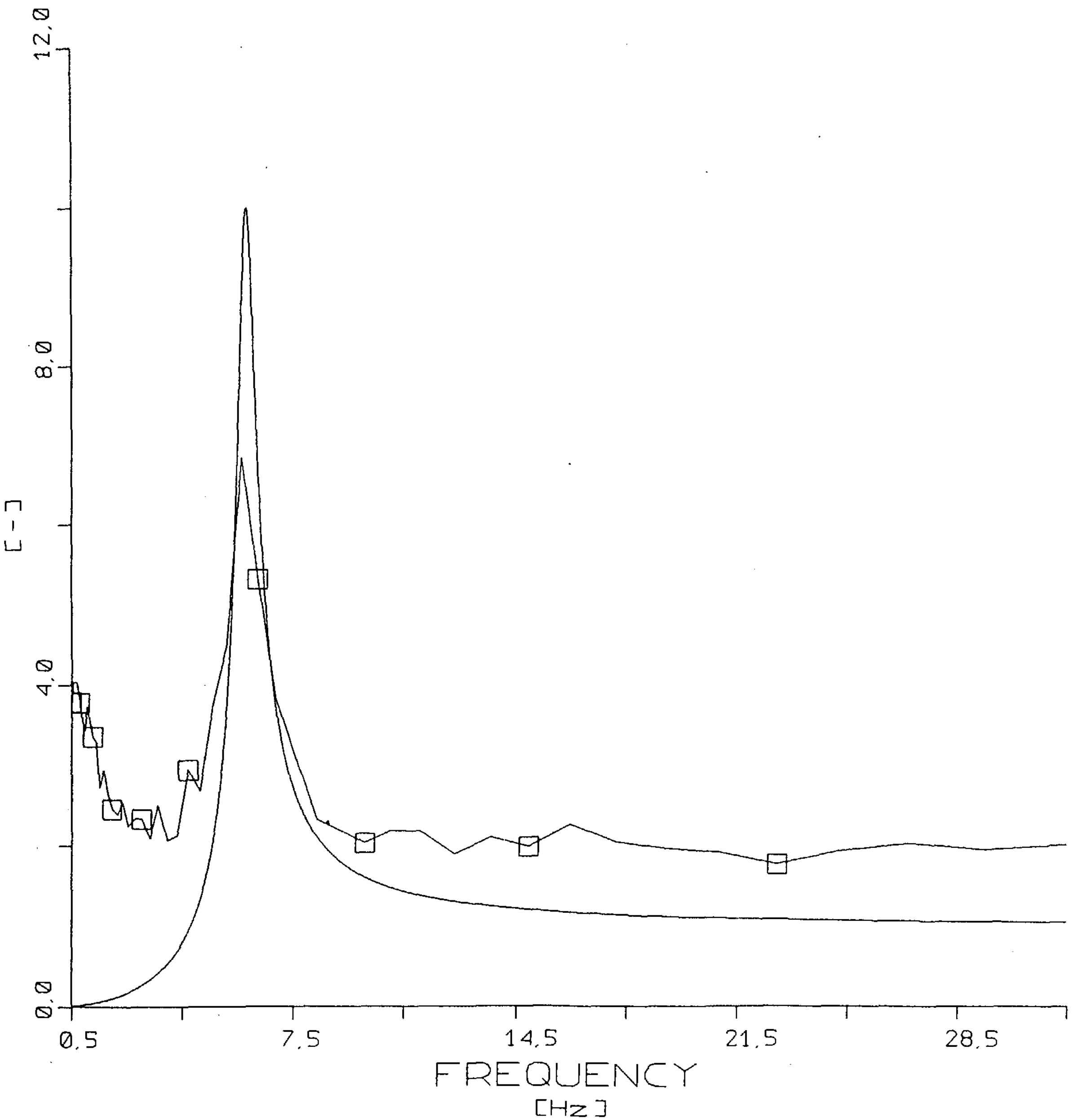
Rev. 00



COMPARISON BETWEEN THE RATIO
RESP ACCELER /BASE ACCELER
RESPONSE SPECTRA (AMPSR1)
AND THE ANALYTICAL TRANSFER
FUNCTION OF SDOF (M9)

E 40	P 3	S AMPSR1
E PAKS1	P 1	S M9

FIG. 8

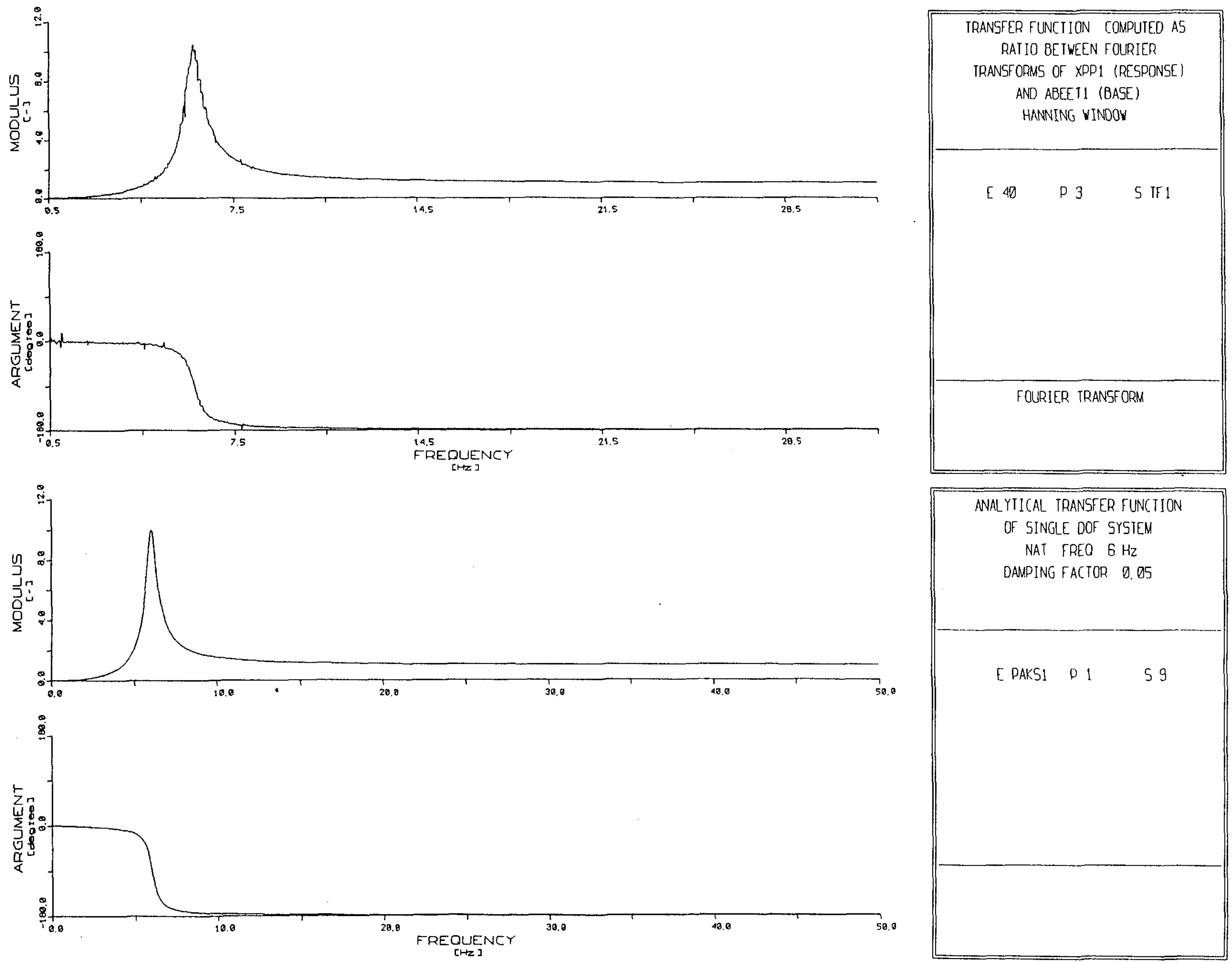
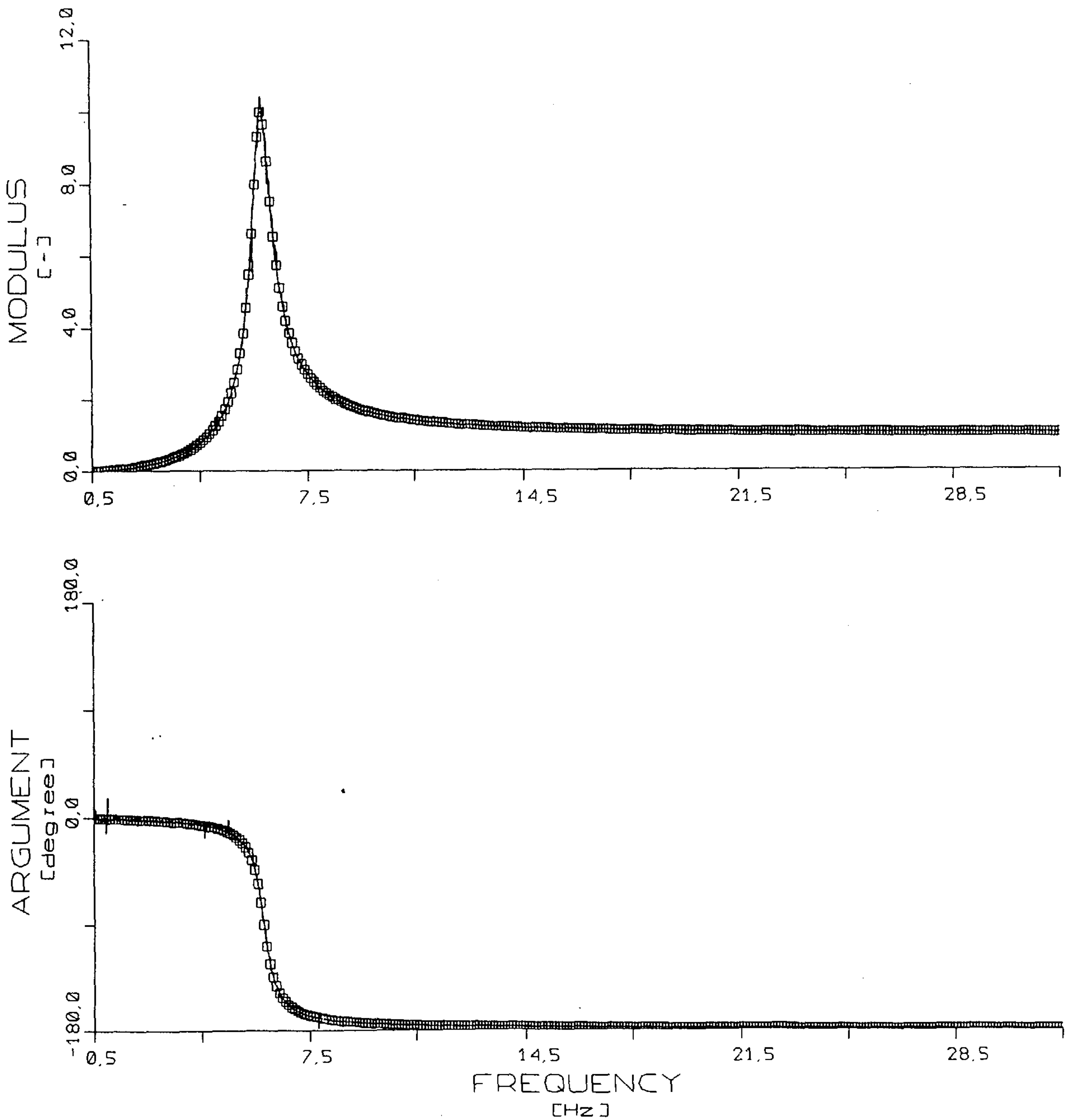

MODULUS [-]
ARGUMENT [degrees]
FREQUENCY [Hz]
TRANSFER FUNCTION COMPUTED AS RATIO BETWEEN FOURIER TRANSFORMS OF XPP1 (RESPONSE) AND ABEET1 (BASE) HANNING WINDOW
E 40 P 3 S TF1
FOURIER TRANSFORM
MODULUS [-]
ARGUMENT [degrees]
FREQUENCY [Hz]
ANALYTICAL TRANSFER FUNCTION OF SINGLE DOF SYSTEM NAT FREQ 6 Hz DAMPING FACTOR 0.05
E PAKS1 P 1 S 9

FIG. 9



COMPARISON BETWEEN TRANSFER FUNCTIONS TF1 (COMPUTED ON FOURIER TRANSFORMS) AND PAKS1/1/9 (ANALYTICAL)

E 40	P 3	S TF1
E PAKS1	P 1	S 9

FOURIER TRANSFORM

REWIEW REPORT ON THE DYNAMICAL STUDY OF THE MAIN BUILDING OF THE PAKS NPP

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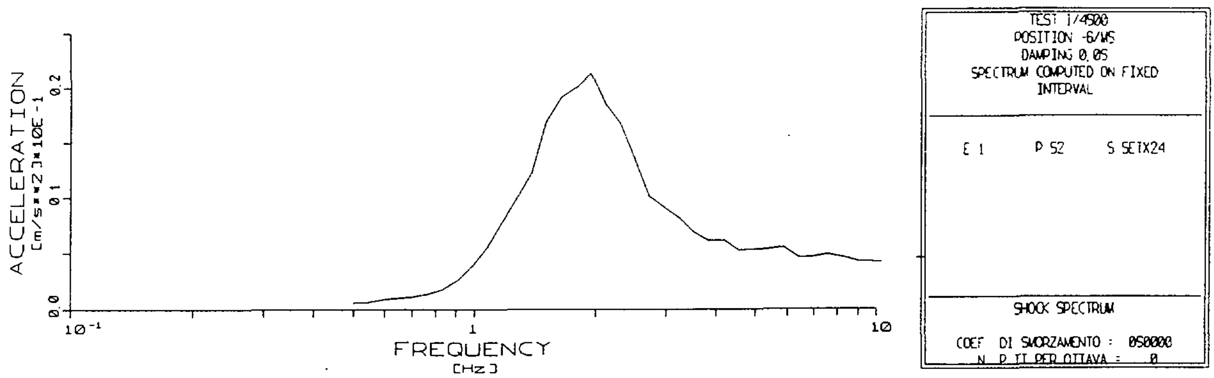
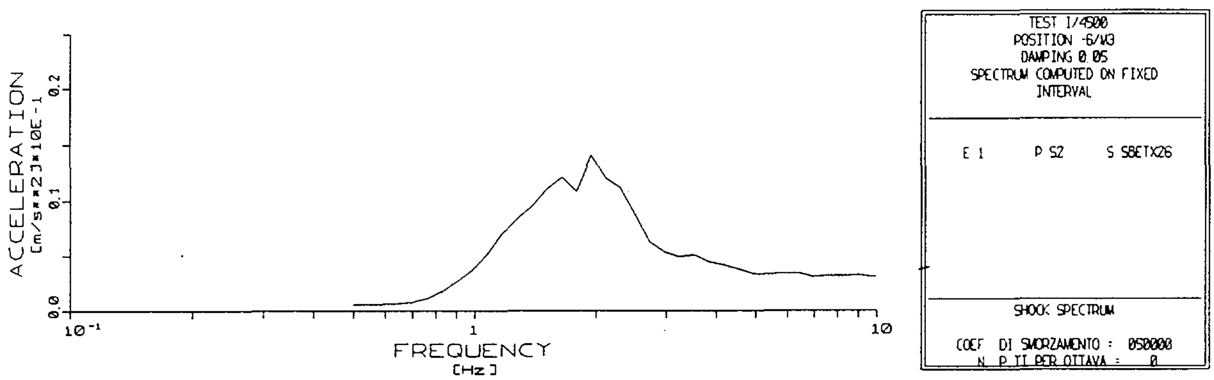
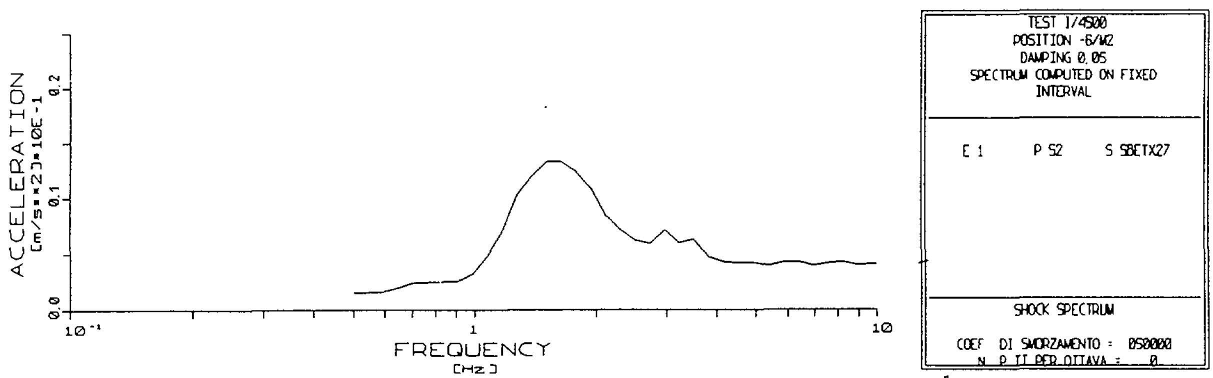
FIG. 10


ACCELERATION
[m/s**2]*10E-1
0.0
0.1
0.2
10^{-1}
1
10
FREQUENCY
[Hz]
TEST 1/4500
POSITION -6/W5
DAMPING 0.05
SPECTRUM COMPUTED ON FIXED INTERVAL
E 1 P 52 S SETX24
SHOCK SPECTRUM
COEF DI SMORZAMENTO : 050000
N P.TI PER OTTAVA : 0


ACCELERATION
[m/s**2]*10E-1
0.0
0.1
0.2
10^{-1}
1
10
FREQUENCY
[Hz]
TEST 1/4500
POSITION -6/W3
DAMPING 0.05
SPECTRUM COMPUTED ON FIXED INTERVAL
E 1 P 52 S SBETX26
SHOCK SPECTRUM
COEF DI SMORZAMENTO : 050000
N P.TI PER OTTAVA : 0


ACCELERATION
[m/s**2]*10E-1
0.0
0.1
0.2
10^{-1}
1
10
FREQUENCY
[Hz]
TEST 1/4500
POSITION -6/W2
DAMPING 0.05
SPECTRUM COMPUTED ON FIXED INTERVAL
E 1 P 52 S SBETX27
SHOCK SPECTRUM
COEF DI SMORZAMENTO : 050000
N P.TI PER OTTAVA : 0

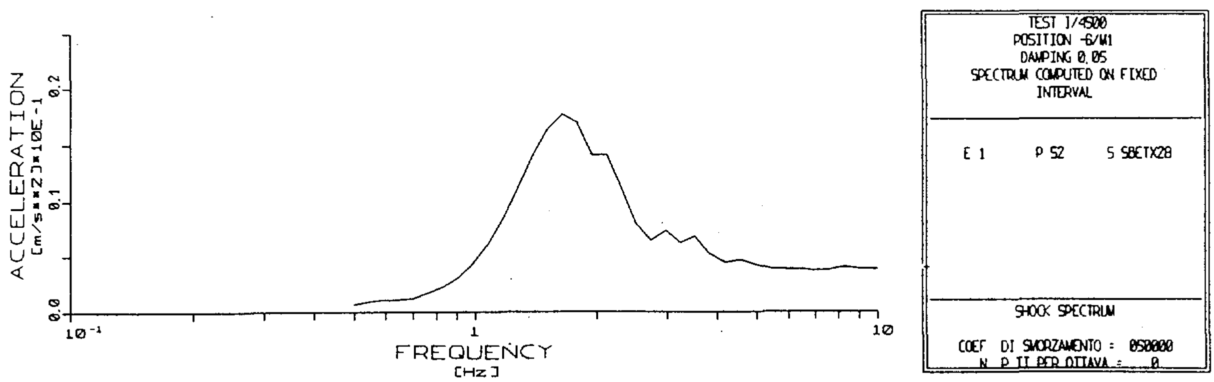

ACCELERATION
[m/s**2]*10E-1
0.0
0.1
0.2
10^{-1}
1
10
FREQUENCY
[Hz]
TEST 1/4500
POSITION -6/W1
DAMPING 0.05
SPECTRUM COMPUTED ON FIXED INTERVAL
E 1 P 52 S SBETX28
SHOCK SPECTRUM
COEF DI SMORZAMENTO : 050000
N P.TI PER OTTAVA : 0

FIG. 11