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Contributor: **P. Contri (Ismes)**

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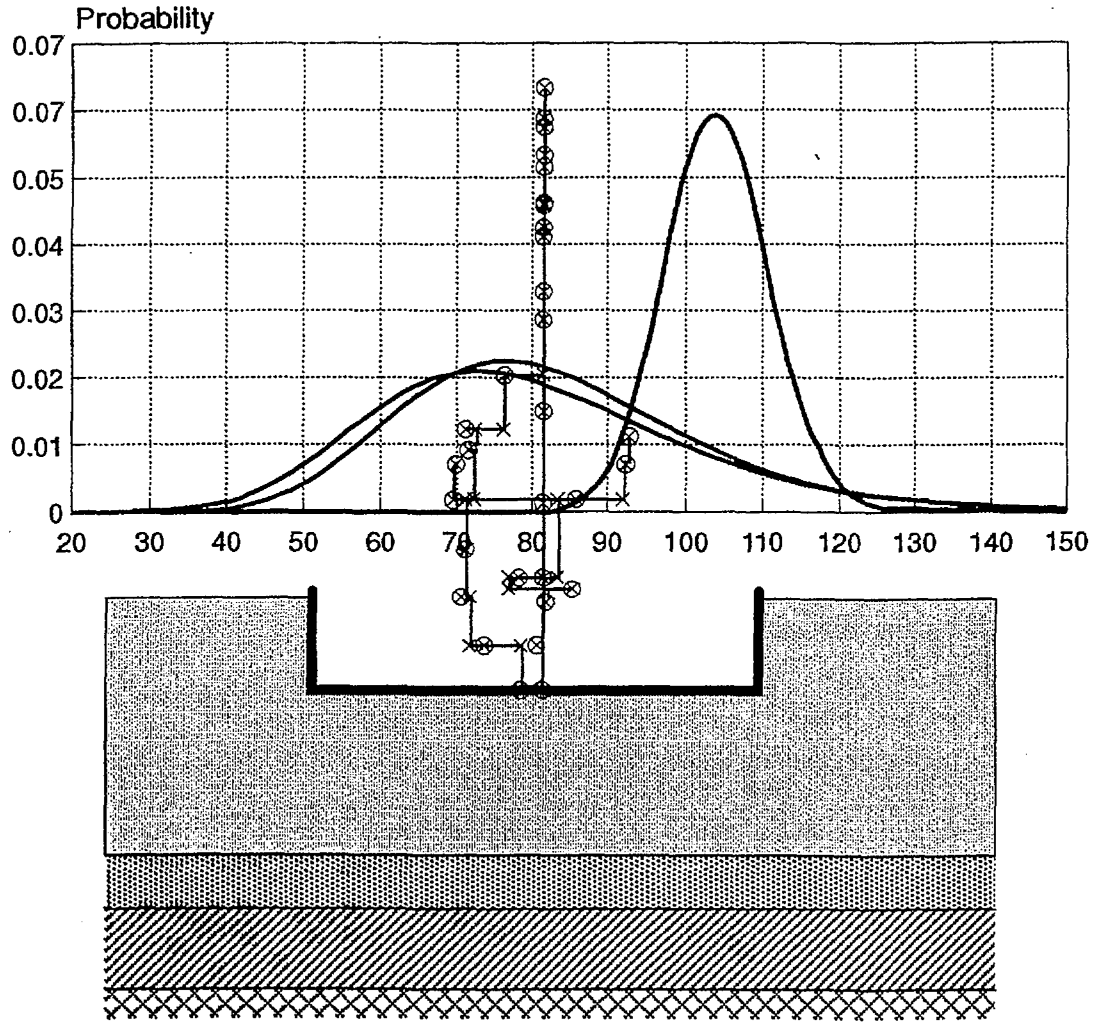
OPTIMAL ORGANIZATION OF STRUCTURAL ANALYSIS AND SITE INSPECTION FOR THE SEISMIC REQUALIFICATION OF THE NUCLEAR POWER PLANT OF PAKS, HUNGARY

MAY 1995



Prepared for:

The International Atomic Energy Agency

ismes spa
viale G. Cesare 29, 24124 Bergamo
tel. +39-35/307.111 fax +39-35/21.11.91
Seriate: via Pastrengo 9, 24068 Seriate BG • *Milano*: Via S.Sandri 2, 20121 Milano • *Roma*: via dei Crociferi 44, 00187 Roma

OPTIMAL ORGANIZATION OF STRUCTURAL ANALYSIS AND SITE INSPECTION FOR THE SEISMIC REQUALIFICATION OF THE NUCLEAR POWER PLANT OF PAKS, HUNGARY

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SUMMARY

1. INTRODUCTION

The analysis described in this report deals with a numerical procedure aimed at the assessment of a methodology for the optimal organization of data collection,in a contest of seismic requalification of structures and components of existing nuclear power stations (NPPs).

The activity has been carried out in the frame of IAEA benchmark study for the seismic analysis of existing Nuclear Power Plants.

This study starts from the assumption that seismic qualification of existing NPPs usually has to be carried out even in lack of sufficient data on structural behaviour and site conditions.

In this framework, the organization of the analysis possibly requires a special approach, based on reliability analysis, able to give the distributions of dependent structural variables. This result can in fact be used in iterative updating of the analysis, leading at last at a required uncertainty target level for the structural evaluation.

Therefore, the global uncertainty can be reduced by the reduction of the uncertainties of the variables that affect most the structural behaviour: the proposed procedure is able to drive this process in an optimal way.

The analysis manager can therefore organize additional experimental inspections (for example in geotechnics, geophysics, structural behaviour)and data collections with the confidence of a minimum effort required for the prescribed target in terms of seismic safety.

The procedure presented in this report has quite a general application following the general description provided in [14];therefore the example test has been chosen for the Paks NPP in Hungary, where a seismic requalification is in progress.

To this aim, in the following specific reference will be made to the variables of interest for the on going job, namely:

- the probability distribution of some structural parameters, such as acceleration or shear force in critical points, giving a global overview on the reliability of structural calculations;
- the sensitivity coefficient table giving the influence of the uncertainty related to input data on the cited structural parameters, useful to drive data collection for global uncertainty reduction.

The assessment has been carried out with reference to the following main tasks:

- structure and soil data analysis;
- numerical model generation;
- deterministic dynamic analysis description;
- reliability analysis framework discussion;
- transfer function calculation via response surface approach;
- sensitivities evaluation.

2. MODEL CONFIGURATION

2.1 Geometry

The building considered in the analysis is the reactor building of Paks NPP (fig.0), consisting of four main parts:

- Reactor Building
- Condensing Tower
- Galleries
- Turbine Hall

The stick model (figg.1-2) has been provided by Siemens AG, exploiting previous validation on experimental testing.

The model has 50 dynamic degrees of freedom, and is made up by 28 nodal points, 12 beam elements and 12 concentrated masses.

The total weight of the model is approximately 1.500.000 kN.

2.2 Soil properties

According to experimental data collected in Siemens report [6], the soil can be described as an horizontally layered, viscoelastic half-space, every of five layers having different properties evaluated on the base of site inspections (as shown in tab. 1) in terms of thickness, density and shear modulus (or corresponding shear wave velocity, such as these two variables are linked through the law:

$$G = \rho \cdot V_s^2$$

where G is the shear modulus, and V_s the shear wave velocity).

Water table is assumed at ground level.

Layer	Thickness (m)	Density (kN/m^3)	Shear modulus (kN/m^2)
1	9.5	19.62	50.000
2	10.5	19.62	50.000
3	7.0	19.62	100.000
4	83.0	21.58	350.000
5	400.0	21.58	450.000

Tab. 1

These values are referred to the mean profile of the corresponding random variables. Later, in this report, the full random variability will be explicitly taken into account in the reliability analysis.

To investigate dynamic response of the site, a schematization of the interaction between soil and structure has been considered: the soil has been modelled by six discrete couples of springs and dampers (one for each degree of freedom).

Equivalent springs and damping values (or so called impedances, by analogy with electrotechnology) have been derived from soil properties through a specific computer program, SOILFLEX [1].

The theory implemented in this program starts from the assumption that, under a certain load, layered foundation soil stiffness is proportional to the total strain energy, and this energy can be evaluated as if the distribution of stresses within every layer is equal to the distribution of an elastic half-space with the same properties.

Due to non-linear behaviour of the soil, shear modulus decreases, and material damping increases, with increasing strains. The relationships between these variables (shear modulus vs. strain and damping vs. strain) can vary from site to site, and have to be determined by site inspections. In lack of such investigations, standard curve suggested by Seed and Idriss [2] (fig. 3) has been used for shear modulus, and a constant mean value for soil hystheretic damping, has been assumed.

A reduction factor of 30% for traslational components of radiation damping and 50% for rotational components has been applied, to consider the trapping effect for shear waves, due to the significant increase of shear modulus descending from 2nd to 3rd layer (see tab. 1).

For a reference mean value of horizzontal acceleration of 0.3g (see § 3.3) the resulting global impedances and radiation damping ratios are shown below:

Direction	SOILFLEX		SIEMENS	
	Stiffness	Radiation damping ratio (%)	Stiffness	Radiation damping ratio (%)
x	13.47 E +06	18.79	17.E+06	27
y	13.50 E +06	18.79	17.E+06	27
z	50.92 E +06	32.91	50.E+06	49
xx	33.25 E +09	13.08	38.E+09	12
yy	34.80 E +09	13.58	37.E+09	12
zz	20.85 E +09	12.01	20.E+09	15

Tab. 2

These assumptions are the main responsible of the differences between ISMES and Siemens values, giving a preliminary idea of the sensitivity of the results to the definition of geotechnical parameters.

2.3 Damping

Two soil damping coefficients have been used for modal damping procedure (see § 3.2), one for each kind of damping: hysteretic and radiative. For the former a constant value of 7.4 % (valid for sand and strains compatible with seismic excitation, [2]) has been assumed, for the latter the value assumed is variable with the direction, and evaluated as described at the previous paragraph.

With respect to the structure, an hysteretic damping of 5 % (referred to critical damping) has been considered ($\beta_{ave.}$).

2.4 Material properties

From the material point of view the building has four main parts:

1) Reactor building and Condensing Tower: composed by monolithic concrete walls and floors;

2) Reactor Hall: concrete walls and floors covered by a frame steel structure;

3) Galleries: composed by steel frames

4) Turbine Hall: also composed by steel frames.

The following material properties have been adopted for the stick model:

- concrete: $E = 31000\ N/mm^2$ $\nu = 0.2$
- steel: $E = 206000\ N/mm^2$ $\nu = 0.3$

2.5 Seismic load

Seismic load has been assumed as a function of general damping coefficient of soil-structure interaction.

With respect to the Regulatory Guide 1.60 of U.S. Atomic Energy Commission [3], the amplification factor of input acceleration spectrum has to be evaluated with reference to the modal damping approach: the damping coefficient to be used for each mode shape falls in the range between the structural and the soil value, according with the strain energy content of soil and structure for each mode.

An extrapolation of the amplification factors till a 20 % damping coefficient is therefore necessary starting from spectrum provided in the Reg. Guide: it is based upon a linear regression (using least squares method) on a semi-logarithmic graph. This extrapolation is made for five reference frequencies, corresponding to the points where response spectra change its slope; the points are then connected with a straight line on the logarithmic graph.

3. DYNAMIC ANALYSIS: GENERAL PROCEDURE

3.1 Eigenvalue extraction

As first step of dynamic analysis, a modal analysis has been performed to calculate system eigenvalues and eigenvectors and corresponding frequencies, taking all the frequencies lower of 100 Hz.

The frequency of the first horizontal and vertical modes of vibration are respectively 1.32 Hz and 2.52 Hz, with a reasonable agreement to the values found by SIEMENS (1.35 Hz and 2.00 Hz) in a previous work [4], [5], [6] on the same structure, but with some differences in equivalent soil stiffness.

Modal participation factors for each frequency have also been evaluated and presented in fig. 4. The first 6 frequencies involve more than 99% of total mass.

Mode shapes for the more relevant modes of vibration have been determined, and plotted in figg. 5÷7.

3.2 Modal damping procedure

A general damping coefficient has been assumed with a variable contribution of structure and soil. The global equivalent damping (ξ) has been evaluated by the Roesset method [7], which gives a weight to each damping coefficient as function of the strain energy of each element of the system for a particular mode, and of the total energy of that mode:

$$\xi_i = \frac{\beta E_i^S + \gamma E_i^T + \delta\omega_i \sum_j \frac{E_{ij}^T}{\omega_j}}{E_i^S + E_i^T}$$

β hysteretic damping coefficient of the structure

γ hysteretic damping coefficient of the soil

δ radiation damping coefficient of the soil

E_i^S strain energy of the structure in the *i*-th mode

E_i^T strain energy of the soil in the *i*-th mode

E_{ij}^T strain energy of the j-th spring (soil model) in the *i*-th mode

ω_i frequency of the *i*-th mode

ω_j reference frequency for damping in the *j*-th spring, assuming the structure as a rigid body

Maximum damping is assumed = 20%.

In the diagram of fig. 8, for each normal mode of the structure, the strain energies values are represented (in percentage with respect to the total energy) splitting in soil and structure contributions.

The main part of the energy is absorbed by the soil for the first six frequencies ($f < 3.5$ Hz) which involve more than 99% of total mass (see § 3.1), and by the structure for higher frequencies.

3.3 Response spectrum analysis

The results of response spectrum analysis are expressed in terms of three reference parameters, able to synthetize global building response: shear force and bending moment at the base of the structure (S and M) and top acceleration a.

The analyses have been carried out by modal superposition, with SRSS (square root of sum of squares) convention to combine modal responses.

According to the assumptions of SIEMENS, reference earthquake has three components of acceleration in the three directions x, y, and z; maximum intensity of acceleration is 0.3 g for horizontal (x and y), and 0.2 g for vertical (z) excitation.

The three components are combined with the SRSS convention.

The seismic analyses have been performed by the finite element program MSC/NASTRAN [8], [9].

4. RELIABILITY ANALYSIS

Starting from deterministic evaluation of structural response parameter, scope of this section is the presentation of the reliability analysis on the same structure. This activity had two major difficulties:

- the new definition of input data in a probabilistic framework: the choice of an appropriate probability distribution has to be checked with significance to numerical testing;
- the calculation of the probability distribution of shear force and acceleration at the top requires the knowledge of the structural transfer function between variables and structural response.

The first step is managed by special probabilistic environment tool where variable definition, objective specification and probabilistic calculation are available menu.

The second step has required the set up of a special procedure to couple F. E. analysis program with the probability analysis tool. The presented procedure aims to evaluate preliminarly an explicit algebraic transfer function to be inserted in the final probability calculation, but many assumptions have been done and need to be discussed to minimize their impact on final results.

The above discussed steps are therefore discussed in a very schematic approach in the following.

4.1 Variables

The variables of the reliability problem have been identified as in the following:

- seismic intensity;
- shear modulus profiles;
- damping coefficients.

For the first variable, seismic intensity, a uniform distribution has been adopted (see fig. 9). This variable is voluntarily kept out of probabilistic analysis as it can be applied at the end of the calculation, having no influence on structural response evaluation. Moreover, as it will be explained better in the last chapter, its influence is greater than the other variables, and so it could hide the other contributions.

The variability of shear moduli for the five layers (G_i) is defined as follows;according to Siemens analysis of data variation, as shown in report [6]:

- mean (G_{ave})
- lower bound $G_{min} = 0.5\ G_{ave}$
- upper bound $G_{max} = G_0$ (shear modulus at smallest strains).

The soil profiles are plotted in fig. 10.

The corresponding probabilistic distribution for G has been assumed of beta type with extreme values on G_{min} and G_{max}.

For the damping coefficients (structure, β, and soil hysteretic, γ), the ranges of variability has been assumed with reference to bibliography [2], [11]:

	from	to
β:	60 %	140 % of β_{ave}
γ:	13.5 %	135 % of γ_{ave}

The distribution adopted for all variables (except seismic intensity) is the Beta type, which has demonstrated to be able to fit in the best way the random variables bounded and distributed asymmetrically. The formula that defines Beta probability density function is the following:

$$f(x) = \frac{(x-a)^{r-1}(b-x)^{t-r-1}}{(b-a)^{t-1}B(a,b)} \qquad a \le x \le b,\ r>0,\ t>r$$

$$\text{where } B(m,n) = \int_0^1 u^{n-1}(1-u)^{m-1}\,du$$

r and *t* parameters, which define function *f*(*x*) together with *a* and *b* (the boundaries), are determined with the following two conditions:

1) the most likely value of the distribution matches the experimental value;

2) standard deviation $s = \frac{1}{4}(b-a)$.

These conditions provide a realistic fit to the experimental distribution.

In figg. 11-12 Beta distribution functions of β and γ are plotted.

4.2 Response parameters

The response will be evaluated in terms of shear and moment at the foundation and acceleration at the top of the structure.

The random variables affect the values of the two target parameters, shear *S*, moment *M* and acceleration *a*, so that they also become affected by uncertainty(i.e. dependent random variables).

4.3 Structural transfer function evaluation

A certain number of dynamic analyses have been required, assuming for each one a different set of the variables according to a pre-defined scheme. The results of these analyses are used to determine the "transfer functions" for each of the three parameters *S*, *M* and *a*.
Modal analyses have been carried out starting from different set of values for shear moduli and damping coefficients chosen according with central composite design [12]: if x is the generic random variable, the following values of x have therefore been assumed:

$$
\begin{aligned}
x_1 &= \text{lower bound} \\
x_2 &= x_3 - \frac{(x_3 - x_1)}{\sqrt[4]{2^6}} \\
x_3 &= \frac{(x_1 + x_5)}{2} \\
x_4 &= x_3 + \frac{(x_5 - x_3)}{\sqrt[4]{2^6}} \\
x_5 &= \text{upper bound}
\end{aligned}
$$

Extreme values x_1 and x_5 represent the range of variability of the estimated variable.

Starting from the results of modal analyses, where only the first four[1] variables affect the results (shear moduli), the seismic response spectra analyses have been carried out.

β and γ affect the modal damping and therefore the structural response to the dynamic load.

Response spectrum analyses have given the variability measure of shear force and bending moment at the base of the structure, *S* and *M*, and top acceleration *a*. This variability allows to synthetize the behaviour of *S*, *M* and *a* with three polynomial expressions which approximate experimental results:

---

1 - The first layer has no variability in the shear modulus (see fig. 8), so G_1 is not taken as a random variable, but a constant.

$$S = S(G_2, G_3, G_4, G_5, \beta, \gamma)$$

$$M = M(G_2, G_3, G_4, G_5, \beta, \gamma)$$

$$a = a(G_2, G_3, G_4, G_5, \beta, \gamma)$$

The choice of the polynomial expression is a critical item to obtain a good approximation of the analysis results, affected by a higher variability of certain variables with respect to the others; therefore, also for the comparison with other expressions, the most suitable choice is a second order polynomial, lacking of the mixed terms (as it results from the comparison of the sum of squares of residuals for some different polynomials, shown in fig. 13):

$$P(G_2, G_3, G_4, G_5, b, g) = a_{11} + a_{21}G_2 + a_{22}G_2^2 + a_{31}G_3 + a_{32}G_3^2 + a_{41}G_4 + a_{42}G_4^2 + a_{51}G_5 + a_{52}G_5^2 + \\ + a_{61}b + a_{62}b^2 + a_{71}g + a_{72}g^2$$

The search for the best fit set of 13 coefficients has been carried out by the least squares method.

Three probability density functions for *S*, *M* and *a*, have been calculated from the probability functions of the six random variables G_2, G_3, G_4, G_5, β and γ.

The probabilistic analysis has been carried out with SESAM PROBAN [13], using Montecarlo method to sample random variables.

The results of the probabilistic analysis are shown in the diagrams in figg. 14, 15 and 16, in which the probability density functions of *S*, *M* and *a* are plotted. The statistic sensitivities of the parameters are presented in the following tables 3 and 4: they provide a measure of the influence of a single base variable with respect to a certain dependent random parameter (e.g. shear, moment or acceleration). The statistic sensitivity of a random parameter with respect to a variable is defined as the first derivative of this variable with respect to the parameter, evaluated giving a standard 10% increase to the variable, to make it comparable with other sensitivities. The derivative, obtained with hermitian interpolation of calculated points, is taken with its sign.

Sensitivity measure for mean			
	a	S	M
G_2	-1.36E-01	-6.18E+03	-2.37E+05
G_3	-4.75E-02	-2.28E+03	-1.19E+05
G_4	-4.69E-02	-2.73E+03	-1.15E+05
G_5	-1.33E-01	-4.00E+03	-2.18E+05
β	-5.20E-03	-4.70E+01	-6.01E+03
γ	-6.26E-02	-1.11E+03	-8.29E+04

Tab. 3

Sensitivity measure for standard deviation			
	a	S	M
G_2	3.53E-02	3.17E+03	6.31E+04
G_3	2.15E-03	1.53E+02	7.57E+03
G_4	1.84E-03	6.53E+01	6.84E+03
G_5	4.00E-02	1.02E+03	6.89E+04
β	4.85E-04	6.04E+01	1.70E+03
γ	8.44E-02	7.65E+02	8.54E+04

Tab. 4

From the analysis of this table the engineer can extract the mutual effect of uncertainties and therefore point out the most convenient variable to be investigated in order to keep the uncertainty of the response below the required level.

5. CONCLUSIONS

In the following, the main conclusions from the application point of view are discussed .

A) **Influence of the mean value of random variables G_2, G_3, G_4, G_5, β and γ on the mean value of S, M and a parameters**
The behaviour is governed by the sign of sensitivity factors for mean values of the parameters (see tab. 3); it can be interpreted in a univocal way for all variables (the sign is always negative): an increase of the mean value (which is the same as an increase of shear modulus or damping) determines a decrease of the mean value of shear, moment and acceleration. It has to be noted that this logical conclusion, based on mean values, does not have any probabilistic contribution: it could be reached even in a deterministic approach with a sensitivity analysis in a single step.

B) **Variables with the highest influence on the structure response**
The variable with major contributions on S, M and a are shear moduli: G_2, followed by G_5; G_3, G_4 and γ (hysteretic damping of soil) have less influence, and the influence of β (hysteretic damping of structure) is almost negligible.
This result can be understood taking in mind that the second layer is the nearest to the building foundation and that the fourth is the thickest one.

C) **Random variables whose uncertainty has to be reduced**
From the analysis of the sensitivity measures of standard deviation of S, M and a with respect to the standard deviation of six random variables (see tab. 4), it can be observed that the sign of sensitivity measures is always positive: a reduction of uncertainty in the random variables reduce the uncertainty of the response. Furthermore a comparison between the sensitivities shows that the influence of the variables on the three parameters is almost the

same as described at point 2, remarking only that here the variable with major contributions on a is γ (hysteretic damping of soil).

D) Influence of seismic intensity on the parameters

The random variable "Seismic Intensity" (S.I.) (defined as a multiplying coefficient of maximum soil accelerations, described at § 3.3) has been voluntarily kept out of probabilistic analysis. This variable is infact perfectly separable from the others for the structure response because of the linear hypothesis, and it affects the results in a very different way. In the graph of fig. 17 it can be seen the influence of the introduction of this parameter in the analysis on the probability distribution.

The influence is strong: the other variable contribution looks like a "background noise"; in a linear approach to seismic analysis infact seismic intensity affects almost proportionally the results. As an example, sensitivity measure of the acceleration (mean) for the upper bound of S.I. is the greatest (0.447) and it's more than three times (in absolute value) of the sensitivity measure greatest of others,G_2 (-0.136).This evaluation has not been extended to the analysis of equivalent spring variation due to seismic level. As a reference, this additional effect has been subsequently quantified in ± 9% of stiffness variation when ± 40% of seismic intensity has been applied. Peak acceleration variation are expected of the same order.

As a general conclusion, this analysis has shown that the most efforts in research must be spent to estimate seismic intensity of the site (e. g. geophysical prospectings and analyses) with the best accuracy.

According to the limit values for S,M,a failure probabilities, as specified for structural analysis and component qualifications, the analysis discussed in this report can provide the engineers with a priority table to drive geotechnical and structural investigations on Paks site.

These limits on failure probabilities are dependent from $\overline{S}$, $\overline{M}$, $\overline{a}$ values which are the maximum allowable values, according with the selected structural and for qualifications limit states (fig.18).

If the probability of exceeding $\overline{S}$, $\overline{M}$, $\overline{a}$ are greater than the required limits, two major actions are foreseen:

- reduction of variable uncertainty to reduce response uncertainty

- strengthen of the structure.

The results discussed with present report could easily be applied to this final evaluation, provided $\overline{S}$, $\overline{M}$, $\overline{a}$ values are specified according with site and structural details.

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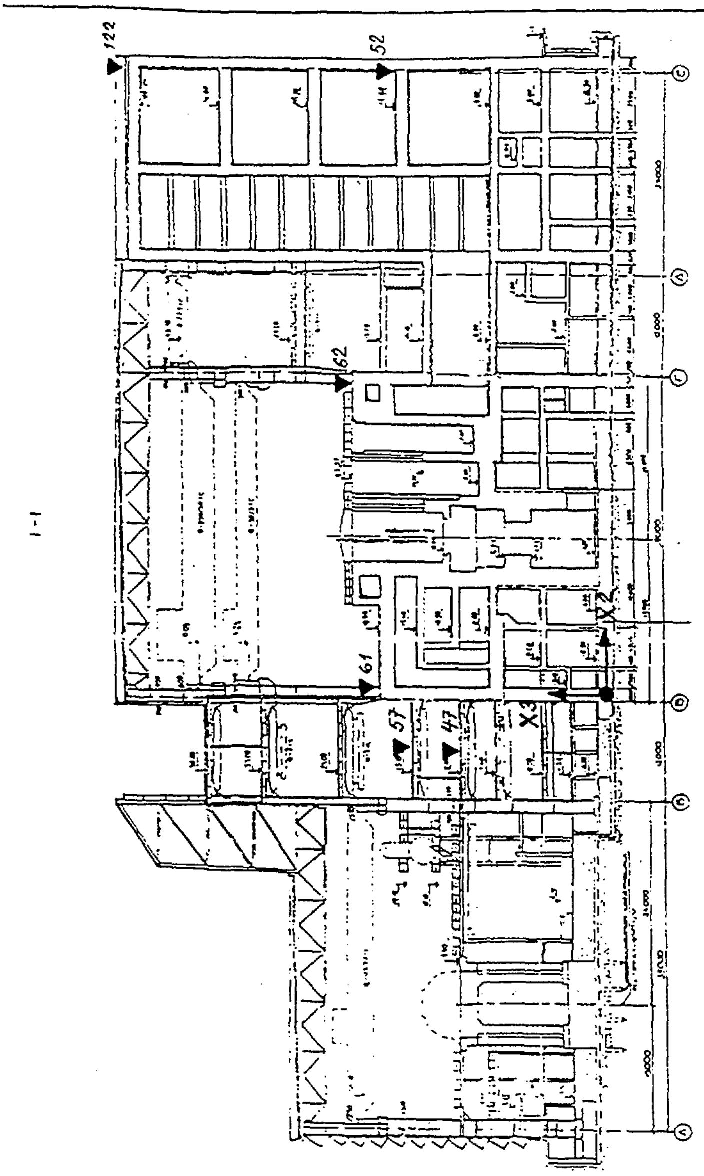

Figure 0 Main Building

Siemens AG - Bereich Energieerzeugung (KWU)
Siemens AG - Power Generation Group (KWU)

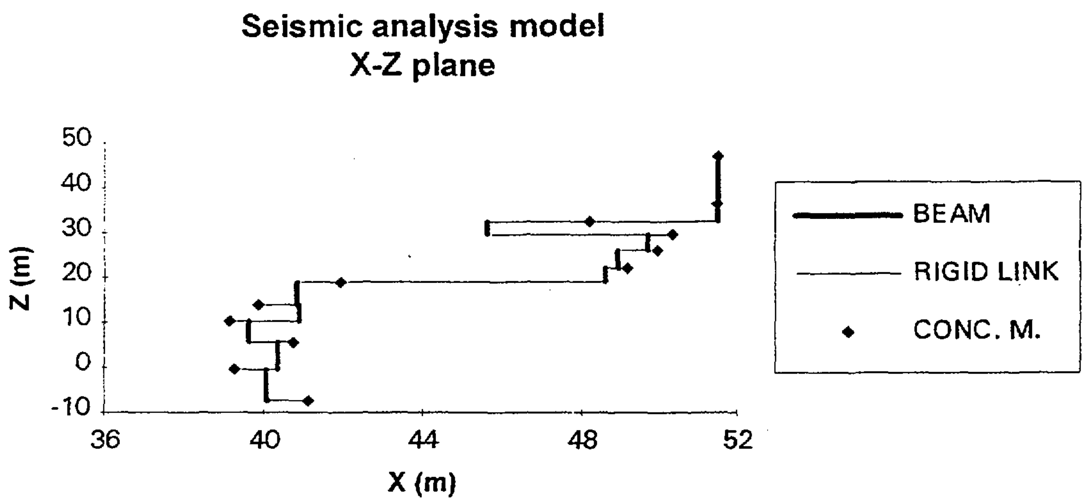
Seismic analysis model
X-Z plane
Z (m)
50
40
30
20
10
0
-10
36
40
44
48
52
X (m)
BEAM
RIGID LINK
CONC. M.

Fig. 1

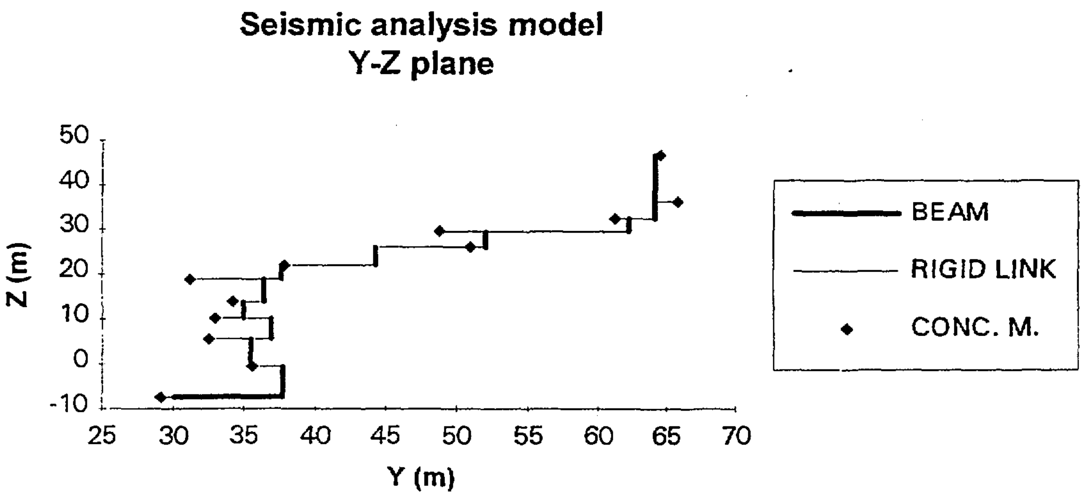
Seismic analysis model
Y-Z plane
Z (m)
50
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Fig. 2

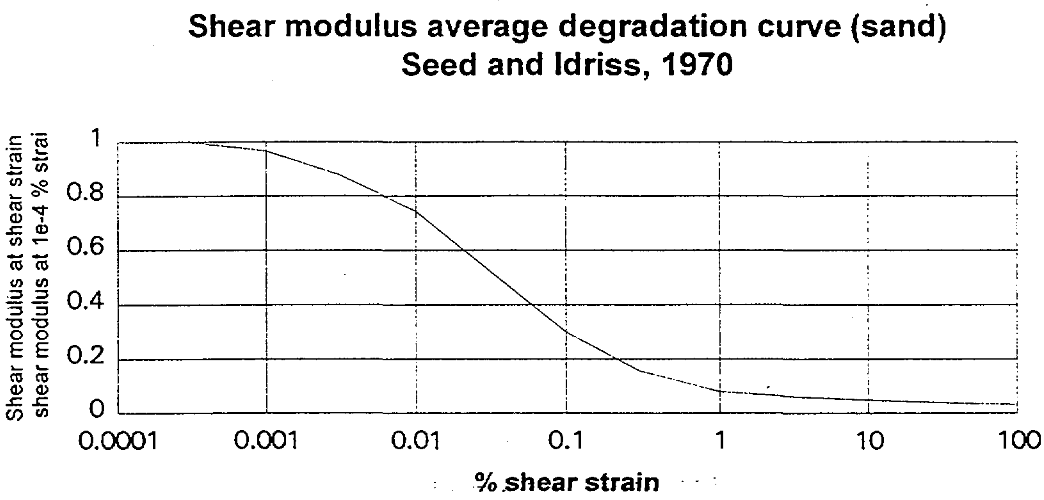

Shear modulus average degradation curve (sand)
Seed and Idriss, 1970
Shear modulus at shear strain
shear modulus at 1e-4 % strai
1
0.8
0.6
0.4
0.2
0
0.0001
0.001
0.01
0.1
1
10
100
% shear strain

Fig. 3

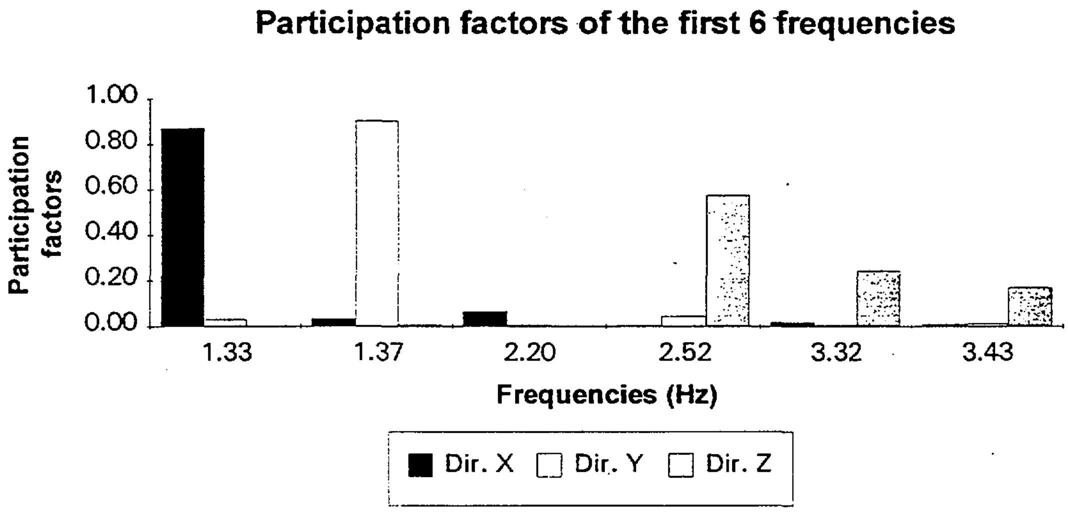
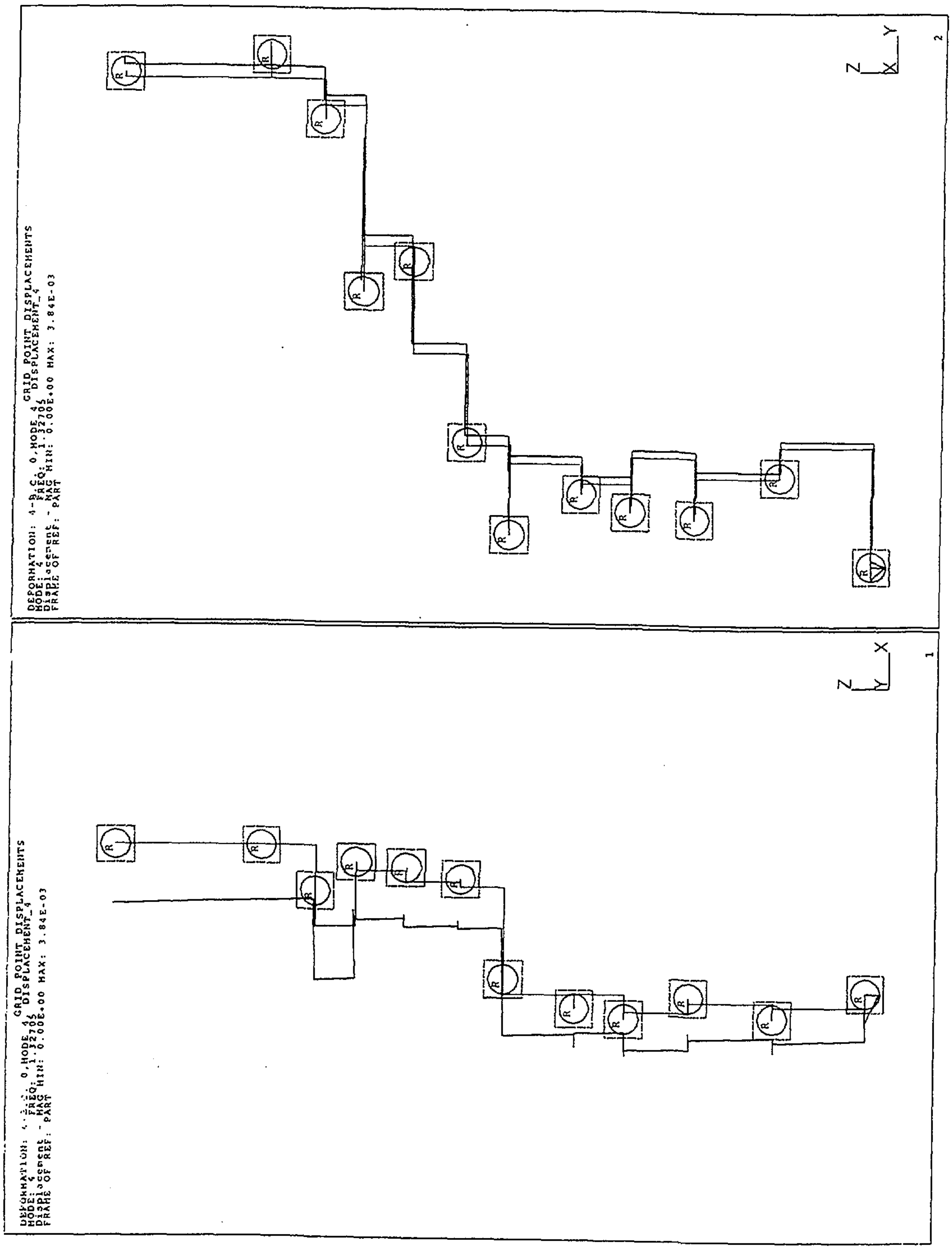
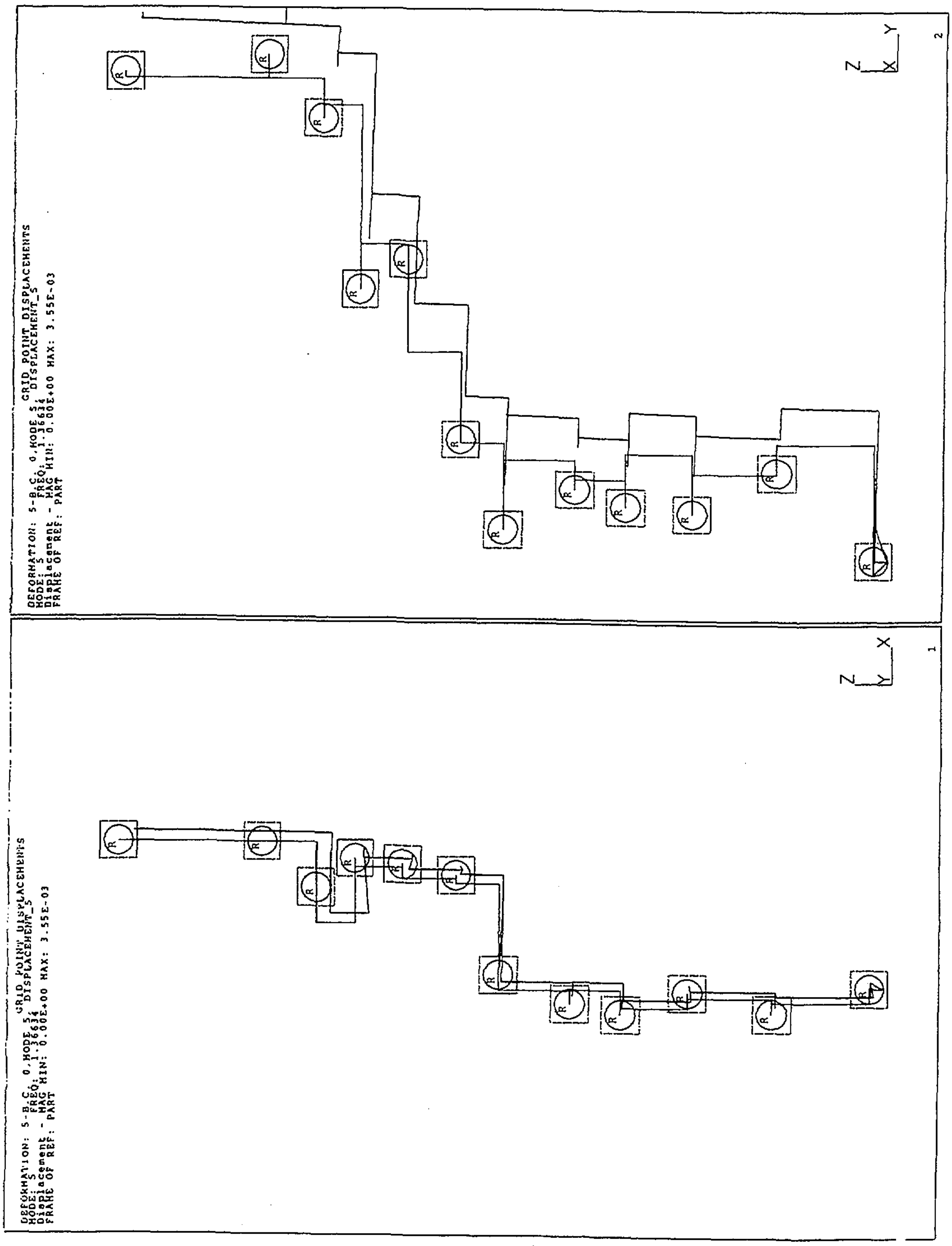

Participation factors of the first 6 frequencies
Participation factors
1.00
0.80
0.60
0.40
0.20
0.00
1.33
1.37
2.20
2.52
3.32
3.43
Frequencies (Hz)
Dir. X
Dir. Y
Dir. Z

Fig. 4



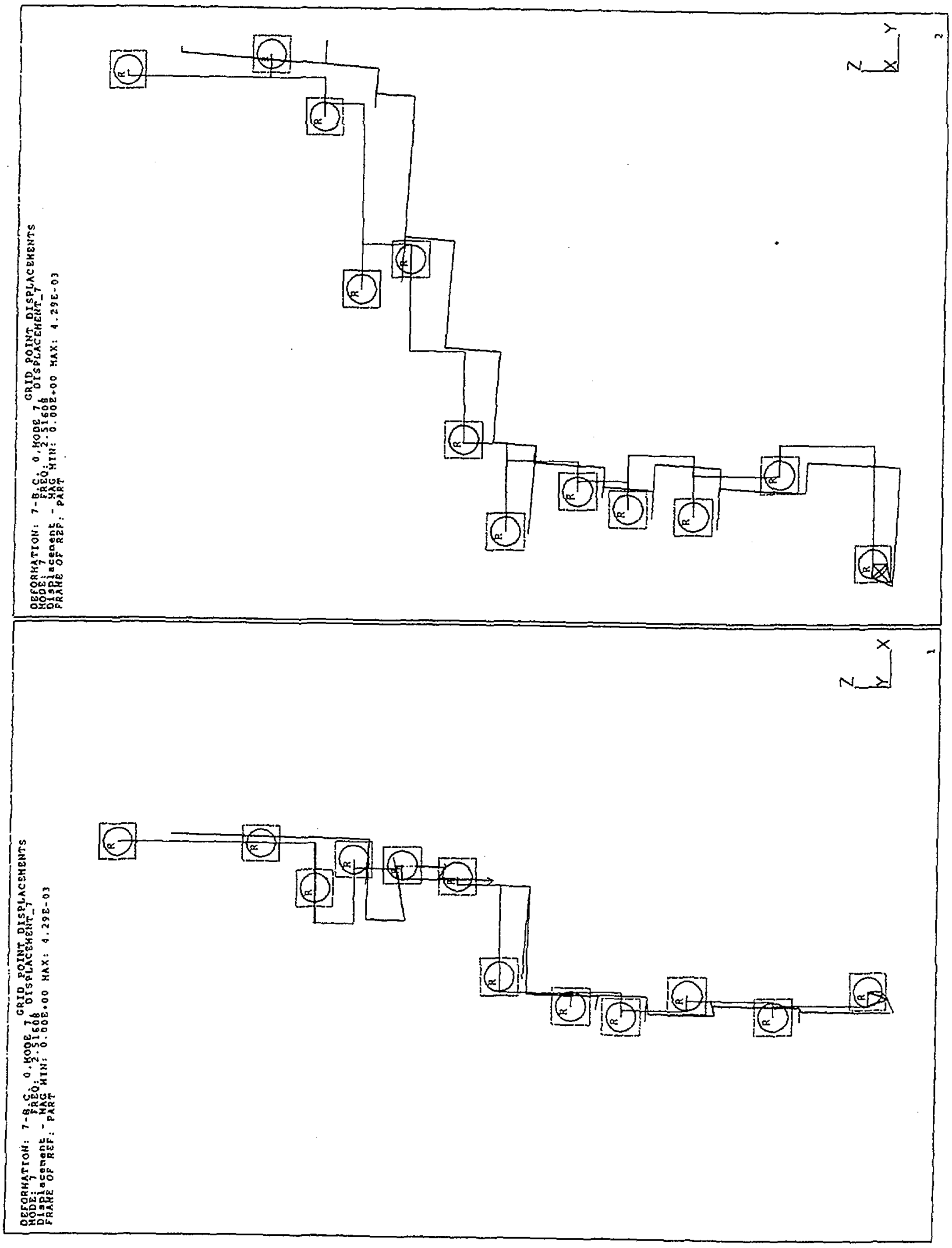
Modal deformation n.4 - f=1.32 hz

Fig. 5



Modal deformation n.5 - f=1.37 hz

Fig. 6



Modal deformation n.7 - f=2.52 hz   Fig. 7

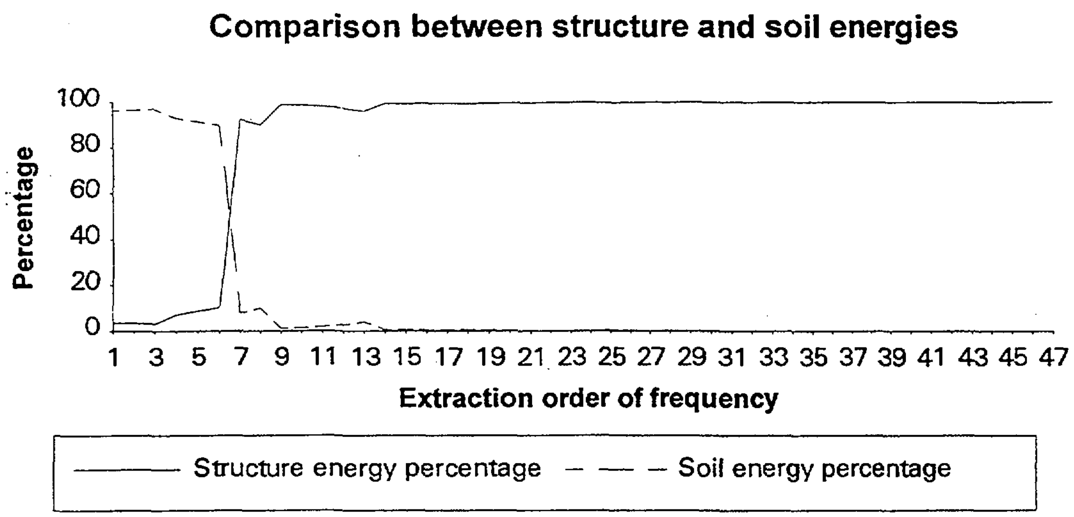

Comparison between structure and soil energies
Percentage
100
80
60
40
20
0
1 3 5 7 9 11 13 15 17 19 21 23 25 27 29 31 33 35 37 39 41 43 45 47
Extraction order of frequency
Structure energy percentage
Soil energy percentage

Fig. 8

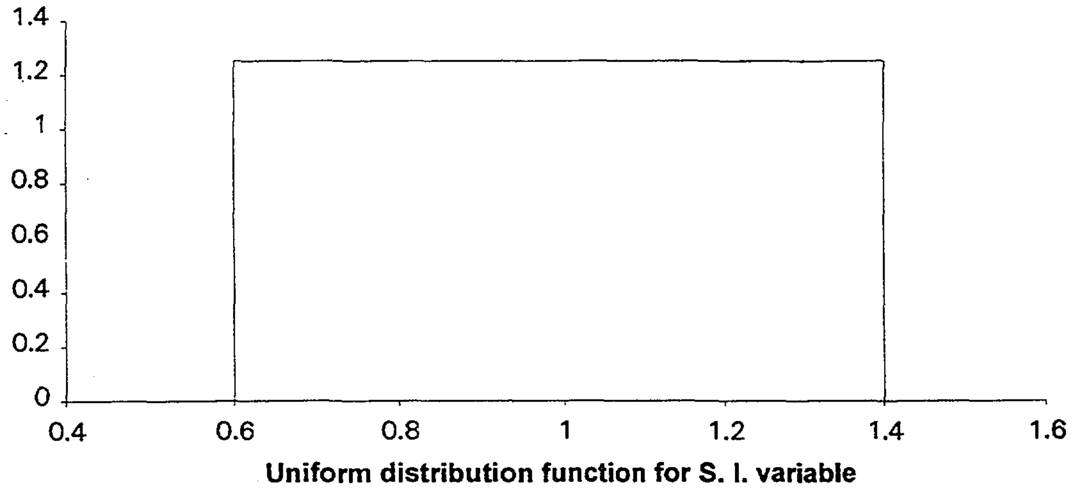

1.4
1.2
1
0.8
0.6
0.4
0.2
0
0.4
0.6
0.8
1
1.2
1.4
1.6
Uniform distribution function for S. I. variable

Fig. 9

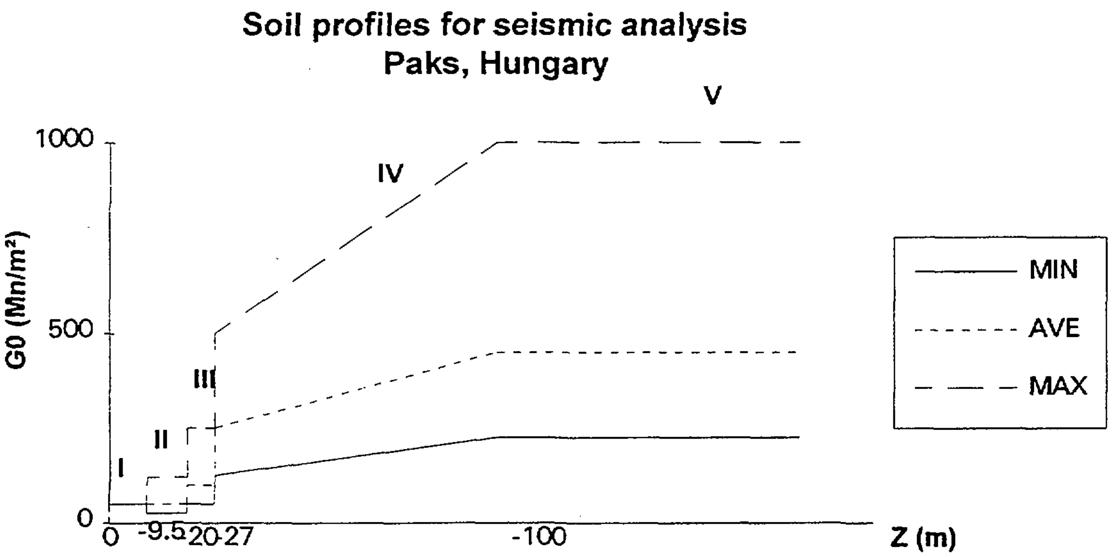

Soil profiles for seismic analysis
Paks, Hungary
V
IV
III
II
I
1000
500
0
G0 (Mn/m²)
0
-9.5
20
27
-100
Z (m)
MIN
AVE
MAX

Fig. 10

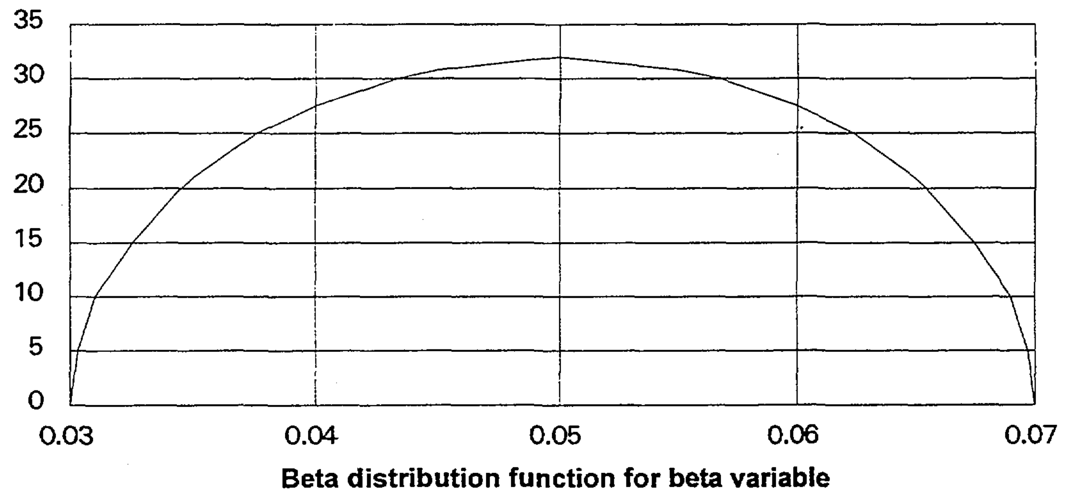

35
30
25
20
15
10
5
0
0.03
0.04
0.05
0.06
0.07
Beta distribution function for beta variable

Fig. 11

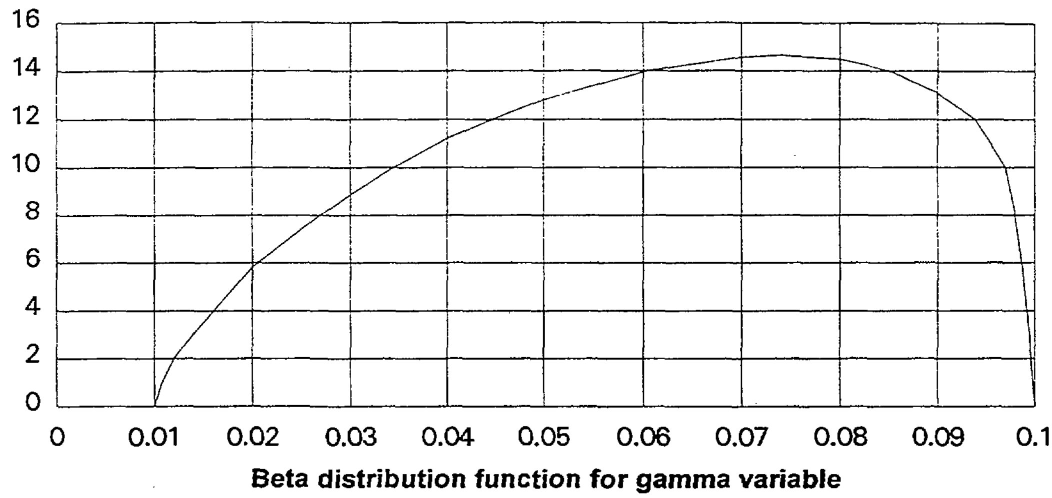

16
14
12
10
8
6
4
2
0
0
0.01
0.02
0.03
0.04
0.05
0.06
0.07
0.08
0.09
0.1
Beta distribution function for gamma variable

Fig. 12

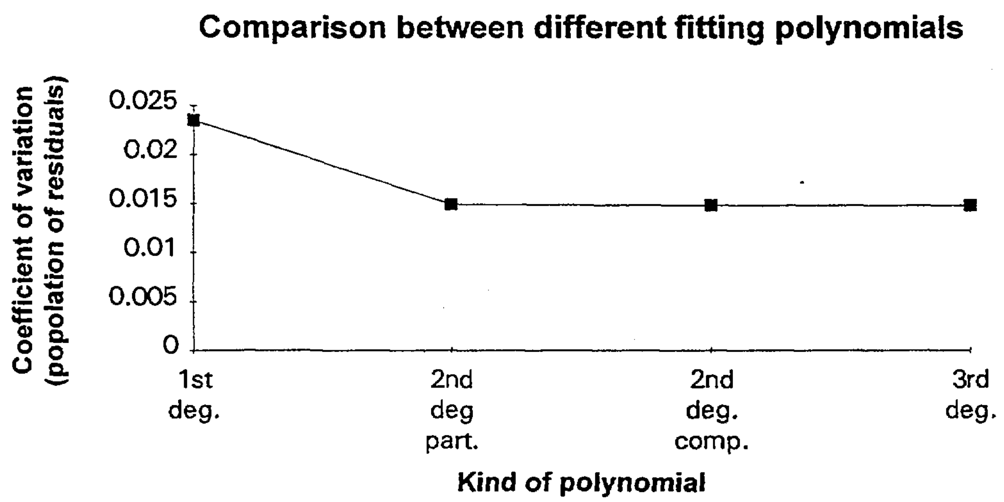

Comparison between different fitting polynomials
Coefficient of variation (popolation of residuals)
0.025
0.02
0.015
0.01
0.005
0
1st deg.
2nd deg part.
2nd deg. comp.
3rd deg.
Kind of polynomial

Fig. 13

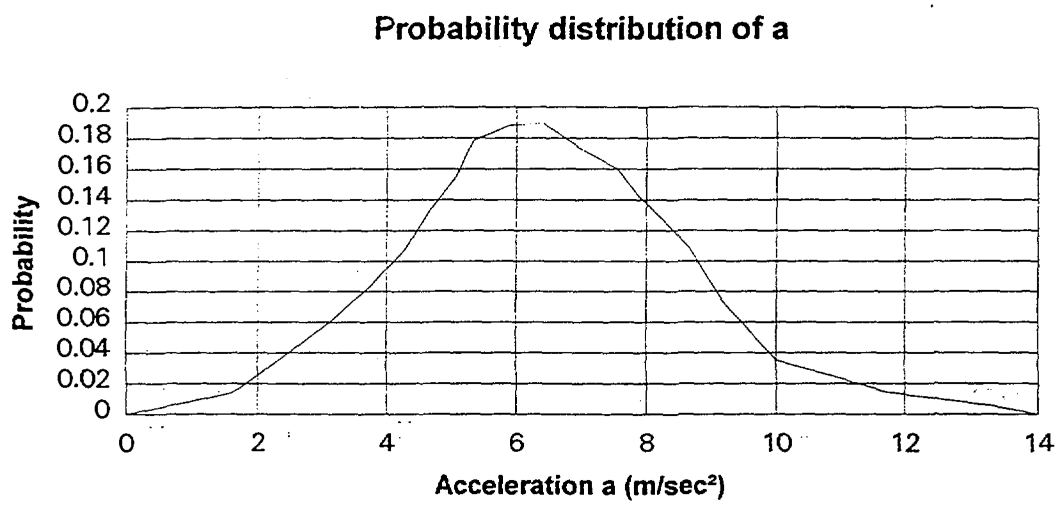

Probability distribution of a
Probability
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
2
4
6
8
10
12
14
Acceleration a (m/sec²)

Fig. 14

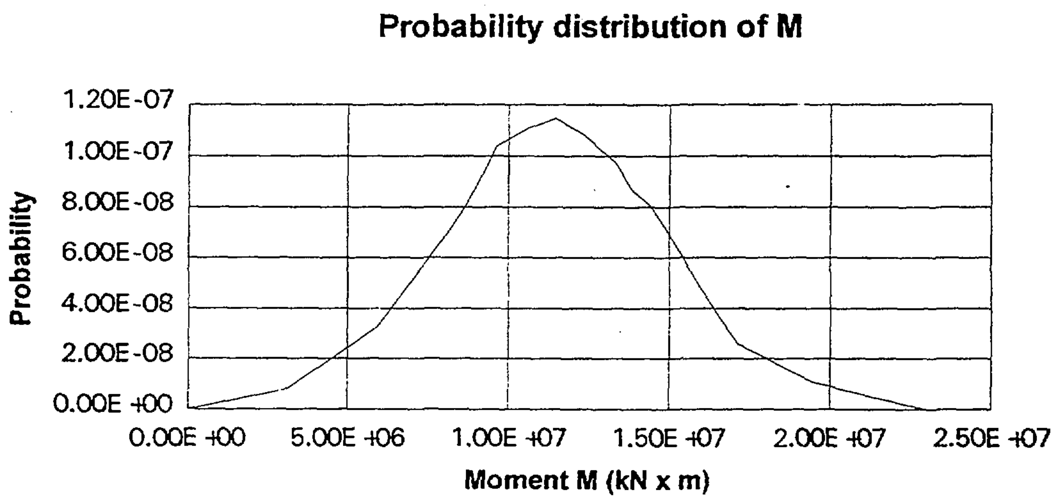

Probability distribution of M
Probability
1.20E-07
1.00E-07
8.00E-08
6.00E-08
4.00E-08
2.00E-08
0.00E +00
0.00E +00
5.00E +06
1.00E +07
1.50E +07
2.00E +07
2.50E +07
Moment M (kN x m)

Fig. 15

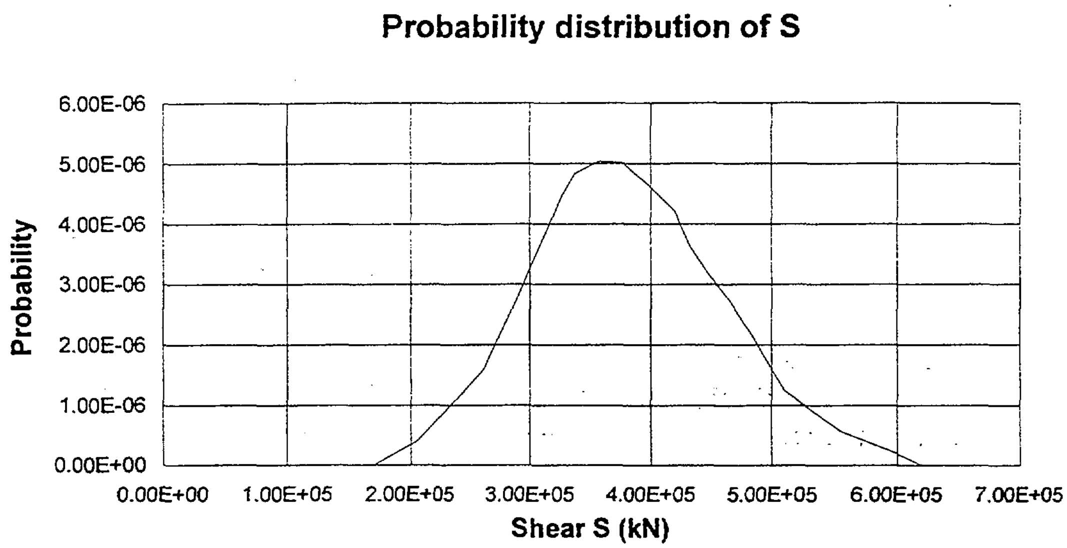

Probability distribution of S
Probability
6.00E-06
5.00E-06
4.00E-06
3.00E-06
2.00E-06
1.00E-06
0.00E+00
0.00E+00
1.00E+05
2.00E+05
3.00E+05
4.00E+05
5.00E+05
6.00E+05
7.00E+05
Shear S (kN)

Fig. 16

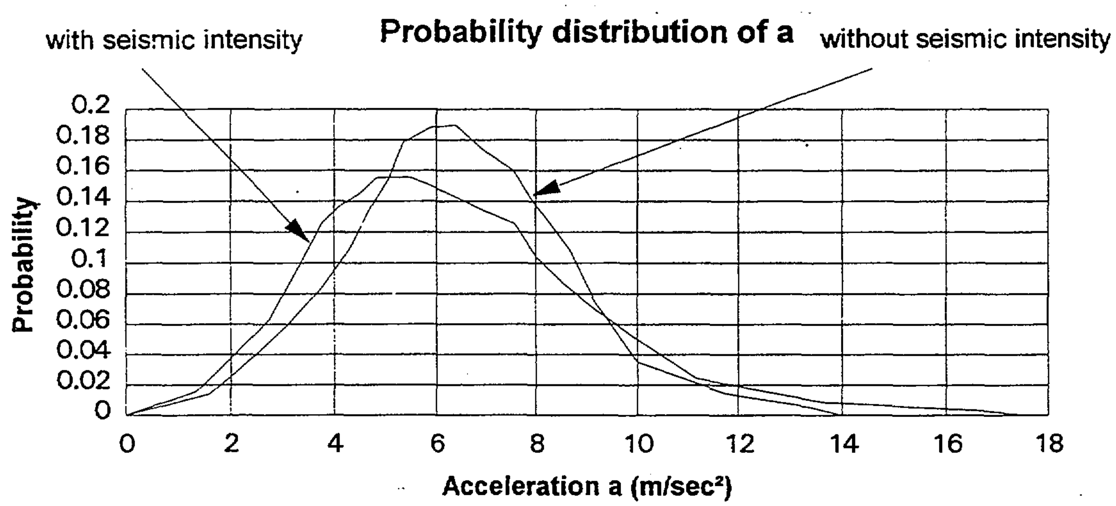

with seismic intensity
Probability distribution of a
without seismic intensity
Probability
0.2
0.18
0.16
0.14
0.12
0.1
0.08
0.06
0.04
0.02
0
0
2
4
6
8
10
12
14
16
18
Acceleration a (m/sec²)

Fig. 17

Probability distribution of a

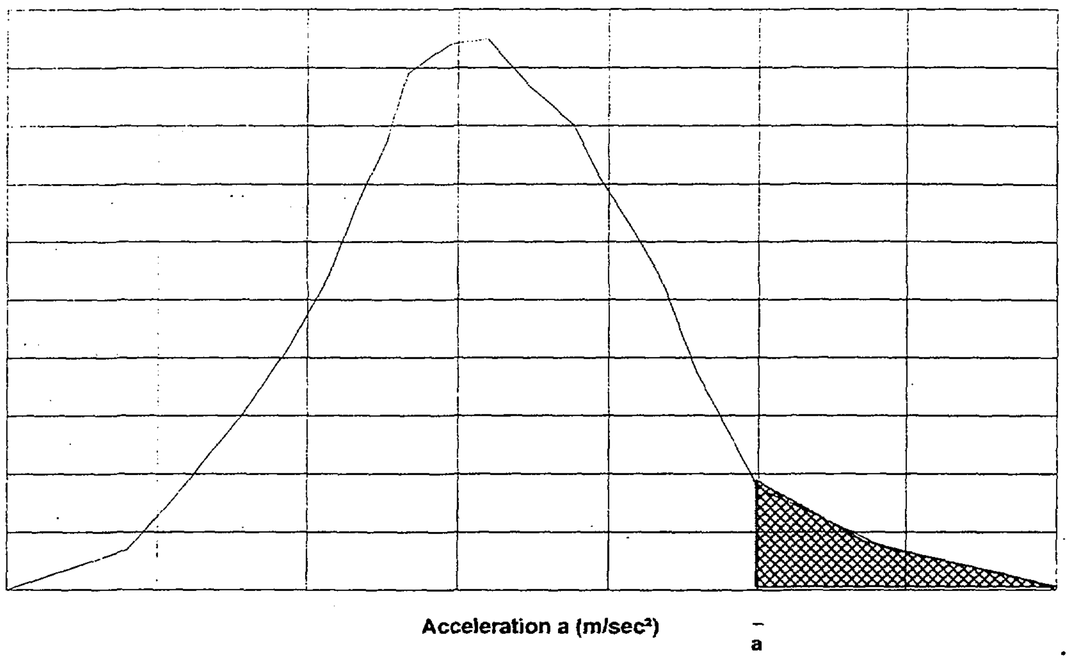


Fig. 18